MASTER DRAINAGE PLAN
WESTERN DEVELOPMENT LANDS
VILLAGE OF RICHMOND

FOR

RICHMOND VILLAGE (SOUTH) LIMITED

CITY OF OTTAWA

PROJECT NO.: 11-468

NOVEMBER 2013 – REV 3

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1.0 INTRODUCTION

Richmond Village (South) Limited has retained David Schaeffer Engineering Ltd. (DSEL) to prepare a Stormwater Management Report in support of their application for draft plan of subdivision.

This study constitutes a resubmission of previously submitted work and addresses the City of Ottawa comments on the Stormwater Management Report submitted by DSEL for Richmond Village (South) Limited in November 2012. The City and Agency comments received to date are summarized in ***Appendix A***.

The November 2012 stormwater management report presented facility treatment types, locations, and provided a recommended stormwater facility solution for the subject lands. This study expands upon the previously recommended solution. Furthermore, this report outlines various storm conveyance alternatives and presents a detailed review of the recommended conveyance solution.

The objective of this report is to provide sufficient detail with respect to the stormwater management system including minor and major system conveyance and stormwater facilities to support the Richmond Village (South) Limited's draft plan of subdivision application. Furthermore, this study provides a stormwater solution for the adjacent development lands.

1.1 Site Context

The Village of Richmond is located within the City of Ottawa planning boundary and is approximately 10km south of Stittsville and 12km west of Manotick, as illustrated on ***Figure 1***.

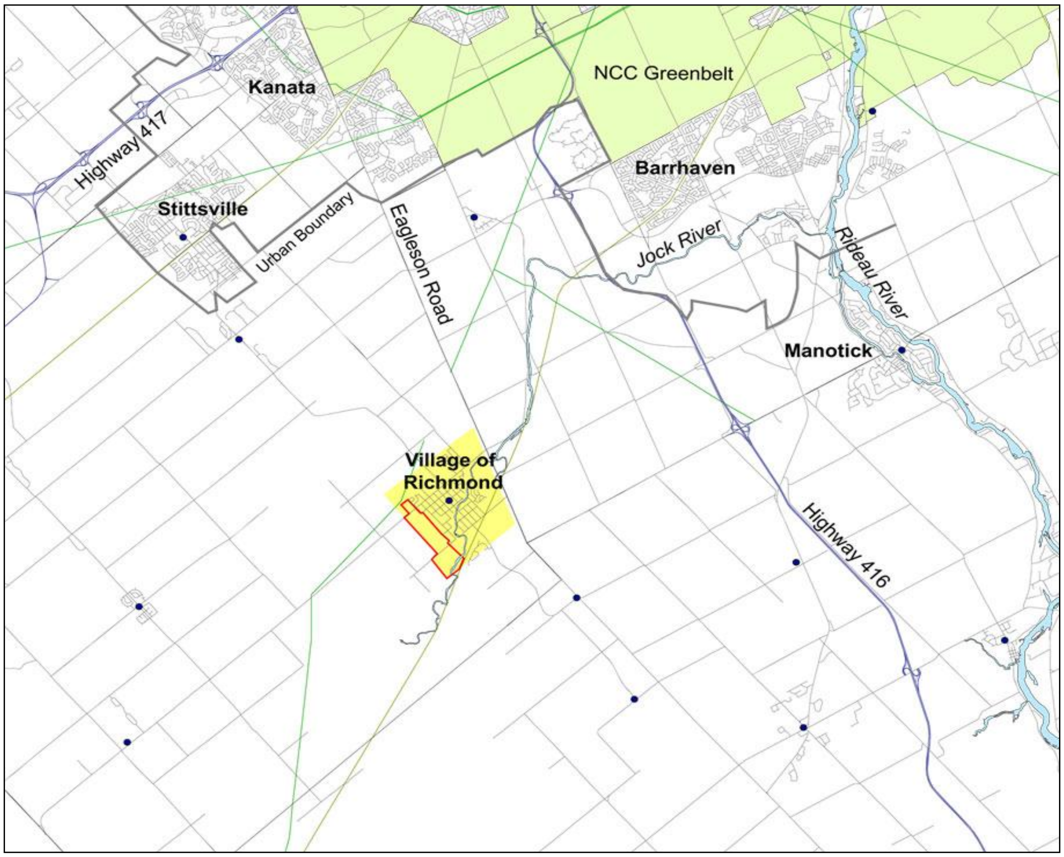

Figure 1: Location of the Village of Richmond

The subject lands lie along the western perimeter of Village of Richmond planning boundary. The subject land extends north of Perth Street and south of Perth Street to the Jock River, as illustrated on ***Figure 2***. The existing property is currently being farmed and is relatively flat with slopes ranging from 0.1% to 0.5%.

The majority of the subject lands are within the Van Gaal sub-watershed which is a tributary to the Jock River. The sub-watershed area is approximately 1,115 ha and is mostly undeveloped, consisting of wooded, wetlands, and agricultural lands fallow, and row crop areas.

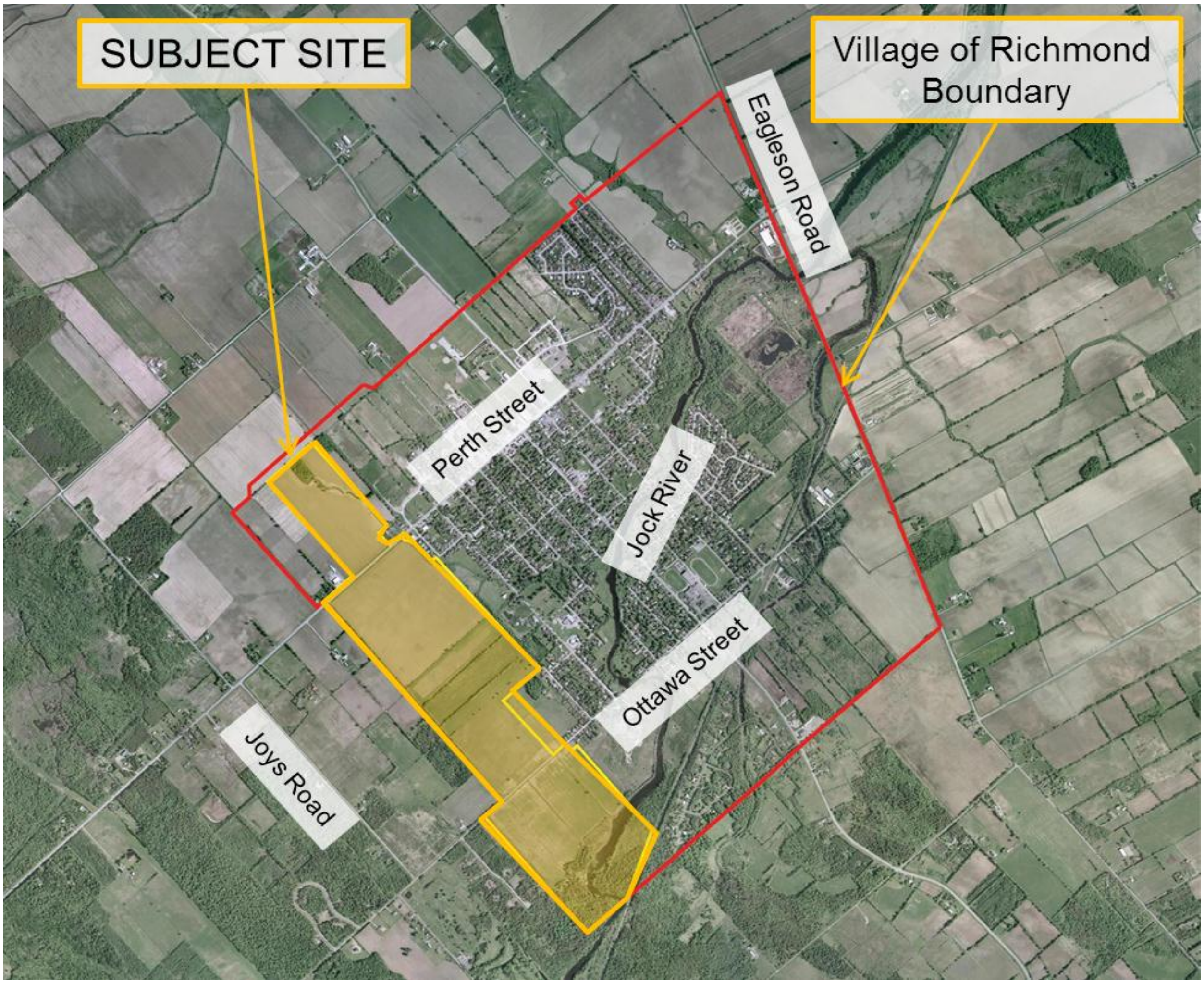

Figure 2: Site Context

1.2 Required Permits / Approvals

Richmond Village (South) Limited is subject to the following permits and approvals:

1.2.1 City of Ottawa

The City of Ottawa is required to approve the engineering design drawings and reports for the Richmond Village (South) Limited Subdivision. The City of Ottawa must review and sign off on the design and forward to the Ministry of the Environment (MOE) for their transfer of review program.

1.2.2 Ministry of the Environment

The MOE is required to review the engineering design and issue Environmental Compliance Approvals (ECA) for Sanitary and Storm Sewers and Stormwater

Management. The ECA is a new instrument of Environmental Approval which replaces the Certificate of Approval (CofA).

Richmond Village (South) Limited subdivision will not require an Environmental Compliance Approval (ECA) for watermains. The City will review the watermains on behalf of the MOE.

The MOE will also be required to issue an approval for any proposed temporary sedimentation systems such as on-site ditches and / or temporary sediment ponds. To allow dewatering of the subject site, a Permit to take Water (PTTW) is required.

1.2.3 Rideau Valley Conservation Authority (RVCA)

Concurrent with the City and MOE approvals, approvals are required from the Rideau Valley Conservation Authority as it relates to Ontario Regulation 174/06 "The Development, Interference with Wetlands and Alterations to Shorelines and Watercourses" for the ditch creation(s), alteration(s) and / or enclosure(s).

There are two floodplain amendments being proposed to support the proposed development area. Additional information regarding the proposed amendments is discussed under ***Section 3.2***.

1.2.4 Department of Fisheries and Oceans (DFO)

Approval will be required from the RVCA, acting as an agent for the DFO, regarding the infilling and re-alignment of the Van Gaal Drain north of Perth Street and the closure of the existing ditch(es) located between Perth Street and Ottawa Street.

1.3 Study Process

The Municipal Class Environmental Assessment is an approved provincial planning and design procedure developed to ensure that the potential social, economic, and natural environmental effects are considered in undertaking certain projects. The approach is provided in the Municipal Engineers Association Municipal Class Environmental Assessment document prepared in October 2000 and amended in 2007. The Class EA planning process is a self-directed process (by the proponent), which represents an acceptable procedure for municipalities to carry out individual assessments for most municipal water and wastewater projects in Ontario. The Class EA deals with various aspects of municipal servicing projects (water and wastewater), including:

- Maintenance and operational activities
- Reconstruction and modification of existing supply sources/treatment facilities and distribution/collection systems
- Construction of facilities

Since water and wastewater projects undertaken by municipalities vary in their environmental impact, projects are further classified in terms of schedules:

> Schedule A (Pre-Approved Activities) projects are limited in scale, have minimal adverse environmental effects and include a number of municipal maintenance and operational activities. These projects are pre-approved and may proceed to implementation without following the Class EA planning process. Schedule A projects generally include normal or emergency operational and maintenance activities.
>
> Schedule A+ (Pre-Approved Activities with Public Advisory) projects are similar to Schedule A projects but include projects where it is appropriate to inform the public of the municipal infrastructure project(s) being constructed or implemented in their area.
>
> Schedule B projects have the potential for some adverse environmental effects. The proponent is required to undertake a screening process, involving mandatory contact with directly affected public and relevant review agencies, to ensure that they are aware of the project and that their concerns are addressed. If there are no outstanding concerns, then the proponent may proceed to implementation. Schedule B projects generally include improvements and minor expansions to existing facilities.
>
> Schedule C projects have the potential for significant environmental effects and must proceed under the full planning and documentation procedures specified in the Class EA document. Schedule C projects require that an Environmental Study Report be prepared and filed for review by the public and review agencies. Schedule C projects generally include the construction of new facilities and major expansions to existing facilities.

Note: There is an appeal mechanism for Schedule B and Schedule C projects – members of the public, interest groups and/or review agencies may request the Minister or delegate to require a proponent to comply with Part II of the EA Act before proceeding with the undertaking (the Minister or delegate will determine if this is necessary). Schedule A and Schedule A+ projects are pre-approved and there is no ability for the public to request a Part II Order (public comments on these projects should be directed to local municipal councils).

The selection of the applicable schedule is determined for certain projects by the environmental impact and other projects are determined to fall within a schedule based on cost.

Richmond Village (South) Limited is the proponent and lead of the Stormwater Management and Drainage Plan report. As such, the lead proponent shall be subject to the terms and conditions of this Class EA.

There are a number of stormwater project types identified as wastewater projects under Schedule "A', "A+" and "B" projects, including those that are intended to:

Schedule A

1. Construction of stormwater management facilities which are required as a condition of approval on a consent, site plan, plan of subdivision or condominium which will come into effect under the Planning Act prior to the construction of the facility.

2. Any project which would otherwise be subject to this Class EA and has fulfilled the requirements outlined in Section A.2.9 of this Class EA and for which the relevant Planning Act documents have been approved or have come into effect under the Planning Act, R.S.O. 1990, Chapter P.13, as amended.

Schedule A+

1. Establish, extend, or enlarge a sewage collection system and all necessary works to connect the system to an existing sewage or natural drainage outlet, provided all such facilities are in either an existing road allowances or an existing utility corridor, including the use of Trenchless Technology for water crossings.

Schedule B

2. Establish new stormwater retention/detention ponds and appurtenances or infiltration systems including outfalls to receiving water body.

3. Enclose a watercourse in a storm sewer.

Section A.1.3 of the Class EA document discusses the application of the EA Act for private sector development. Projects undertaken by the private sector developers which are designated as an undertaking to which the Ontario EA Act applies (i.e. Schedule C project that are servicing residential developments – see Ontario Regulation 345/93) are subject to all requirements of this Class EA document.

The potential stormwater management projects to support the Richmond Village (South) Limited development are Schedule A, A+ and B projects. As this is a private sector lead exercise, subject to Planning Approval with no identified Schedule C undertakings, the projects fall under Schedule A undertaking. However, the Class EA document encourages municipalities to consider requiring developers to fully consider appropriate alternatives even if the project is exempt under Ontario Regulation 345/93.

In this regard, Mattamy's Stormwater Management and Drainage Plan considered alternatives and followed Phases 1 and 2 of the Class EA process in order to determine the preferred stormwater scheme for the Richmond Western Development lands. There was no formal filing of the document under the Class EA provisions. However, the document had gone through a public process as a supporting study of the Official Plan Amendment application.

1.4 Public and Agency Consultation

Numerous public and agency consultations took place between April 2008 and present. The Stormwater Management Report submitted by DSEL for Mattamy Richmond Lands in March 2010 summarizes the meeting results and times under sub-section 1.3. This previous summary is provided in ***Appendix A*** for ease of reference.

The Stormwater Management Report is submitted as supporting documentation for the Planning Act application. The public will have an opportunity to review the documentation through the City of Ottawa Development Application Review process and the public may submit appeals / objection to the Ontario Municipal Board regarding the Planning Act application. The Stormwater Management Report has been conducted in general accordance with the steps/phases as outlined in the Municipal Class EA Process.

2.0 GUIDELINES, PREVIOUS STUDIES, AND REPORTS

2.1 Existing Studies, Guidelines, and Reports

The following studies were utilized in the preparation of this report.

- **Ottawa Sewer Design Guidelines,**
  City of Ottawa, October 2012.
  (City Standards)
  - **Technical Bulletin ISD-2012-1**
    City of Ottawa, January 31, 2012.
    (*ISD-2012-1*)
- **Stormwater Planning and Design Manual,**
  Ministry of the Environment, March 2003.
  (SWMP Design Manual)
- **Ontario Building Code Compendium**
  Ministry of Municipal Affairs and Housing Building Development Branch, January 1, 2010 Update
  (OBC)
- **Hydrology Report – Jock River Flood Risk Mapping**
  Rideau Valley Conservation Authority, July 2004.
- **Hydraulics Report – Jock River Flood Risk Mapping**
  Rideau Valley Conservation Authority, November 2004.
- **Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond**
  Rideau Valley Conservation Authority, November 2009.
- **Stormwater Management and Drainage Plan for the Mattamy Richmond Lands**
  DSEL, JFSA, AECOM, and Kilgour & Associates Ltd., March 2010.
  (March 2010 SWM Report)
- **Village of Richmond Community Design Plan**
  City of Ottawa, July 2010.
  (CDP)
- **Preliminary Geotechnical Investigation Report**
  Jacques Whitford, June 22, 2007.
  (Geotechnical Study)

3.0 EXISTING CONDITIONS

3.1 Geotechnical

Jacques Whitford carried out a Geotechnical Investigation of the subject property in June 2007, included in ***Appendix B***. The following summarizes the results of their investigation.

North of Perth Street, the soils consist of a thick deposit of clay overlying a till deposit overlying inferred bedrock. Bedrock is anticipated at depths in excess of 6 m below ground surface to the north of Perth Street and becoming shallower to the south of Perth Street.

Between Perth and Ottawa Street the soils consist of a thin deposit of clay overlying a sandy silt deposit over a till deposit over inferred bedrock. Bedrock is anticipated at depths between 3 m to 4 m below ground surface.

South of Ottawa Street the soils consist of a deposit sandy silt over a till deposit over inferred bedrock. Bedrock is anticipated at depths ranging from greater than 4 m to less than 1 m below ground surface.

A compressible deposit of clay was encountered within the northern section of the site. Due to the compressible nature of the clay, grade raises over sections of the site should be restricted to minimize total settlements. The ***Table 1*** summarizes the preliminary grade raise restrictions for the site. ***Drawing 1 – Constraints Plan*** depicts the grade raise constraint areas.

Table 1
Summary of Maximum Grade Raise Constraints

Site Area	Maximum Grade Raise above Existing Site Grades
PIN 0062, 0061; North of Perth Street	1.0m
PIN 0285, 0286; Parcel to the south of Perth Street	1.5m
PIN 0287; Parcel north of Ottawa Street	2.0m
PIN 0714, 0746, 0047, 0075; Parcels north and south of Ottawa Street	4.0m

3.2 Regulatory Floodplain

The Jock River and Van Gaal Drain are adjacent to the subject lands and have established regulatory floodplains that are subject to modifications approved through the CDP process. The following sections describe the regulatory floodplain limits and the amendment process.

3.2.1 Jock River

The Rideau Valley Conservation Authority completed floodplain mapping for the Jock River in November 2004.

Figure 3 was extracted from the Jock River Flood Risk Map and illustrates a significant portion of the subject lands south of Ottawa Street within the Regulatory Flood Limit.

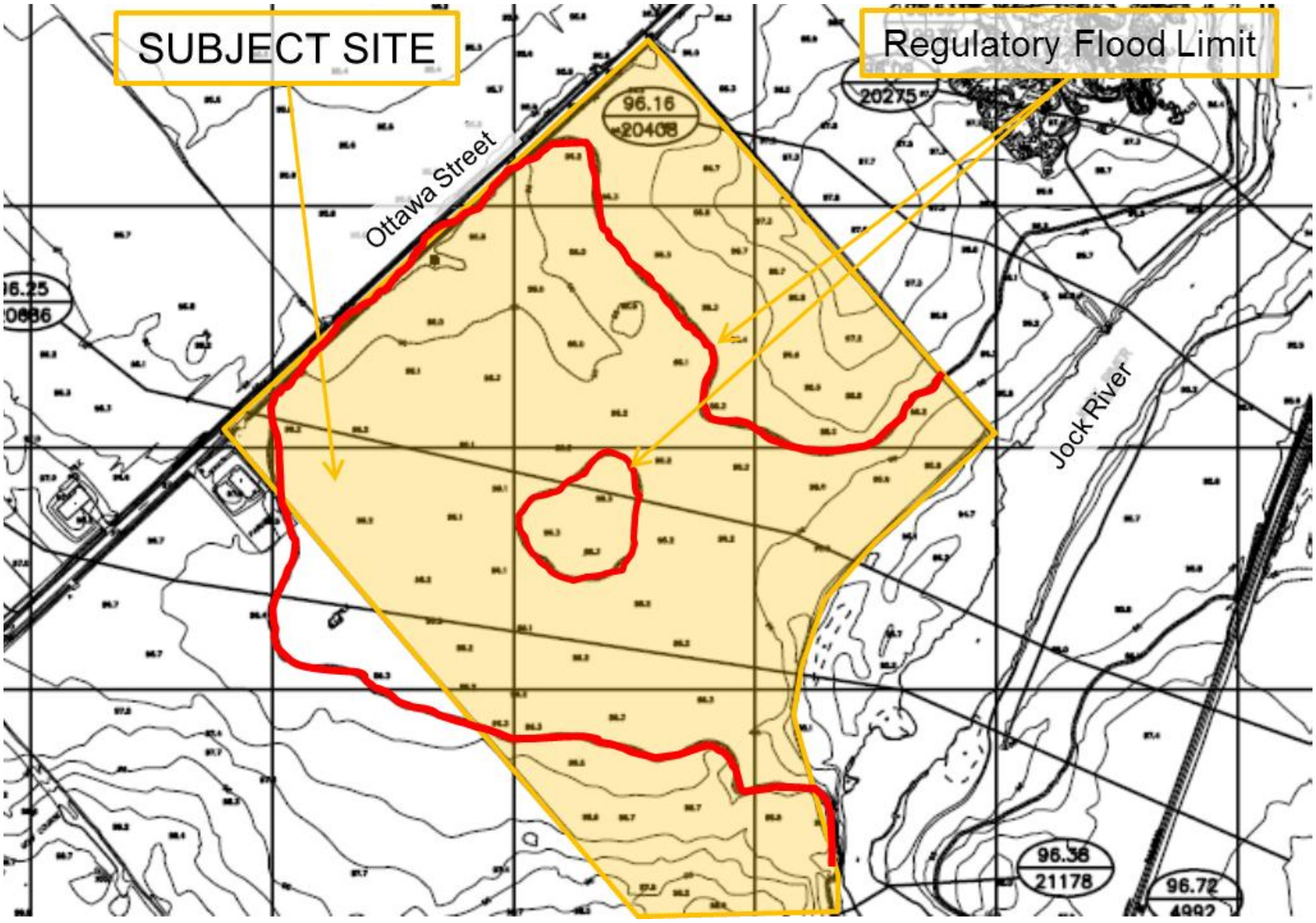

Figure 3: Regulatory Floodplain Mapping – Jock River

In December 2005, a letter of permission was issued by the RVCA to the original landowner for the construction of a berm to maintain flood risk mapping land levels as per (the 1980 Acres Floodplain) Mapping Study (96.0m) south of Ottawa Street. On March 3, 2009, the RVCA issued a letter of permission, included in ***Appendix B***, authorizing works to be conducted based on past approvals granted on the property. The authorized works involve removal of the existing berm and relocation to the approved 2005 location. The existing flap gate and culvert from the drainage easement are to be removed. The berm will also extend parallel along both sides of the drainage easement north up to Ottawa Street. The permission letter also includes the placement of fill between the new berm and Ottawa Street to a maximum level of 96.5**.**

3.2.2 Van Gaal Drain

The Rideau Valley Conservation Authority completed floodplain mapping for the Van Gaal Drain in November 2009.

Figure 4 was extracted from the Van Gaal and Arbuckle Drain Floodplain Mapping and illustrates a significant portion of the subject lands north of Perth Street within the Regulatory Flood Limit.

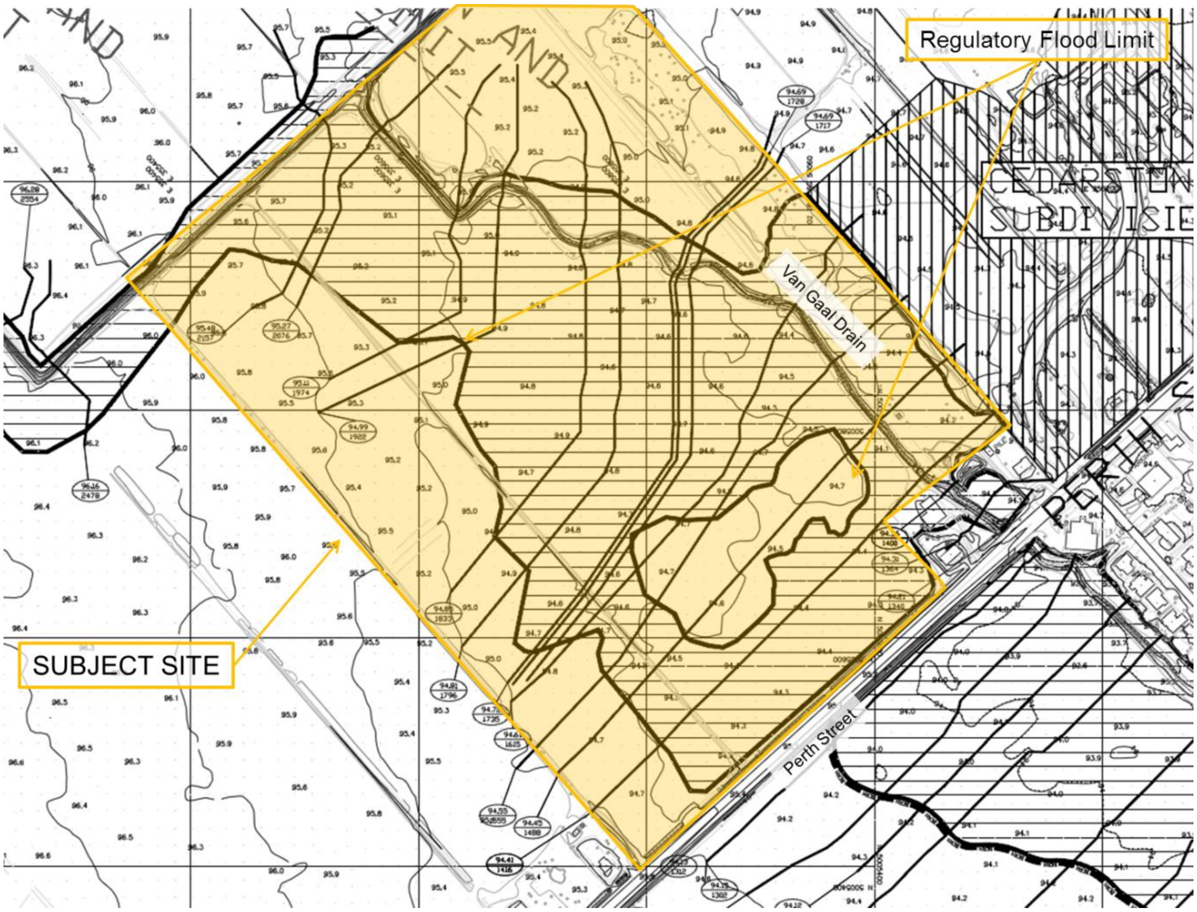

Figure 4: Regulatory Floodplain Mapping – Van Gaal Drain

The "*Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drain in the Village of Richmond (November 2009)"* was supported by RVCA staff and was brought forward for approval to the January 28, 2010 RVCA Executive Board Meeting. At this meeting, the Board approved the report and mapping as the regulatory floodplain mapping. The Board also approved the RVCA staff recommendation to allow for channel modifications to be undertaken north of Perth Street that would allow for an amendment to the regulatory floodplain limit. The approach and process are documented in the January 14, 2010 minutes of meeting which are contained in ***Appendix C*.** In summary, additional channel modifications will be completed north of Perth Street to increase the channel's conveyance capacity that meet the 1:100 year water surface profile in J.F. Sabourin & Associates Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains Report (November 2009). On approval and completion of the channel modifications, RVCA will amend its flood hazard and regulation limits mapping based on the completed works.

DSEL prepared a preliminary design for the proposed Van Gaal Drain modifications based on comments received during a pre-consultation meeting that took place on May 19, 2011 and a revised draft plan. The submission package is included in ***Appendix C***. RVCA reviewed the preliminary proposal and issued comments via email on October 16, 2012, included in ***Appendix C***. The RVCA comments are in the process of being addressed and a detailed submission is forthcoming.

3.3 Geomorphology

Richmond Village (South) Limited have retained Coldwater Consulting Ltd. (Coldwater) to review and confirm the previously submitted erosion threshold analysis. Coldwater had conducted field observations and sampling as well as desktop modelling to assess the condition as of the existing Van Gaal Drain / Arbuckle Drain, their completed study is included as ***Appendix D***.

The analysis concluded that erosion/deposition was presently occurring along the Van Gaal Drain south of Perth Street and that these sites will continue to erode in the post-development condition even without the proposed development's stormwater management plan. The erosion hazard model predicts that the proposed stormwater management plan will reduce erosion by an average of 21%.

Coldwater recommends rehabilitation of portions of the Van Gaal / Arbuckle drain, in select areas, to create a functional system to control flow energy and mitigate erosion in critical areas.

3.4 Existing Drainage Conditions

JFSA was retained to develop hydrologic and hydraulic models of the existing areas to assess the impact of the proposed urban development. The calculated flows included in ***Appendices E and I*** were prepared based upon the same methodology used for the 2009 Richmond Floodplain Mapping study, taking into consideration the timing of peak flows of the Van Gaal drain and Jock River. The calculated WSELs were determined using the 2009 Richmond Floodplain HEC-RAS spring and summer models, which do not include features including the berm that has been constructed upstream of Perth Street or any modification to Fortune Street culvert.

Table 2
Summary of Peak Flows to be conveyed through Subdivision

Tributary Area	100-year, 3-hour Chicago Storm	100-year, 12-hour SCS Type II Storm	100-year 10 Day Spring Snowmelt and Rainfall Event
	(m^3/s)	(m^3/s)	(m^3/s)
VG-2 & VG-5	2.55	3.19	1.86
VG-6	1.42	1.89	1.61
VG-7 & JR-1	1.60	2.01	1.30

Refer to JFSA memorandum in ***Appendix I*** and ***Drawing 1***, for locations of external areas to be conveyed through the proposed development.

Figure 5 illustrates the Existing Conditions Storm Drainage. The subject property is tributary to both the Van Gaal Drain and the Jock River, ultimately all stormwater from the subject area reaches the Jock River. As depicted, 1,147ha is tributary to the Van Gaal Drain, of which approximately 115ha is the subject site. Approximately 32ha of the subject development is tributary to the Jock River. Furthermore, approximately 34ha west of the subject site and south of Ottawa Street drains through the development site to the Jock River.

3.5 Hydrogeology

Richmond Village (South) Limited have retained Golder and Associates (Golder) to evaluate the proposed drainage plan with respect to the hydrogeological conditions encountered at the site. Golder has conducted and compiled field observations and sampling and have analysed the hydrogeological effects of subsurface drainage for the proposed development. ***Appendix F*** contains Golder's technical memorandum of their one year review of groundwater monitoring as well as their technical memorandum assessment of subsurface drainage and analysis of the 100 year flood event. ***Sections 5.2.4 and 6.3*** of this study discuss the results of their findings in detail.

4.0 SWM SERVICING ALTERNATIVES

In December 2008, a three-day design workshop in Richmond to develop a land use concept plan for their lands. Looney Ricks Kiss (LRK) facilitated the design workshop with input from the consultant team, City staff, residents and other stakeholders. Building upon the visioning principles established through the Village of Richmond Community Design Plan process, a preliminary land use concept plan was developed (***Figure 6***). In order to finalize and support the development land use concept plan, the stormwater management requirements need to be established.

Figure 6: Preliminary Land Use Concept Plan

4.1.1 End of Pipe Stormwater Alternatives

There are several suitable end-of-pipe options for the treatment of stormwater runoff from urban areas including – Infiltration Basins, Wetlands, Dry Ponds, Wet Ponds, and Hydrodynamic Separation Units. ***Table 3*** presents the four options and their suitability as described in the Stormwater Management Planning and Design Manual (MOE, March 2005).

Table 3
End of Pipe Treatment Systems Considered

Stormwater Management Practice	Description
Infiltration Basins	Infiltration basins are above-ground pond systems which are constructed in highly pervious soils. Water infiltrates into the basin and either recharges the groundwater system or is collected by an underground perforated pipe network and is discharged to a downstream outlet.
Wet Ponds	Wet ponds are the most common end-of-pie stormwater facilities in Ontario. The performance does not depend on soil characteristics, permanent pool minimizes re-suspension of captured solids and minimizes blockages at the outlet. Furthermore, the biological removal of pollutants occurs. Wet ponds are suited to drainage areas 5ha and greater
Wetlands	Wetlands are normally more land-intensive than wet ponds because of their shallower permanent pool depth. They provide similar quality benefits as wet ponds, although the biological processes are enhanced.
Dry Ponds	Dry ponds have no permanent pool of water. As such the removal of containments is purely a function of the detention time in the pond.
Hydrodynamic Separation Units	Hydrodynamic Separation Units or Oil / Grit separator are manufactured concrete units for the expressed purpose of trapping sediment and oil. The processes are patented and sizing is dependent on the manufactures specifications and tends to work well with small (less than 5.0ha) catchments. These units tend to occupy less land area.

In developing the various end of pipe stormwater management alternatives, two additional considerations were given priority. First, siting a SWMP at the lowest elevations of the site was considered over higher elevations. Second, the ponds should be situated nearest to their respective outlet locations.

Jock River Outlet Options

One of the SWMP options conceived was to locate a facility at the southernmost extent of the site to outlet directly to the Jock River within the subject lands. However, as illustrated on ***Drawing 1*** the 100-year flood elevation at this location is 96.40, which is higher than the northern portion of the development. An alternate outlet location was considered at the end of Ottawa Street where the 100-year level in the Jock River lowers dramatically, by approximately 2.0m, 1200m downstream at Ottawa Street which could facilitate drainage of the majority of the subject lands.

Outlet routes from the subject area to Jock River at Ottawa Street were considered along Queen Charlotte, Royal York Street, Burke Street, and Ottawa Street. Based on 'as-built' information received from the City of Ottawa Information Centre, it was found that an existing sanitary sewer would be in conflict with a proposed sewer routing along Royal York and Burke Streets at Fortune Street. An outlet route to the Jock River via Martin, Hamilton and Perth is not possible as the storm sewer is unable to cross under the existing Van Gaal Drain. As such, routing along Queen Charlotte Street and Ottawa Street are the storm sewer route most feasible from the subject lands should flows be directed to the Jock River at the end of Ottawa Street.

Van Gaal/Arbuckle Outlet Options

An outlet directly to the Van Gaal/Arbuckle Drain was considered for the development area north and south of Perth Street. It will be required to demonstrate that there would be no increase in flood levels along the Van Gaal Drain as well as providing erosion impulse control in accordance with the ***Geomorphology Study***.

Preliminary Screening of End of Pipe Stormwater Management Alternatives

Infiltration basins require low ground water tables and permeable soils. The ***Geotechnical Study*** illustrated that ground water elevations are within 0.60m to 1.2m of the existing ground surface. Furthermore, the soils are predominately clays with low percolation rates. Therefore, infiltration basins are not suitable in this application.

According to the ***SWMPDM*** wetlands tend to raise the temperature more than wet ponds. The ***Environmental Study*** indicated that the downstream watercourse has thermal sensitive species. Therefore, and end of pipe facility that minimizes temperature increase was given priority.

Based on the site characteristics, constraints, and requirements; stormwater management solutions incorporating wet ponds, dry ponds, and oil/grit separators will be investigated in additional detail.

4.1.2 Lot Level Stormwater Management Alternatives

Table 4 summarizes investigated lot level stormwater management practices.

Table 4
Lot Level Treatment Systems Considered

Stormwater Management Practice	Description
Rain Barrel	Harvesting rainwater by capturing rooftop runoff by connecting roof leaders to 'barrels' for watering during periods of dry weather.
Cistern	Harvesting rainwater by capturing rooftop runoff and directing stormwater to an underground storage tank. Water is pumped for watering during dry periods.
Green Roof	Consist of a thin layer of vegetation and growing medium installed on top of a convention flat or sloped roof. Reduces the 'heat' island effect and reduces runoff volume.
Roof downspout disconnection	Roof downspouts are disconnected from the weeping tile and are directed to grassed areas.
Soakaway, infiltration trench or chamber	Rectangular or circular excavations lined with geotextile filter cloth and filled with clear stone designed to promote groundwater infiltration.
Bioretention / Biofilter	Consists of a filter bed consisting of a mixture of sand, soil, and organic material. Bioretention facilities are designed to capture small storm events to retain and filter stormwater runoff. Plantings promote evapotranspiration.
Permeable pavement	An alternative to traditional impervious pavement to allow stormwater to drain through into an aggregate reservoir and infiltrate into the ground water.
Enhanced grass swale	Vegetated open channels designed to convey, treat, and attenuate stormwater runoff. Check dams and vegetation in the swale promote attenuation and infiltration.
Dry Swale	A dry swale incorporates an engineered soil medium and a perforated pipe under drain.
Perforated pipe system	Underground stormwater conveyance systems usually incorporated into the right-of-way drainage system.

Residential subdivisions typically consist of urban right-of-way cross-sections and residential homes with peaked roofs.

As such the following measures were not considered:

- ➢ Cisterns – Would increase the cost of each home.
- ➢ Green Roofs – Not standard practice in residential homes.

- Biorentention / Biofilter – not part of a standard City of Ottawa cross-section. Would increase right of way maintenance.
- Permeable Pavement – not standard practice in the City of Ottawa.
- Enhanced Grass Swale – Would have to take place in rear yards. It is anticipated that home owners will remove check dams.
- Perforated pipe system – not typical sewer design practice in the City of Ottawa. Would increase maintenance costs.

The proposed subdivision will consist of the following:

- Roof Leaders to Grassed Areas.
- Dry Swales.
- An education program to promote rain barrels.

4.1.3 Screening of Options

Three stormwater management servicing alternatives, illustrated on ***Figures 7, 8, 9*** were developed for evaluation:

Option 1 (Figure 7)

- Four Stormwater Management Ponds (SWMPs)
- Three facility are “Wet Ponds” with MOE ‘Enhanced’ TSS removal. While Pond 3 is a dry pond for quantity control and a hydrodynamic separator to provide quality control.
- External drainage tributary to the subject lands will be conveyed through proposed storm sewers and drainage ditches. The existing channels identified as the Moore Tributary and existing outlet to the Jock River will be enclosed.
- External drainage west of the development currently being conveyed along the Perth Street roadside ditches will continue to outlet to the Van Gaal Drain. Once the road is widened to an urban cross-section the external drainage from west of Perth Street will be conveyed to the Van Gaal Drain via storm sewers.
- External drainage currently being conveyed through the Moore Tributary will be conveyed to Pond 1 via new storm sewers.
- Jock River Estates drainage will be conveyed north of Ottawa Street through the new storm sewer system to Pond 1.
- Pond 1 will be designed to receive flow from the majority of the area between Perth Street and Ottawa Street, approximately 58 ha.

- This wet pond will be designed to attenuate post-development runoff rates to predevelopment levels, while flows up to and including the 2-year event will be attenuated to 330L/s in accordance with the ***Geomorphic Study***.
- Pond 2 will be designed to receive runoff from approximately 8 ha north of Ottawa and the developable land south of Ottawa Street.
- Pond 2 is a wet pond and has one outlet directing post-development runoff rates to the Jock River via a proposed storm sewer along Ottawa Street. The 100-year release rate from Pond 2 will be restricted to the free flowing capacity of the outlet sewer.
- Pond 3 will be designed as a dry pond with a hydrodynamic separator to collect and retain runoff from approximately 3 ha north of Perth Street east of the Van Gaal Drain.
- This pond will be designed to attenuate flows to 330L/s in accordance with the ***Geomorphic Study***.
- The proposed facility will incorporate a hydrodynamic separator to provide 80% TSS removal per ***SWMPDM***.
- Pond 4 will be designed to collect and retain runoff from approximately 28 ha north of Perth Street east of the Van Gaal Drain and will outlet to the Van Gaal Drain.
- This pond will be designed to attenuate post-development runoff rates to predevelopment levels, while flows up to and including the 2-year event will be attenuated to 330L/s in accordance with the ***Geomorphic Study***.

Option 2 (Figure 8)

- Three Stormwater Management Ponds (SWMPs)
- Ponds 1 and 2 are “Wet Ponds” with MOE ‘Enhanced’ TSS removal. Pond 3 is a dry pond for quantity control and a hydrodynamic separator to provide quality control.
- External drainage tributary to the subject lands will be conveyed through proposed storm sewers and therefore the existing tributaries identified as Moore and Jock River Estates will be enclosed. A portion of the Moore tributary along Queen Charlotte will remain open.
- External drainage west of the development currently being conveyed along the Perth Street roadside ditches will continue to outlet to the Van Gaal Drain. Once the road is widened to an urban cross-section the external drainage from west of Perth Street will be conveyed to the Van Gaal Drain via storm sewers.
- External drainage currently being conveyed through the Moore Tributary will be conveyed to Pond 1 via new storm sewers.

- Jock River Estates drainage will be conveyed north of Ottawa Street through the new storm sewer system to Pond 1.
- Pond 1 will be designed to receive runoff from 45 ha between Ottawa and Perth Streets in addition to 28 ha north of Perth Street on the west side of the Van Gaal Drain. Pond 1 will have two outlets:
- The first channel will be designed to convey low flows up to and including the 2-year event attenuated to 330L/s in accordance with the ***Geomorphic Study***. The channel will provide both surface and subsurface conveyance. The channel will be bordered by strategic planting to promote shaded cover, while the subsurface component will enhance cooling opportunities.
- The second channel will be designed to convey the treated stormwater runoff from the less frequent storm events generated during the 5 to 100 year return periods.
- Pond 2 will be designed to receive runoff from 21 ha north of Ottawa and the developable land south of Ottawa Street.
- Pond 2 has one outlet directing post-development runoff rates to the Jock River via a proposed storm sewer along Ottawa Street. The 100-year release rate from pond 2 will be restricted to the free flowing capacity of the outlet sewer.
- Pond 3 will be designed as a dry pond for quantity control / hydrodynamic separator to collect and retain runoff from approximately 3 ha north of Perth Street east of the Van Gaal Drain.
- This pond will be designed to attenuate flows to 330L/s in accordance with the ***Geomorphic Study***.
- The pond outlet structure will be designed to mitigate increases in water levels in the Van Gaal Drain
- The proposed facility will incorporate an oil / grit sedimentation chamber to provide 80% TSS removal per ***SWMPDM***.

Option 3 (Figure 9)

- Three Stormwater Management Ponds (SWMPs)
- Ponds 1 and 2 are “Wet Ponds” with MOE ‘Enhanced’ TSS removal, while Pond 3 is conceived to be a dry pond for quantity control and a hydrodynamic separator to provide quality control.
- In this stormwater management option, the Moore Tributary for its entire length will be left open and Jock River Estates drain will be enclosed within the development area. The existing channel will need to be redesigned to ensure that the channel contains the 100yr event. JFSA prepared a hydrologic and hydraulic model to confirm the proposed cross-sections, see ***Appendix I*** for the detailed analysis.

- External drainage west of the development currently being conveyed along the Perth Street roadside ditches will continue to outlet to the Van Gaal Drain. Once the road is widened to an urban cross-section, the external drainage from west of Perth Street will be conveyed to Pond 1 via storm sewers.
- External drainage currently being conveyed through the Moore Tributary will be conveyed to the Van Gaal/Arbuckle Drain via the redesigned Moore Tributary channel.
- Jock River Estates drainage will be conveyed along Ottawa Street through a new to the new SWM Wetland 2 facility. The envisioned sequencing of events that has been vetted through RVCA is to: (1) construct SWM Pond 2 and trunk sewer along Ottawa Street; (2) close the existing drain outlet south of Ottawa Street in coordination with the RVCA.
- The Fortune Street Culvert will be modified to lower 100-year summer water levels upstream of Fortune Street.
- <u>Pond 1</u> will be designed to receive runoff from 45 ha between Ottawa and Perth Streets in addition to 28 ha north of Perth Street on the west side of the Van Gaal Drain. Pond 1 will have two outlets:
- The first outlet will be designed to convey low flows up to and including the 2-year event attenuated to 330L/s in accordance with the ***Geomorphic Study***. The pond will be designed to enhance cooling opportunities where possible.
- The second outlet will be designed to convey the treated stormwater runoff from the less frequent storm events generated during the 5 to 100 year return periods.
- Pond 1 is situated in the 100-year regulatory floodplain, outside the 100-year erosion limit and 100-year summer flood elevation of the Van Gaal/Arbuckle Drain.
- <u>Pond 2</u> will be designed to receive runoff from 21 ha north of Ottawa and the developable land south of Ottawa Street.
- Pond 2 has one outlet directing post-development runoff rates to the Jock River via a proposed storm sewer along Ottawa Street. The 100-year release rate from pond 2 will be restricted to the free flowing capacity of the outlet sewer.
- <u>Pond 3</u> will be designed to collect and retain runoff from approximately 3 ha north of Perth Street east of the Van Gaal Drain.
- This pond will be designed to attenuate flows to 330L/s in accordance with the ***Geomorphic Study***.
- The proposed facility will incorporate an oil / grit sedimentation chamber to provide 80% TSS removal per the ***SWMPDM***.

4.2 Selection of Preferred SWMP

4.2.1 Evaluation Process

The three stormwater management options presented in ***Section 4.1*** were brought forward for evaluation through a pair-wise comparison matrix. The evaluation matrix was developed as part of the Village of Richmond Master Servicing Study (***Servicing Study***). These evaluation criteria were presented and reviewed by the Technical Advisory Committee and the public.

The evaluation criteria consist of criterion in four major categories: Natural Environment, Caring and Healthy Communities, Constructability and Functionality, and Cost. Each of these major categories has been assigned a weighting which is summarized in ***Table 5***.

Table 5
Summary of Decision Matrix Categories

Parameter	Indicators	Weighting
Natural Environment		**21%**
N1 Impact on significant natural features	Loss, displacement, disruption fragmentation of natural areas (wetlands, woodlands, terrestrial ecology, ANSI's and associated corridors).	3%
N2 Impact on ecological processes	Fragmentation of natural areas, interruption of natural linkages.	3%
N3 Impact on aquatic systems	Number of stream crossings, impact on significant fish habitat	7%
N6 Effects on green space and open space	Interference with linear green way systems.	7%
Caring and Healthy Communities		**25%**
C3 Impact on level of service	Maintains or improves level of service to the existing and future village residents.	13%
C4 Disruption to community	Compatibility with existing community character.	6%
C9 Consistency with infrastructure planning policies	Compatibility with infrastructure servicing corridors and flexibility for enhancements to land use.	6%
Constructability and Functionality		**29%**
CO2 Schedule / Staging Opportunities	Ability to phase infrastructure to facilitate development phasing	6%
CO3 Construction Risk	Conforms to geotechnical, geomorphology, hydrological	6%
CO4 Impact on existing infrastructure	Relocation of existing services (i.e. sanitary sewers, wells) and other utilities	6%
CO5 Disruption during construction	Location of new infrastructure in built up areas and nuisance effects	6%
CO6 Operation and maintenance	Proven track record, ease of operating and maintenance	6%
Cost		**25%**
E9 Total 25 year life cycle cost	Cost effective life cycle costs	6%
E11 Total Capital Cost	Cost effective capital costs.	19%

Each alternative is ranked based on the criteria presented previously. The ranking values assigned to the alternatives based on the various criteria are given over a relative range from 1 to 5. The description of these rankings is presented in **Table 6**:

Table 6
Decision Matrix Categories Ranking System

Ranking	Description
5 - Positive or No Impact	The alternative meets all applicable requirements, provides tangible benefits
4 – Minor Impact	The alternative has some minor negative impacts or dis-benefits that may easily be mitigated or compensated for
3- Moderate Impact	The alternative has noticeable negative impacts, however, the severity of the impacts may be reduced or compensated for
2 – Noticeable Negative Impact	The alternative has significant negative impacts which may be mitigated, although these may be costly, time consuming or result in other negative impacts
1 - Negative or Significant Impact	The alternative does not meet applicable requirements, results in significant dis-benefits and/or negative impacts cannot be mitigated

Under this ranking system, each individual criteria is ranked relatively for each alternative. For example, for Criteria N1 (Impact on Natural Features), the 1 to 5 ranking for an individual alternative is determined based on the relative impact on the environment compared to all the other alternatives being evaluated.

4.2.2 Discussion of Preferred Option

The results of the evaluation of the three stormwater management options are contained in ***Table 7***. Kilgour & Associates were retained to complete the evaluation of the three options for the Natural Environment criteria.

Table 7
Stormwater Management Evaluation

Parameter		**Option 1**			**Option 2**			**Option 3**		
	Weighting	**Description**	**Score**	**Weighting**	**Description**	**Score**	**Weighting**	**Description**	**Score**	**Weighting**
Natural Environment	**21%**									
N1 – Impact on Significant Natural Features	3%	There is no footprint of the SWM facilities on significant natural features (NESS Area 422, Significant Woodlands, Jock River corridor).	5	0.15	There is no footprint of the SWM facilities on significant natural features (NESS Area 422, Significant Woodlands, Jock River corridor).	5	0.15	There is no footprint of the SWM facilities on significant natural features (NESS Area 422, Significant Woodlands, Jock River corridor).	5	0.15
N2 – Impact on Ecological Processes	3%	There are no terrestrial corridors impacted by this option. New outlets being introduced into watercourses.	4	0.12	There are no terrestrial corridors impacted by this option. New outlets being introduced into watercourses.	4	0.12	There are no terrestrial corridors impacted by this option. New outlets being introduced into watercourses. Riparian corridors being enhanced.	5	0.15
N3 – Impact on Aquatic Systems	7%	Loss of some 3660 m^2 of indirect fish habitat; loss of some 2510 m^2 of direct fish habitat	2	0.14	Loss of some 3285 m^2 of indirect fish habitat; loss of some 177 m^2 of direct fish habitat	2	0.14	Conversion of existing indirect fish habitat to direct fish habitat, for a total gain of direct fish habitat of some 3386 m^2. Creation of new potential fish spawning habitats in outlet channel.	5	0.35
N6 – Effects on Greenspace and Open Space	7%	Less greenspace as hedgerows removed and entire lengh of Moore Tributary enclosed.	2	0.14	Less greenspace as portion of Moore Tributary (VG-R2-2) enclosed and hedgerows removed. VG-R3-1 remains open retaining existing vegetation.	3	0.21	Entire length of Moore Tributary open and hedgerow reestablished along VG-R3-2. Enhancement of tributary and SWM Pond integrared with Martin Street Pedestrian Extension	4	0.28

Caring and Healthy Communities	**25%**									
C3 – Impact on Level of Service	13%	New facilities. All options meet swm criteria and flood protection for downstream recipents.	4	0.52	New facilities. All options meet swm criteria and flood protection for downstream recipents.	4	0.14	New facilities. All options meet swm criteria and flood protection for downstream recipents.	4	0.52
C4 – Disruption to Community	6%	Village currently does not have wet pond SWM facilities. Ponds situated in development lands.	3	0.18	Village currently does not have wet pond SWM facilities. Ponds situated in development lands.	3	0.21	Village currently does not have wet pond SWM facilities. Ponds situated in development lands.	3	0.18
C9 – Consistency with Infrastructure Planning Policies	6%	Pond technology and design consistent with the City and Ministry Guidelines. Grading north of Perth Street exceeds Geotechnical recommendations.	2	0.12	Pond technology and design consistent with the City and Ministry Guidelines.	4	0.24	Pond technology and design consistent with the City and Ministry Guidelines. Location of ponds in floodplain not common but permitted under the Provincial Policy Statement based on certain criteria being met	3	0.18

Constructability and Functionality	**29%**									
CO2 – Schedule /Staging Opportunities	6%	Four ponds equally distributed to allow for ease in project phasing	4	0.24	One large centrally located facility does not provide the ease of construction phasing	3	0.18	One large centrally located facility does not provide the ease of construction phasing	3	0.18
CO3 – Construction Risk	6%	Exceeds geotechnical recommendations.	1	0.06	Exceeds geotechnical recommendations.	1	0.06	Conforms to geotechnical recommendations	5	0.30
CO4 – Impact on Existing Utilities	6%	Stormsewer outfall along Queen Charlotte and Ottawa street	2	0.12	Stormsewer outfall along Ottawa Street	3	0.18	Stormsewer outfall along Ottawa Street	3	0.18
CO5 – Disruption during Construction	6%	4 Ponds – additional pond construction over other options	2	0.12	3 Ponds with Pond 1 setback farther from existing residents	3	0.18	3 Ponds with Pond 1 situated closer to existing residents	2	0.12
CO6 – Operation and Maintenance	6%	Use of proven technology – additional pond to maintain	3	0.18	Use of proven technology	4	0.24	Use of proven technology	4	0.24
Economy	**25%**									
E9 – Total 25 year Life Cycle Costs	8%	Highest O&M costs.	2	0.12	Lower O&M than option 3	4	0.24	Lower O&M than option 1	3	0.18
E11 – Total Capital Costs	13%	Highest total capital cost	1	0.19	Lower capital cost than 1.	3	0.57	Lower capital cost than 2.	5	0.95
Total				**2.4**			**3.2**			**4.0**
Ranking				**3**			**2**			**1**

Based on the above analysis recommended alternative was found to be Option 3, where it scored ***4.0***.

For Natural Environment Criteria, all three options received the same rating for N1 – Impact on Significant Natural Features as all ponds have been situated outside of significant woodlands and the Jock River Corridor. For the remaining criterion, the Fish Habitat Risk Assessment was relied on to assess each option. This assessment identified re-grading of the Mattamy land holdings, and the subsequent construction and operation of the SWM ponds will cause some moderate changes under Options 1 and 2, and a net gain in direct fish habitat in Option 3. Fish habitats that would be altered are generally indirect intermittent habitats or are man-made. SWM Option 3 is anticipated to provide a significant and net benefit to direct fish habitat, in association with the following aspects of the proposed design. Sections 6 and 7 of the Moore Branch will be re-graded to enhance the conveyance function of the feature. That will result in a change in the status of Sections 7 and 8, which are currently classified as indirect intermittent fish habitat, to direct intermittent fish habitat. Fish will continue to be able to access Section 7 for spawning, while the improved grading is anticipated to allow larvae/fry to migrate out of the system as water levels recede over the course of the spring/summer. A French drain will be incorporated in the SWM pond design to provide cool base flow to the lower Moore Branch, and maintain the cool-water function of that feature. The outlet channel for SWM Pond 1 will be designed to provide spring fish spawning habitat. Additional riparian plantings along the Moore Branch will enhance its ability to cool surface waters and to provide a naturalized corridor. Riparian plantings along the main stem of the Arbuckle Drain will provide additional shade and course woody material to that feature. Option 3, with a large SWM pond in the 100-year floodplain of the Arbuckle Drain, would provide net benefits to fish habitat with up to an additional 3,386 m^2 of fish habitat created as a result of the undertaking. As such, Option 3 ranked highest for criterion N2, N3, N6. *(Kilgour and Associates)*

For Caring and Healthy Communities criteria, the three options have similar ranking for the three criteria except for C9 – Consistency with Infrastructure Planning Policies. All options will meet the swm criteria established for the receiving watercourses. C4 – Disruption to Community is defined as compatibility with existing Village character. The Village does not have stormwater management so this is new infrastructure being introduced, however, all ponds are contained on future development lands. Option 2 ranked highest for C9 as it is most consistent with applicable policies.

For Constructability and Functionality, Option 1 ranks the highest for CO2 – Schedule/Staging opportunities as the additional pond provides greater staging flexibility. CO3 – Construction Risk was based meeting JWL grade raise limits. Option 3 ranked highest as it conforms to the grade raise limits with Option 1 and 2 exceeding geotechnical recommendations. CO4 – Impact on Existing Utilities was defined as any new storm sewers required within existing right-of-ways. Option 1 ranked the lowest as it proposes a storm sewer along Queen Charlotte Street. Option 2 ranked highest for CO5 – Disruption during Construction as it has 3 ponds with Pond 1 situated farther away from existing residents. All options were equal for CO6 – Operations and Maintenance as the technology is consistent among the options and has a proven track record.

Economy represents 25% of the score. Based on a relative comparison of costs, Option 2 ranks highest for E9, Total 25 year Life Cycle Costs as it had the lowest life cycle costs. For E11, Total Capital Costs, Option 3 had the lowest costs and therefore ranked first.

Figure 10 illustrates the post development drainage tributary to the Van Gaal Drain and the Jock River. As described previously the subject property is tributary to both the Van Gaal Drain and the Jock River, ultimately all stormwater from the subject area reaches the Jock River. The proposed stormwater management scheme will reduce the total area tributary to the Van Gaal Drain by approximately 67ha. The external areas VG-7, JR-1will be tributary to Pond 2, which outlets to the Jock River. Areas tributary to the Jock River will remain so in the post development scenario.

5.0 EVALUATION STORM CONVEYANCE SYSTEMS

Modern storm conveyance systems were considered in light of ***City Standards,*** the Technical Memorandum Technical Bulletin ISD-2012-1, as well as the proposed subdivision plan and streetscaping plan presented in the ***CDP***.

The City of Ottawa provided feedback on the evaluation matrix utilized to assess the preferred stormwater management scheme presented in the ***March 2010 SWM Report***. ***Table 8*** summarizes the evaluation criterion employed to select the preferred conveyance and servicing system.

Table 8
Summary of Decision Matrix Categories

Parameter	Indicators	Weighting
Caring and Healthy Communities		**30%**
C3 Impact on level of service	Maintains or improves level of service to the existing and future village residents.	15%
C9 Consistency with infrastructure planning policies	Compatibility with infrastructure servicing corridors and flexibility for enhancements to land use.	15%
Constructability and Functionality		**30%**
CO2 Schedule / Staging Opportunities	Ability to phase infrastructure to facilitate development phasing	6%
CO3 Construction Risk	Conforms to geotechnical, geomorphology, hydrological, etc.	9%
CO5 Disruption during construction	Location of new infrastructure in built up areas and nuisance effects	6%
CO6 Operation and maintenance	Proven track record, ease of operating and maintenance	9%
Cost		**40%**
E9 Annual Cost	Estimated annual maintenance cost	15%
E11 Total Capital Cost	Estimated capital costs.	25%

Each alternative is ranked based on the criteria presented in ***Table 9***. Under this ranking system, each individual criterion was ranked relatively for each alternative. For example, the 1 to 5 ranking for an individual alternative is determined based on the relative impact compared to all the other alternatives being evaluated. In regards to cost, the least costly will be automatically assigned a 5, while the most will be 1, with the remaining option prorated between 1 and 5 according to their estimated costs.

Capital cost for each scenario considered cut / fill requirements, trunk routing, and local storm services. While life cycle cost were considered to be the anticipated maintenance and operation of the sewer collection system, excluding the stormwater management facilities. The estimated costs are based on past data and are presented for comparison of alternatives only and are not for budgetary purposes.

Table 9
Decision Matrix Categories Ranking System

Ranking	**Description**
5 - Positive or No Impact	The alternative meets all applicable requirements, provides tangible benefits
4 – Minor Impact	The alternative has some minor negative impacts or dis-benefits that may easily be mitigated or compensated for
3- Moderate Impact	The alternative has noticeable negative impacts, however, the severity of the impacts may be reduced or compensated for
2 – Noticeable Negative Impact	The alternative has significant negative impacts which may be mitigated, although these may be costly, time consuming or result in other negative impacts
1 - Negative or Significant Impact	The alternative does not meet applicable requirements, results in significant dis-benefits and/or negative impacts cannot be mitigated

5.1 Summary of Alternatives Assessed

The conveyance systems reviewed contain minor and major conveyance components. Minor components were those with a 5-year carrying capacity, while the major system designed to convey runoff in excess of the minor capacity. The following summarizes the conveyance systems and storm service arrangements evaluated for use in the subject area.

- Foundation service to street storm sewer – Gravity connection
- Foundation service to dedicated foundation collector – Gravity connection
- No Foundation Service – Slab on grade units
- Foundation service to street storm sewer – Sump pump

5.2 Description of Alternatives

5.2.1 Foundation service to street storm sewer

A foundation service that outlets to a street storm sewer with a gravity connection is typically applied throughout the City of Ottawa in greenfield developments. In this servicing arrangement homes are established with underside of footings set 0.30m above the modeled 100-year hydraulic grade line in the sewer. ***Figure 11*** illustrates the servicing arrangement. ***Figure 11*** illustrates the estimated sewer size and associated HGL determined for the recommended solution at MH ID 402.

The majority of the existing Village of Richmond is reliant on sump pumps for foundation drainage. A gravity serviced home in the Village of Richmond would raise the level of service in the area as the home owner would not be reliant on maintenance of privately owned sump pumps.

Construction phasing and staging is contingent upon the completion of the receiving stormwater management facility.

As illustrated this servicing arrangement results in an approximate grade raise of 1.6m above existing ground, and therefore will result in areas exceeding grade raise restriction. In order to mitigate the settlement, surcharging is commonly employed. Any surcharging requirements necessary to construct homes would have potential negative impact on construction phasing. Most notably the time spent on waiting for settlement objectives to be reached as well as the transportation of surcharge material.

It was estimated that the site required a total net fill of ***1,906,500m^3***. See ***Figure 12*** for an overview of estimated cut / fill.

The estimated capital cost for the construction of storm sewers and earthworks was ***$74,987,000***, while the 25 year life cycle costs were estimated to be ***$99,000***. See ***Appendix G*** for detailed cost breakdown.

5.2.2 Foundation service to dedicated foundation collector

Foundation services connected to a dedicated foundation collector sewer are not typically employed in Greenfield subdivision developments in the City of Ottawa. However, these systems are used in site plan developments where the parking lot drainage is separated from foundation drainage. The advantage being that the foundation drainage is hydraulically separated the system collecting street and parking lot drainage. ***Figure 13*** illustrates the estimated sewer size and associated HGL determined for the recommended solution at MH ID 402.

Several foundation drain routing options were investigated. Due to the topography of the area, a gravity outlet is not available to the site that would be cost effective from either a fill or sewer routing perspective. Therefore, it was conceived that this servicing arrangement would utilize a lift station to convey foundation drainage collected into the Van Gaal Drain. This would enable the site grading to take place at a much lower elevation. As depicted in ***Figure 13***, the site would be approximately ***1.1m*** lower than ***Option 4.2.1***.

As described in ***Section 5.2.1***, the majority of the Village of Richmond is reliant on sump pumps for foundation drainage. This alternative would increase the level of service to the home owner as the home owner would not be reliant on maintenance of privately owned sump pumps. In this scenario, the liability would be on the municipality to ensure continuous operation of the pump station during periods where foundation drainage was collected.

Construction phasing and staging is contingent upon the completion of the receiving stormwater management facility and foundation drainage lift station.

This servicing arrangement respects the grade raise restriction throughout the development area. It was estimated that the site required a total net fill of ***619,858m^3***. See ***Figure 14*** for an overview of estimated cut / fill.

The estimated capital cost for the construction of storm sewers and earthworks was ***$30,219,000***, while the 25 year life cycle costs were estimated to be ***$288,000***. See ***Appendix G*** for detailed cost breakdown.

5.2.3 No Foundation Service – Slab on grade units

Slab on grade units, i.e. no basements or limited crawl spaces only, do not require foundation drainage. Foundation walls have equal amount of hydrostatic pressure on both sides and there would be no living space in the subsurface that would require a storm service. Slab on grade units have never been constructed for an entire community within the City of Ottawa.

As depicted in ***Figure 15***, the street storm sewer would be placed a minimum elevation below the finished grade.

This servicing arrangement respects the grade raise restriction throughout the development area.

It was estimated that the site required a total net fill of ***967,439m^3***. See ***Figure 16*** for an overview of estimated cut / fill.

The estimated capital cost for the construction of storm sewers and earthworks was ***$29,529,000***, while the 25 year life cycle costs were estimated to be ***$99,000***. See ***Appendix G*** for detailed cost breakdown.

5.2.4 Foundation service to street storm sewer – Sump pump

Foundations equipped with sump pumps that outlet to storm sewers are not typically employed in Greenfield subdivision developments in the City of Ottawa. In this servicing arrangement homes are situated with underside of footings 0.15m above the invert of the receiving sewer. The sump pump will be equipped with a swan neck that is 0.30m above the modeled 100-year hydraulic grade line. ***Figures 17 and 18***, included in ***Figures*** illustrate the servicing arrangement. ***Figure 17*** illustrates the estimated sewer size and associated HGL determined for the recommended solution at MH ID 402

As described in ***Section 5.2.1***, the majority of the Village of Richmond is reliant on sump pumps for foundation drainage. This alternative remains consistent with the existing level of service within the Village of Richmond.

Construction phasing and staging is contingent upon the completion of the receiving stormwater management facility.

As depicted in ***Figure 17***, the street storm sewer would be placed a minimum elevation below the finished grade. This servicing arrangement respects the grade raise restriction throughout the development area.

Golder Associates were retained to assess the hydrogeological effects of subsurface drainage for the subject lands to determine the suitability of employing sump pump as a means to provide foundation drainage. Their technical memorandum is included in ***Appendix F***. Their analysis concluded that the majority of the development would have USFs established above the long-term (steady-state) groundwater elevations. During a 100-year storm event and spring freshet it is anticipated that standard, commercially available sump pumps will function as intended. An extra level of protection with backup sump pumps with reserve power is also proposed alongside of the typical sump pump arrangement. With a predicted inflow rate of up to 2.0m3/day as summarized in the memorandum this would equate to up to 21 days of backup pumping (see DSEL memorandum also found in ***Appendix F***).

It was estimated that the site required a total net fill of ***671,257m***^{3}. See ***Figure 19*** for an overview of estimated cut / fill.

The estimated capital cost for the construction of storm sewers and earthworks was ***$26,586,000*** while the 25 year life cycle costs were estimated to be ***$99,000***. See ***Appendix G*** for detailed cost breakdown.

5.3 Comparison of Alternatives

Table 10
Comparison of Alternatives

Parameter	**Option 4.2.1. Gravity System**	**Option 4.2.2. Foundation Collector**	**Option 4.2.3. Slab on Grade Units**	**Option 4.2.4 Sump Pumps**
Caring and Healthy Communities				
C3 Impact on level of service	Improves level of service when compared to Village of Richmond at large	Improved level of service to individual home owner	Reduced level of service. No basements provided.	Consistent with Village of Richmond level of service.
C9 Consistency with infrastructure planning policies	Consistent with City of Ottawa Greenfield development. Potential negative impact on land use due to fill requirements.	A departure from typical greenfield development.	Limits land uses by not providing foundation drainage services.	A departure from typical greenfield development outside of the Village of Richmond. However, common with Village limits.
Constructability and Functionality				
CO2 Schedule / Staging Opportunities	Phasing potentially inhibited by fill requirements, obtaining fill and potential surcharging requirements in areas exceed grade raise parameters.	Increased capital cost on the outset of the project for lift station. Third pipe will increase construction time.	No impact to construction phasing.	No impact to construction phasing.
CO3 Construction Risk	Exceed geotechnical parameters.	Meets geotechnical parameters.	Meets geotechnical parameters.	Meets geotechnical parameters.
CO5 Disruption during construction	Significantly more imported material to site for filling. Increased construction traffic	Increased construction duration for third pipe and lift station construction.	No additional construction disturbance expected.	No additional construction disturbance expected.
CO6 Operation and maintenance	Sewer arrangement a proven track record, with normal operating and maintenance expected. Anticipate increased road work maintain for potential settlement issues.	Increased operation and maintenance for additional pipe and lift station. A lift station failure would impact entire development.	Sewer arrangement a proven track record, with normal operating and maintenance expected.	Sewer arrangement would require normal operating and maintenance. Sump pump maintenance would be required by home owner.
Cost				
E9 Est. Maintenance cost	$99,000	$288,000	$9(,000	$9(,000
E11 Est. Capital Cost	$74,987,000	$30,219,000	$29,529,000	$26,586,000

5.4 Ranking of Alternatives

Table 11
Ranking of Alternatives

Parameter	**Weighting**	**Option 5.2.1**		**Option 5.2.2**		**Option 5.2.3**		**Option 5.2.4**	
		Score	**Rank**	**Score**	**Rank**	**Score**	**Rank**	**Score**	**Rank**
Caring and Healthy Communities									
C3 Impact on level of service	15%	5	0.75	4	0.60	1	0.15	2	0.30
C9 Consistency with infrastructure planning policies	15%	5	0.75	3	0.45	2	0.30	3	0.45
Constructability and Functionality									
CO2 Schedule / Staging Opportunities	6%	1	0.06	2	0.12	5	0.30	5	0.30
CO3 Construction Risk	9%	1	0.09	5	0.45	5	0.45	5	0.45
CO5 Disruption during construction	6%	1	0.06	2	0.12	5	0.30	5	0.30
CO6 Operation and maintenance	9%	4	0.36	1	0.09	5	0.45	2	0.18
Cost									
E9 Est. Maintenance cost	15%	5	0.75	1	0.15	5	0.75	5	0.75
E11 Est. Capital Cost	25%	1	0.25	4.6	0.51	4.7	0.98	5	1.25
Total	**100%**		**3.07**		**3.14**		**3.87**		**3.98**

5.5 Conveyance System Conclusion

Each alternative was described in detail under ***Section 5.2.*** and was compared side by side in tabular format in ***Section 5.3***. ***Section 5.4*** presented the weighting and ranking that was applied to each based on the discussion presented in the previous subsections. Option 5.2.4 emerged with the highest ranking.

Option 5.2.1, was ranked lowest. The capital cost for developing this option is cost prohibitive. Not only that a substantial amount of fill is required, a majority of the subdivision will have footings above the existing ground. Therefore, structural fill will be required to support the footings. Furthermore, due to exceeding grade raise recommendations, the significant amount of fill required and associated concerns, this option was not selected.

Option 5.2.2 scored poorly in sections C06, E9 and E11. Operating and maintaining a third pipe as well as a lift station set this alternative back.

Option 5.2.3, scored strongly in construction and functionality as well as capital cost. However, it scored slight lower in the C3 and C9 categories than the preferred solution.

Sections ***6.0 - Stormwater Conveyance*** and ***7.0 - Stormwater Management Facilities***, provide master servicing level detailed information designed on the basis of Option 5.2.4. – Foundation Service to street storm sewer with homes equipped with sump pumps.

6.0 STORMWATER CONVEYANCE

As presented in Section 5.0, the recommended stormwater servicing solution consist of a major system, a minor system, and homes with basements will be equipped with sump pump to provide foundation drainage. ***Figures 18 and 19***, included in ***Figures*** illustrate the servicing arrangement.

The following sub-sections provide additional analysis of the recommended stormwater servicing solution.

6.1 Grade Control Plan - Major System

The proposed master grading plan is depicted on ***Drawing 2***. ***Drawing 2*** illustrates centerline of road grades, which were established on three criteria:

- Minimum depth to pipe invert = 2.1m
- Minimum depth to pipe obvert = 1.5m
- Minimum slope of saw tooth road pattern = 0.15% from high point to high point.

Where major system flow is shown to cross Perth Street and Ottawa street, the minor system was designed to covey flow under these streets. Additional information regarding conveyance through the minor system is contained in ***Section 6.2***.

6.2 Minor System

Drawing 3 illustrates the proposed minor system. ***Table 124*** summarizes the minimum parameters utilized to size the stormwater conveyance system.

Table 12
Storm Sewer Design Criteria

Design Parameter	Value
Intensity Duration Frequency Curve (IDF) 5-year storm event. A = 998.071 B = 6.053 C = 0.814	$i = \frac{A}{(t_c + B)^C}$
Minimum Time of Concentration	10 minutes
Rational Method	$Q = CiA$
Storm sewers are to be sized employing the Manning's Equation	$Q = \frac{1}{n} AR^{2/3} S^{1/2}$
Minimum Sewer Size	250mm diameter
Minimum Manning's 'n'	0.013
Service Lateral Size	100mm dia PVC SDR 28 with a minimum slope of 1.0%. Homes to be equipped with sump pump flow.

Minimum Depth of Cover	2.0m from crown of sewer to grade
Minimum Full Flowing Velocity	0.8m/s
Maximum Full Flowing Velocity	3.0m/s
Additional Considerations	Storm sewer maintenance holes serving sewers 900mm diameter and less shall be constructed with 300mm deep sumps. Maintenance holes for storm sewers greater than 900mm must be benched.
Extracted from Sections 5 and 6 of the City of Ottawa Sewer Design Guidelines, October 2012.	

A Rational Method design sheet is contained with ***Appendix H***. The pipe sizes were confirmed through hydraulic modeling as described in ***Section 6.3***. There are currently sections of storm sewers that are shown as box culverts due to site constraints, however, the feasibility of multiple circular pipes in select locations will be evaluated at the detailed design stage.

The subject lands slope generally from west to east. As described in ***Section 3.4***, there are significant areas west of the subject land that currently drains through the development property. As illustrated on ***Drawing 3*** the external areas are summarized below:

- 63.1ha Perth Street road side ditch north;
- 34.4ha Perth Street road side ditch south;
- 94.2ha approximately midpoint between Perth and Ottawa Streets;
- 39.2ha Ottawa Street road side ditch north and;
- 34.5ha Ottawa Street road side ditch south.

Due to anticipated urbanization of Perth Street and Ottawa Street as well as the site at large, these external areas will be collected and conveyed within storm sewers. These areas will be directed through the stormwater management facilities.

6.2.1 Deviations from Design Guidelines

The design of the sewer outfall from SWM Facility #1 (see Section 7.1 of this report for Facility #1 details) results in a circumstance where it has to cross underneath the existing Moore tributary (see ***Drawing 6*** – *Pond Storm Outfall 1*). Due to site constraints (i.e. grading and both conveyances outletting to the same tributary) the current design is resulting in minimum cover between the ditch invert and the obvert of the sewer outfall (0.10m). At the detailed design stage there may be opportunities to mitigate the cover, however, a deviation from the design guidelines may be required whereby smaller twin pipes would be used to cross under the tributary and as result deviate from the required obvert-obvert connections per *Section 6.2.10* of the ***City Standards***. Additional justification for any required guideline deviation in this location, and potentially at the crossing of Fortune Street, can be provided at the detailed design stage.

6.3 Sump Pump Service

In traditional City servicing arrangements the underside of footings are typically 0.30m above the sewer obvert, the proposed configuration is a minor modification to the configuration where footings are 0.15m above the storm invert. Situating the underside of footings above the sewer invert will promote the accumulation of groundwater to drain away from the house footings toward the sewer system.

Golder and Associates prepared a long-term drainage model and a 100-year storm event model to assess the application of sump pump in accordance with the proposed design. Their analysis concluded that standard, commercially available sump pump, are suitable in this application. ***Appendix F*** contains a specification sheet of a style of sump pump contemplated for the subject site. This arrangement includes a sump pump with backup battery as well as audio and visual alarms. This particular sump pump is capable of discharging at ***140m^3/d*** at the low end of the pump rate range. The anticipated daily inflow rate as determine through the groundwater model is ***1.4m^3/d to 2.04m^3/d*** during a 100-year event. The City of Ottawa's sewer design guidelines suggest a rate of ***0.45L/s/home*** in the absence of data (Section 5.4.7.), however the modeled rate for the maximum required pump rate was estimated to be ***0.012L/s/home***.

The sump pump will be equipped with a swan neck that is 0.30m above the modeled 100-year hydraulic grade line in the street storm sewer. The elevation of the swan neck provides protection to the homes during significant storm events in that the maximum modeled water level in the street sewers will remain below the outlet of the storm service to the house. The outlet of the sump pump will be a minimum of 0.30m above the 100-year hydraulic grade line. Note that the modeled hydraulic grade line within the street sewer system includes contributions from sump pumps. Therefore, the basements of the units are hydraulically separated from the street storm sewers by a minimum of 0.30m as in a typical servicing arrangement. ***Figures 17 and 18*** illustrate the servicing arrangement.

The MOE SWM Planning and Design Manual Suggest sump pumps should not be used where the seasonal high water table is within 1m of foundations, where it is proposed to pump to either an infiltration pit or the surface. In this event it is recommended to discharge to a storm sewer. The proposed servicing arrangement is designed to discharge to a storm sewer.

6.4 Hydraulic Grade Line

JFSA was retained to prepare hydrological and hydraulic models to assess the performance of the sewer system. Results of their analysis is included in ***Appendix I***. Their analysis concluded that the proposed drainage systems safely capture and convey the all storms up to and including the 100-year event in accordance with City of Ottawa modeling practices.

The hydraulic analysis assumed that the external areas would be captured in the minor system. Furthermore, sump pump contribution during the simulated 100-year events assumed that 50% of the sump pumps would be on at any given time and that each lot would contribute 0.012L/s as indicated in Golder's analysis.

Profiles of the proposed Trunk sewers are illustrated on ***Drawings 4, 5, and 6***. The profiles illustrate the existing ground, minimum depth of cover to finished grade, and hydraulic grade line elevations.

7.0 STORMWATER MANAGEMENT FACILITIES

JFSA was retained to prepare hydrological and hydraulic models to assess the performance of the proposed stormwater management facilities. Their analysis is included as ***Appendix I***.

The following summarizes each stormwater management facility as well as the operational parameters of each.

7.1 Stormwater Management Facility #1

Pond 1 is located south of Perth Street and is situated between the regulatory flood limit (94.11m) and the maximum water level estimated on the Van Gaal drain (between 93.72m and 94.04m). The regulatory flood limit was established by way of a backwater effect on the Jock River during a 100-year 10-day snowmelt plus rainfall event that was based on AES Ottawa CDA snowmelt plus rainfall IDF curves. The maximum water level established on the Van Gaal Drain was established based on the same event with some improvements to the Fortune Street culvert. It is proposed to increase the size of the existing 4.2m wide culvert at Fortune Street with a 6.3m span.

The Van Gaal Drain peaks at 113.3 hour into the 10-day event, while the Jock River elevation peaks at 122.0 hour at the Van Gaal Drain. At the 122 hour mark flow contributions from the Van Gaal catchment are significantly lower. JFSA have accounted for the interactions between flow from the development and water levels on the Van Gaal drain. Additional modeling details are provided in ***Appendix I***.

Pond 1 is proposed as a wet pond with an erosion, quality, and quantity release components. Pond 1 outlets to the existing Van Gaal drain. Low flows are directed to a storm sewer and outlet approximately 40m downstream of the Fortune Street culvert. ***Figure 20*** illustrates the proposed pond layout and cross-section. The sewer profile of the outlet is depicted on ***Drawing 6.*** Runoff generated during events in excess of a 25-year storm would be released to the Van Gaal drain through the overflow weir in addition to the low flow outlet. The outlet structure will consist of two orifices to attenuate flow to the Quality and Erosion control targets. ***Table 13*** summarizes the outlet control design parameters.

Table 13
Summary of SWMP 1 Outlet Structure Design

Quality Control	Erosion Control	Quantity Control
Vertical Orifice	Vertical Orifice	Rectangular Weir
300mm dia	300mm dia	45m Wide
INV = 92.35	INV = 92.50	INV = 93.68

Table 14 summarizes the pond's operational characteristics.

Table 14
Summary of SWMP 1 Storage Characteristics

Pond Component	Pre-development Outflow (m^3/s)	Pond Level (m)	Pond Outflow$^{(2)}$ (m^3/s)	Volume Used$^{(2)}$ (m^3)
Permanent Pool	N/A	92.35	N/A	27,226
Quality Control	N/A	92.50	0.038	4,443
2yr/24hr SCS	2.767	93.15	0.295	26,914
5yr/24hr SCS	4.290	93.52	0.377	41,388
10yr/24hr SCS	5.348	93.71	0.753	49,336
25hr/24hr SCS	6.694	93.76	2.017	51,535
50yr/24hr SCS	7.749	93.78	2.832	52,536
100yr/24hr SCS	8.894	93.80	3.579	53,454

7.1.1 Base Flow Augmentation

In coordination with the RVCA, a baseflow augmentation outlet to the receiving watercourse is proposed. The contemplated permanent pool depth of 2.0m, whereas the MOE design guidelines suggest a mean depth between 1.0-2.0m. Therefore, up to 1.0m of contemplated permanent pool is available to provide baseflow augmentation. The permanent pool elevation is estimated at 92.35m. Based on the available topographical information, the elevation at the confluence of the existing Moore ditch bisecting the development and the tributary (aka Arbuckle drain) to the Jock River is approximately 91.70m. Therefore, a baseflow augmentation outlet, via a low flow draw down under-drain system, can be created to draw down a maximum of 0.65m of available pond volume. The location of the baseflow outlet can be seen in ***Figure 20*** and ***Drawing 3***. A backwater valve would also be required in accordance with the Section 7.1.2 of this report for the facility operation discussion.

7.1.2 Facility #1 Operation

In the pond design it is anticipated that spring floodwaters will back into Pond 1 for events more frequent than the 25-year spring event via the 300mm diameter quality control orifice (92.35m invert) and 300mm diameter erosion control orifice (92.50m invert). However, as explained below, until the quantity control weir is overtopped, this effectively only raises the permanent pool (i.e. dead storage) elevation and does not negatively impact the quality control performance of the pond. The base flow augmentation outlet, proposed below the permanent pool elevation of 92.35m, will be equipped with a backwater valve to ensure that the Van Gaal Drain will not back into Pond 1 through this outlet.

The Pond 1 quality control and erosion control orifices outlet to the Jock River tributary downstream of the quantity control weir, via a 750mm diameter pipe discharging to the drain between Fortune Street and Maitland Street. As such, while the 25-year spring water level at the quantity control weir is 93.69 m (HEC-RAS cross-section 961), the water level at the quality/ erosion control outlet is 93.43 m (HEC-RAS cross-section 521). Note that the 2-year spring water levels at these locations are 93.28m and 92.65m, respectively.

The active storage volume required for quality control in Pond 1 is 3,673 cu.m. based on 40 cu.m./ha for a 91.82 ha subdivision drainage area (with no quality control required for the 97.5 natural lands draining to Pond 1 from south of the subdivision). The 25-year spring flood level at the quality / erosion control outlet effectively raises the permanent pool (i.e. dead storage) elevation of Pond 1 from 92.35 m to 93.43 m, leaving 11,046 cu.m. of active storage volume between the effective permanent pool and the invert of the quantity control weir (93.68 m).

Therefore, for spring events up to the 25-year return period, more than sufficient active quality control volume is provided in Pond 1 below the quantity control weir to treat runoff from the 91.82 ha subdivision drainage area. As previously noted, for events exceeding the 25-year return period, flood levels will not overtop the quantity control weir until almost 5 days into the 10 day event; well after the "first flush" containing much of the winter's accumulation of road salt and grit has passed through the pond.

Similarly, the "first flush" of the 2- to 25-year spring events, taken as the first 25 mm of rainfall / snowmelt, will pass between 30 to 100 hours before the peak on the Van Gaal Drain.

It is noted that the 80% TSS removal required by the MOE is not a target to be met for every storm, but a long term statistical average. Therefore, some storm events, and in particular large rainfall events, will not have 80% TSS removal; this is balanced by the TSS removal for smaller, more frequent storm events.

7.2 Stormwater Management Facility #2

Pond 2 is located north of Ottawa Street and is proposed to be a wetland facility with quality and quantity release components. A wetland facility is proposed at this location in order to minimize or avoid any rock removal. The use of a wet pond, requiring a deeper permanent pool, would require rock removal and potential negative impacts to neighboring wells and the constructability of the pond. See Golder memo 12-1127-0062 in Appendix F.

Pond 2 outlets to the Jock River via a 1500mm diameter sewer. In the event of a blockage, the emergency overflow is directed to the north towards the Fortune Street ROW and ultimately to the tributary to the Jock River (aka Arbuckle Drain).. ***Figure 21*** illustrates the proposed pond layout and cross-section. The sewer profile of the outlet is shown on ***Drawing 6*** (labelled Outfall 2)***.***

The outlet structure will consist of a vertical orifice and rectangular weir to attenuate flow to the quality and quantity control targets. ***Table 15*** summarizes the outlet control design parameters.

Table 15
Summary of SWMP 2 Outlet Structure Design

Quality Control	Quantity Control
Vertical Orifice	Rectangular Weir
300mm dia	1.50m Wide
INV = 93.20	INV = 93.35

Table 16 summarizes the pond's operational characteristics.

Table 16
Summary of SWMP 2 Storage Characteristics

Pond Component	Pre-development Outflow (m^3/s)	Pond Level (m)	Pond Outflow$^{(2)}$ (m^3/s)	Volume Used$^{(2)}$ (m^3)
Permanent Pool	N/A	93.20	N/A	2,990
Quality Control	N/A	93.35	0.038	1,837
2yr/24hr SCS	0.705	93.68	0.591	6,618
5yr/24hr SCS	1.094	93.82	0.947	8,987
10yr/24hr SCS	1.364	93.91	1.195	10,526
25hr/24hr SCS	1.708	94.02	1.514	12,430
50yr/24hr SCS	1.976	94.11	1.765	13,893
100yr/24hr SCS	2.267	94.20	2.042	15,471

7.3 Pond Maintenance – Sediment Loadings

Routine maintenance will be required to ensure that the facilities continue to function as designed. The proposed sediment forebays will capture much of the suspended solids being transported by the storm events to the facility. Sediment accumulation rates in the forebay will vary, however, a base estimation of sediment loading has been established as per "Table 6.3 – Annual Sediment Loadings" in the ***SWMP Design Manual (MOE, March 2003).***

The sediment drying area has been sized for 10 years of sediment accumulation. The sediment drying areas are located near the maintenance road and are above the 100-year water level. The loading determinations and estimated sediment drying areas required can be found in ***Appendix I*** and are summarized as follows:

SWM Facility #1 – accumulation of 1,488 m^3 with a required drying area of 1,489 m^2 (1,700 m^2 provided)
SWM Facility #2 – accumulation of 567 m^3 with a required drying area of 567 m^2 (1,200 m^2 provided)

7.4 Stormwater Management Conclusions

JFSA reviewed the impact of the proposed subdivision, the associated storm sewer collection system, and stormwater management facility designs in accordance with standard City of Ottawa modeling techniques.

Peak flow to the Van Gaal Drain generally decreases between the existing and proposed conditions. The volume attenuated in stormwater management facility 1 is primarily responsible for the decrease in flow.

8.0 WATER BUDGET

To investigate the effect of proposed developments on existing infiltration rates the pre and post development hydrologic models prepared for this study were converted to continuous simulations. This included the conversion of CALIB NASHYD and CALIB STANDHYD commands to CONTINUOUS NASHYD and CONTINUOUS STANDHYD. The input and output files are included on a CD at the back of this report. These new hydrograph commands add time dependent parameters used in updating various hydrologic data during continuous simulations including initial abstraction recovery time, interval event time, etc. These new hydrographs commands are used with a COMPUTE API (Antecedent Precipitation Index) command which also updates various hydrological parameters during continuous simulations. Simulations were completed using AES (Atmospheric Environment Services Canada) rain gauge data from 1967 through to 2003 (excluding missing 2001 rainfall data).

Table 17 summarizes the estimated average annual infiltration volume for the proposed development under existing conditions and post-development conditions. The complete results of the Continuous simulation from 1967 to 2003 are located in ***Appendix J*** along with a technical memorandum prepared by JFSA.

Table 17
Pre-development Infiltration

Description	**Area (ha)** (1)	**Estimated Annual Infiltrated Volume (m^3/yr)**		
		Average	**Minimum**	**Maximum**
Total Rainfall	126.81	747,066	407,821	1,187,449
Runoff (no Infiltration)	126.81	415,191	233,521	713,598
Runoff (With Infiltration)	126.81	201,113	99,838	403,700
Infiltration	126.81	**214,079**	109,247	328,818

(1) Note: For a 126.81ha drainage area as per the October 2013 Preliminary Stormwater Management Plan memo by JFSA (found in ***Appendix I***).

Table 18 summarizes the average percent decrease in infiltration as a result of development. As demonstrated, directing roof leaders to grassed areas improves post-development infiltration substantially.

Table 18
Post-development Infiltration

Description	Area (ha)	Estimated Annual Infiltrated Volume (m^3/yr)		
		Average	Minimum	Maximum
Total Rainfall	126.81	747,066	407,821	1,187,449
Runoff (no Infiltration)	126.81	425,089	241,345	724,288
Runoff (With Infiltration)	126.81	336,083	189,264	595,639
Infiltration	126.81	**89,007**	47,693	143,321

(1) Note: For a 126.81ha drainage area as per the October 2013 Preliminary Stormwater Management Plan memo by JFSA (found in ***Appendix I***).

Based on the existing and proposed continuous simulations, the average annual infiltration over the drainage are is approximately 58.4% less under proposed conditions (89,007 m^3) than under existing conditions (214,079 m^3).

In addition to the above, there are areas of the development land (i.e. southwest portion) where there are silty sands which may be conducive to the incorporation of infiltration measures. It is proposed that a modified City Standard S29 (Perforated Pipe installation) be utilized which will include an additional depth of clear stone below the pipe invert. This added measure will be limited to rear yards and park spaces where receiving sewers will not be designed to accept flow from streets.

9.0 CONCLUSION AND RECOMMENDATIONS

Richmond Village (South) Limited has retained David Schaeffer Engineering Ltd. (DSEL) to prepare a Stormwater Management Report in support of their application for draft plan of subdivision.

This report provides sufficient detail with respect to the stormwater management system including minor and major system conveyance and stormwater facilities to support the Richmond Village (South) Limited's draft plan of subdivision application. Furthermore, this study provides a stormwater solution for the adjacent development lands.

DSEL recommends the following:

- Works required to amend floodplain limits will need to be implemented prior to the development proceeding;
- A storm sewer system consisting of a minor and major systems with homes serviced via sump pumps is recommended;
- Two stormwater management facilities will service the development lands;
- A site grading scheme was developed to ensure major system conveyance and respect the grade raise restrictions, exceedance was limited to small areas north of Perth Street;
- The stormwater management system results in slightly lower water levels on the Van Gaal Drain;
- The proposed stormwater management system meets the design objectives.

Prepared by,	Reviewed by,
David Schaeffer Engineering Ltd.	**David Schaeffer Engineering Ltd.**
Per: Adam D. Fobert, P.Eng	Per: Stephen J. Pichette, P.Eng

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APPENDIX A

City of Ottawa / Agency Comments and Consultation Summary (from March 2010 Report)

Attachment 1

Item		Comment	Action / Response	Reference	City Response	
City Comments Dated December 14, 2012 - Richmond Village - SWM and Drainage RE: Western Development Lands					**11-Jul-13**	
From: Darlene Conway To: Cheryl McWilliams						
	A. Major system drainage analysis					
NEW		We have noticed that the 100-Year, 24hr SCS rainfall volume used for the drainage area to the Van Gaal Drain and the Jock River is smaller than the 106.7 mm volume that should be used according to the City Ottawa Sewer Design Guidelines. Please clarify.				The 100-year, 24-hour SCS type II storm used to model the Van Gaal Drain and Jock River is based on Ottawa CDA rainfall data (88.6 mm volume) rather than City of Ottawa Sewer Design Guidelines (106.7 mm volume). This design storm was used in order to be consistent with the November 2004 *Jock River Flood Risk Mapping (within the City of Ottawa) Hydraulics Report* , and the November 2009 *Floodplain Mapping Report for the Van Gaal and Arbuckle Drains in the Village of Richmond* .
1	1	The major system drainage analysis has been reviewed based upon the revised draft plan. As previously commented, it appears that subsequent to the STANDHYD command, each catchment is routed within its down drainage area (rather than routing commencing in the adjacent downstream catchment), using a ROUTE CHANNEL command. For example, the total flow from catchment 705b is routed from MH 701 to MH 705 (see Figure 1). This is typical of how the overland flows have been routed which appears to provide more attenuation than may actually occur. We do not agree that this is consistent with standard modeling practice. While this may be the case for DDSWMM (where catchments are typically quite small), this is not the case for how this development has been modeled with SWMHYMO (larger, longer catchments). Channel routing should not be applied to the flow generated within its own catchment at this level of analysis.	In accordance with City comments, the SWMHYMO model will be revised to commence routing for each catchment in the next downstream segment, rather than within the catchment itself. Please note that the model has been updated accordingly for the subsequent Mattamy draft plan application submitted to the City, April 2013.	Appendix I	Comment addressed.	
2	2	Section 6.1 of the DSEL report states: "Where major system flow is shown to cross Perth Street and Ottawa street, the minor system was designed to convey flow under these streets." From the SWMHYMO output files, it appears that the major flow is not fully captured (e.g., carry over is 112 l/s for Trunk 2 at MH 213 and 680 l/s for Trunk 11 at MH 1109). Please revise the modeling and sewer sizing as required and confirm how the major flow is captured before it crosses Perth Street and Ottawa Street (i.e., provide an indication of the CBs required. Please note excessive numbers of catchbasins should be avoided).	100% of the 100-year flows at Perth Street (538 L/s) and Ottawa Street (680 L/s) are captured to the minor system at SWMHYMO segments 1500aX and 1600aX, including the flows identified in Village comments for Trunks 2 and 11. Four double catchbasins (two on each side of the street) should be sufficient to capture these 100-year flows on Perth Street and Ottawa Street even with 50% blockage of the inlet grates.	Appendix I	Comment addressed.	
3	3	It appears that the carry over flow from Trunk 1 is not accounted for downstream of MH 107. In other words, the overland flow from MH 107 to MH 109 is missing (see Figure 2). Please revise the modeling and storm sewer sizing as required.	100% of the 100-year overland flow from MH 107 is captured to the minor system at SWMHYMO segment 1500aX, in order to prevent overland flow from crossing Perth Street.	Appendix I	Comment addressed.	
4	4	To inform the future draft plan, minimum widths for the easements located downstream of MH 710 and MH1108 must be confirmed based upon the respective proposed sewers and overland flows to be conveyed (provide conceptual calculations for each location).	The easement width is depicted as 6.0m per City Design guidelines. The 100-year flow to the 6.0 m wide easement from MH 1108 is approximately 1.04 cms, and the 100-year flow to the 6.0 m wide easement from MH 710 is approximately 1.16 cms. A trapezoidal channel with 4.0 bottom width, 6.0 m top width, 0.5% longitudinal slope and 3H:1V side slopes would be sufficient to convey these flows at depths of less than 30 cm. Therefore, the indicated easement width is sufficient to convey the 100-year flow and contain the proposed storm sewer.		Please provide typical details, calculations and confirm that sufficient freeboard is provided.	Note: Updated modelling area breakdown and flow routes have resulted in flow rates that differ than those discussed previously. A 4.0 m wide overland flow route in the easement from MH 710 to SWM Facility 1, with a curb cut width of 4.0 m, will convey the 100-year overland flow safely to the SWM facility without flooding any of the properties within the subdivision. Note that the 100-year flow depth of 12.8 cm is contained within the 6.0 m wide easement assuming 3H:1V side slopes for 1 m on either side of the 4.0 m bottom width (33.3 cm maximum depth). A 4.0 m wide overland flow route in the easement from MH 1108 to SWM Facility 2, with a curb cut width of 4.0 m, will convey the 100-year overland flow safely to the SWM facility without flooding any of the properties within the subdivision. Note that the 100-year flow depth of 24.6 cm is contained within the 6.0 m wide easement assuming 3H:1V side slopes for 1 m on either side of the 4.0 m bottom width (33.3 cm maximum depth).
5	5	As previously commented, it must be confirmed how the major system flows from south of Pond 1 will be conveyed into Pond 1 without filling in the floodplain (see figure 3). Again, provide conceptual design/grading, easement width, cross-section, etc.	Conceptual grading is illustrated on Drawing 2. Upstream road grade is 95.78m, 100-year floodplain elevation is 94.11 at this location. No filling is being proposed with floodplain.	Drawing 2	Comment not addressed: The level of detail shown on Drawing 2 of the latest Functional Servicing Report (DSEL, April 2013) does not demonstrate that the major system flow can be conveyed to Pond 1 without filling in the flood plain. Please provide sufficient detail to demonstrate this.	No filling of the floodplain (100-year Spring = 94.11 m) will take place in order to convey overland flows to Pond 1; the subdivision areas to be developed south of Pond 1 are outside of the floodplain under existing conditions, as the limits of the pond block are defined by the floodplain. The roads south of Pond 1 are graded between 94.76 m and 95.05 m under proposed conditions. It should be noted that overland flows from 30.79 ha of the subdivision will discharge directly to the Van Gaal Drain upstream of Perth Street, and overland flows from 21.75 ha of the subdivision (and 94.2 ha of natural lands to the south) will discharge directly to the Moore Drain Tributary. Furthermore, 100% of the 100-year flows draining to Perth Street from 3.71 ha of the subdivision (and 97.5 ha of natural lands to the south) will be captured to the minor system. As such, only 35.57 ha of the subdivision will drain overland to Pond 1.

6	6	Section 6.1 of the DSEL report states: "The hydraulic analysis assumed that the external areas would be captured in the minor system". Please confirm the conceptual design of the inlets required to convey this external flow.	The following 100-year flows are to be captured to the minor system from external areas (refer to Attachment 1 of the November 9, 2012 memo for area characteristics): 2392 L/s from VG-2; 992 L/s from VG-3; 867 L/s from VG-5; 908 L/s from VG-7 and 1259 L/s from JR-1. A 1200 mm x 600 mm ditch inlet catchbasin (DICB) has a capacity of approximately 741 L/s under a 30 cm head, and a 600 mm x 600 mm DICB has a 370 L/s capacity under 30 cm of head. The lead pipes of these ditch inlet catchbasins will be sized, based on head over the lead pipe to be determined at the detailed design stage, such that they do not restrict these inflows. One or two ditch inlet catchbasins on each area, as appropriate, should therefore be sufficient to capture the 100-year flows to the minor system for all but VG-2. The 2392 L/s 100-year flow on VG-2 may be captured by four 1200 mm x 600 mm DICBs, or by other means deemed appropriate at the detailed design stage.	Appendix I	Comment addressed.	
7	7	As per the Technical Bulletin (January 2012), the maximum flow depth on streets under either static or dynamic conditions shall not exceed 30 cm. From the modeling results provided, Table 1 summarizes the locations where the dynamic depth exceeds 15 cm. Please confirm that the assumptions used to account for the static ponding (stage-storage relationship) account for the reduced static ponding depths that would be available in these locations.	The model assumes that 30 cu.m./ha of surface storage, on average, will be provided on the streets. Exact surface storage volumes for each street segment will be calculated and modelled at the detailed design stage. Any reduced surface storage in segments with increased flow depths can be compensated for in other segments with the opportunity for greater than average surface storage, as necessary.	Appendix I	As previously requested, please confirm that the assumptions used to account for the static ponding (stage-storage relationship) account for the reduced static ponding depths that would be available in these locations.	Yes, assumptions used to account for the static ponding account for the reduced static ponding depths that would be available in locations with significant flow depth.
8	8	Where are details of the future cross-section of the Moore Drain provided? Confirmation should be provided that the cross-section has sufficient capacity to convey the 100 year peak flow through the subdivision.	Figure 22 illustrates the conceptual design of the Moore channel. The Moore Channel was included in the overall proposed conditions Van Gaal Drain / Jock River model. The model shows some encroachment outside of the block due to the culverts for the road crossing. While the channel is adequate the culverts will need to be reviewed. Note that the proposed conditions model also included: - proposed conditions flows as simulated in SWMHYMO - the realignment of Van Gaal Drain upstream of Perth Street - the replacement of the Forture Street culvert	Appendix I and Figure 22.	There is insufficient clearance between the storm sewer and the proposed Moore tributary channel grade at Street 3. In addition, the design of the culvert must be revised to avoid any encroachments of the 100-year water level outside of the block. Please also plot the 100-year water level on the drawing to clearly demonstrate it is contained and ensure that sufficient freeboard is provided to the adjacent lots. What roughness has been assumed for the channel? To aovid the need for excessive maintenance, a manicured condition should not be assumed.	The 900 mm diameter circular culverts proposed at the three proposed crossings of the Moore Drain Tributary have been revised to 1800 mm by 900 mm rectangular culverts; under these conditions, there are no encroachments of the 100-year flood level outside of the channel block. The Moore Drain Tributary was modelled under summer conditions using Manning's roughness coefficients of 0.035 and 0.08 in the low flow channel (3 m top width) and on the banks, respectively. Under spring conditions, the Manning's roughness coefficient will remain as 0.035 in the low flow channel, but is reduced to 0.05 on the banks to reflect the lack of standing vegetation during that time of year. These values are appropriate for channels that are not subject to regular maintenance.
	B. Erosion Concerns					
9	1	To complete the review of the report (Van Gaal Drain Erosion Assessment, JTBES, October 26, 2012) and follow-up memo (Richmond Village Development: Exiting Erosion Remediation Costs, JTBES, November 22, 2012) a location plan identifying the location of the erosion sites identified is required. Please provide such a figure (i.e., locating all erosion sites and clearly identifying those proposed for remediation). Also, some description/summary of the high and medium priority class sites should be provided in the body of the report and/or in a summary table format, i.e., what is the particular infrastructure and/or property threatened? private or public, etc.?	JTBES prepared "Van Gaal Drain Restoration Memo" - January 25, 2013 and has included figures indicating a location plan.	JTBES - Van Gaal Drain Restoration Memo, January 25, 2013.	Refer to separate memo from Darlene Conway, P. Eng., dated July 11, 2013, for comments on the JTBES memo of January 25, 2013.	Please refer to October 31, 2013 *Richmond Village (South) Limited Subdivision / Continuous Erosion Analysis* memo by JFSA found in Appendix D. Coldwater Consulting Ltd report can be found in Appendix D.
10	2	While the report uses the term "Van Gaal Drain," it is important to note that the reaches in question are not part of the Van Gaal, but part of the Arbuckle Award Drain which does not have municipal drain status. As much, if not all, of the reaches in question are on private property, access to the erosion sites for remediation purposes will require a Drainage Act process in order to secure access to private property. This will presumably have to be coordinated with the proposed realignment of the Van Gaal Drain upstream of Perth St. In the alternative, drainage easements across all private properties affected will have to be negotiated if the advantage of using the Drainage Act is not taken. Please confirm how the required remediation work is to proceed.	Richmond Village Ltd will pursue coordinating the finalization of the engineer's report to create the Arbuckle Drain.		Confirmation from the proponent(s) of their commitment to implement the recommended works via a Drainage Act process and to the satisfaction of the City is required.	Caivan
11	3	Given that the subsequent JTBES memo of November 22, 2012 recommends rehabilitation of the erosion sites identified in the October 2012 JTBES report, detailed comments will be not be provided on the October 2012 report. However, based upon the information provided to date, the City questions the report's conclusion that the development will not exacerbate existing erosion rates in the drain. Nevertheless, subject to confirmation of the proposed remediation work (provide further details as noted above) and the proponent's commitment to undertake the required works to the satisfaction of the City and in keeping with the required Drainage Act process (or negotiation of required easements), this conclusion would appear to be moot.	No comment.		Confirmation from the proponent(s) of their commitment to implement the recommended works via a Drainage Act process and to the satisfaction of the City is required.	Caivan

12	4	The recommendation for the application of the suggested erosion threshold to the design of SWM pond 1 requires clarification. Specifically, on p.15, the report notes: Assessment of the conditions of the creek show that the banks are comprised of consolidated clay materials, ranging from coarse to fine clay. When these materials are exposed to flowing water, velocities of between 0.225 metres per second (coarse clay) and 0.400 metres per second (fine clay) are required to entrain (erode) these materials (ref. Hjulstrom, 1935). The report then recommends the following re: discharge from the SWM pond: Stormwater discharge from new facilities to the Drain be controlled to a maximum velocity of 0.225 metres per second for all flows up to and including the 2-year event, and to as many return events as is possible above the 2-year event. If the 0.225 m/s criterion is based on the erodability of the drain's banks, then the impact of the attenuated pond discharge should presumably be assessed based upon the total flow in the receiver, i.e., the pond discharge plus the upstream flow and how this does or does not meet the specified critical velocity and associated discharge (in the receiver)?	JTBES indicates that flows from the pond be controlled to a maximum velocity of 0.225m/s. This low target will not exacerbate the existing erosion occurring within the drain. Having said that, Richmond Village Ltd are pursuing the finalization of the Arbuckle Drain. JTBES indicates that the drainage works will provide an erosion threshold release target of at least 330L/s.	JTBES - letter Re: Richmond Village Development: Van Gaal Drain Erosion Thresholds, March 6, 2013.	**Comment not addressed:** Related to this comment, the purpose of the March 6, 2013 letter is not clear and it appears the intent of this comment may not have been understood. Further discussion with City staff is recommended to facilitate the resolution of this and other related comments.	Please refer to October 31, 2013 *Richmond Village (South) Limited Subdivision / Continuous Erosion Analysis* memo by JFSA. Coldwater Consulting Ltd report can be found in Appendix D.
14	5	There also appears to be a lack of coordination between the design of SWM pond 1 and the recommendations from the October 2012 JTBES report. i) From the JFSA memo of November 9, 2012 (Appendix I of DSEL November 2012 report): Erosion control for SWM Pond 1 will be provided by controlling the 2-year release rate from each pond to 330 L/s or less, where 330 L/s is the erosion threshold for the Van Gaal Drain identified by Parish Geomorphic in the Natural Environment & Impact Assessment Study for the Mattamy Richmond Lands (March 2009). Furthermore, the October 26, 2012 Van Gaal Drain Erosion Assessment memo by JTB Environmental Systems Inc. indicates that the 2-year outflows from Pond 1 should discharge to the Van Gaal Drain at a velocity of 0.225 m/s or less. This may be achieved by a plunge pool or other velocity reduction measures at the Pond 1 extended detention outlet pipe to the Van Gaal Drain. Is there any relationship between the Parish release rate and the JTBES critical velocity? Also, as noted above, the relevance of a plunge pool to limit discharge velocities from the pond is not apparent?	Addressed by JTBES. We may continue to use 330L/s.	JTBES - letter Re: Richmond Village Development: Van Gaal Drain Erosion Thresholds, March 6, 2013.	**Comment not addressed:** Related to this comment, the purpose of the March 6, 2013 letter is not clear and it appears that the intent of this comment may not have been understood. As previously requested, the provision of a continuous pre- and post simulation exercise is required, coordinated and integrated with the fluvial geomorphological work. Further discussion with City staff is recommended to facilitate the resolution of this and other related comments.	Please refer to October 31, 2013 *Richmond Village (South) Limited Subdivision / Continuous Erosion Analysis* memo by JFSA. Coldwater Consulting Ltd report can be found in Appendix D.
15	6	In summary, an understanding should be provided of the overall response in the receiver and a comparison made between existing and post-development conditions. As previously requested, a continuous simulation should be undertaken to assess the impacts of increased runoff volume, peak flow attenuation and the increased duration of flows. To demonstrate that the recommended targets are achieved, assessments of velocities at critical cross-sections in the drain for representative events should be provided for existing and post development conditions with the proposed stormwater management solution and instream works in place.	In accordance with City comments, a continuous model will be prepared to compare pre- and post-development peak flows and flow duration over a given threshold. Furthermore, velocities at critical cross-sections in the drain will be compared under pre- and post-development conditions for an appropriate design storm.	To be provided.	**Comment not addressed:** Please provide the information requested for review.	Please refer to October 31, 2013 *Richmond Village (South) Limited Subdivision / Continuous Erosion Analysis* memo by JFSA found in Appendix D. Coldwater Consulting Ltd report can be found in Appendix D.
	C. Class EA					
16	1	The most recent response to this issue, provided in the Roberts memo of November 9, 2012, does not address the issue of piecemealing as originally raised in July 2010 (see Attachment 1). If the position is that this approach does not represent piecemealing as defined in the MEA Class EA then this should be documented by the proponent. As there appears to be a difference of professional opinion on this matter and given that piecemealing is in contravention of the EA Act, this should also be confirmed with the Ministry of the Environment.	Having reviewed the correspondence related to piecemealing, it has become evident that there is a misunderstanding on the operation of the sump pumps. The sump pumps are not connected to the sanitary sewer system. As attached to the referenced correspondence, the MOE had provided feedback on this matter.	Delcan / Soloway Wright / RVCA / MOE	**Comment not addressed:** There is no misunderstanding. The issue of piecemealing has not been addressed: please refer to the correspondence noted in the previous comment. As per the e-mail from Cheryl McWilliams to Sarah Millar Martin (February 13, 2013), it is the City's position that an addendum to the Master Servicing Class EA is required.	Caivan?
	D. Pond design/modelling					
17	1	Please expand the summary tables that compare pre- and post-development peak flows and water levels to include the full range of frequency events.	In accordance with City comments, pre- and post-development peak flows and water levels on the drain under spring and summer conditions will be compared for the 2-, 5-, 10-, 25- and 100-year return periods.	To be provided.	**Comment not addressed:** Please provide the information requested for review and approval.	Please refer to Attachment 7 of the October 31, 2013 *Richmond Village (South) Limited Subdivision / Preliminary Stormwater Management Plan* memo by JFSA found in Appendix I.

18	2	Per previous comments: As currently proposed, the bottom of Pond 2 is some 3 to 4 meters below existing grade and into bedrock. The bottom of Pond 1, some 3 meters below existing grade as currently proposed, may be at or marginally below bedrock. Some of the sewers (storm and sanitary) may also intercept the bedrock, in particular below Ottawa St. There has been no discussion provided as to the potential impacts on pond operation, existing wells, contribution to base flows, etc. Some discussion should be provided regarding these potential concerns and recommendations provided to mitigate any potential impacts provided as required.	The results of the subsurface investigations indicate that the hydraulic conductivity of the shallow bedrock in the area of Pond 1 and Pond 2 is moderate (1x10-5 m/s and 5x10-6 m/s at MW10-3 and MW10-6 respectively), similar to the hydraulic conductivity of the overlying silty sand and silty clay deposits. However, the design of the ponds should consider the possibility that vertical fractures in the shallow bedrock might permit increased groundwater flow through the bottom of the ponds, which could hamper their construction. Therefore, shallower pond designs should be considered if construction in the bedrock is not required, and a plan to control the inflow of water from the shallow bedrock during construction of the ponds should also be developed, if necessary. The normal operating levels in Ponds 1 and 2 will be approximately 1.5 and 1.2 metres below the typical groundwater levels as measured in groundwater monitors MW10-3 and MW10-6 respectively. Therefore, groundwater levels in in the areas immediately adjacent to the ponds (including in the upper bedrock) will be reduced by a maximum of approximately 1.2 to 1.5 metres. The effect of the ponds on groundwater levels will diminish at greater distance from the pond. Considering that local water wells are typically cased a minimum of 6 metres below ground surface and are on the order of 30 meters deep, such small and localized reductions in the groundwater levels is not expected to negatively impact the water supply of local water wells. DSEL has reviewed pond 2 in further detail. Pond 2 is revised to a dry pond employing hydro-dynamic separators to meet quality control objectives. The bottom of Pond 2 is situated above the rock elevation per test pit data.	To be provided.	It is not possible to review or comment on this response in the absence of supporting documentation and design information. Please provide the supporting information for review, sealed as required by the professional making the recommendation(s). Please also note that the City is inquiring about the risk to wells from pond leakage through fractures. Notwithstanding the hydraulic conductivity derived from a continuum (macroscopic) approach, porosity will vary erratically at volumes lower than the representative elementary volume. Individual fractures become significant at the finer level, particularly for contaminant transport. Also, please note that the City was not inquiring about the construction methodology of the pond, but rather about the long-term effect of groundwater on its operation, existing wells, contribution to base flows, etc.	As discussed with City staff a wetland facility is proposed for Pond #2. Incorporation of this type of facility has allowed for a bottom elevation that is above the anticipated rock surface and therefore minimizes disturbance. Golder Associates has provided a memo for the justification of avoiding rock excavation (see Appendix F and discussion in Section 7.2)
19	3	From the Nov.9, 2012 JFSA memo (Appendix I) - where is the future 16.5ha commercial site that is referenced located?	The 16.5ha property are the lands north of Perth street, immediately west of the Richmond Village (North) Ltd., parcel, inside the Western development expansion area, and includes the existing retail parcel on Perth Street. This is to be revised to represent the land uses contemplated in the CDP.	Appendix I	Please clearly delineate all external areas on the drainage plan for existing and future conditions.	Base plans have been updated to reflect the areas. Note that the 16.5ha area is no longer being considered as a commercial site and is being modelled as residential in line with the CDP.
		Pond 1				
20	4	Per previous comments: The justification for locating the pond in the floodplain is based upon locating it above the summer event given that the higher regulatory flood level results from a backwater effect from the Jock River. However, the 100-year spring event on the drain (93.83m at 910), though lower than the backwater condition from the Jock, is still higher than the summer event (93.60 at 910, Fortune St. culvert upgraded). While the timing of peak flows may apply to the much larger Jock River and the spring event backwater condition, it is not apparent this is an appropriate assumption when comparing spring and summer events on the much smaller drain. Further, the rationale of locating the pond only above the summer condition floodplain could set a precedent such that ponds are proposed in future within the regulatory (spring) floodplain where there is no comparable backwater effect. From a technical perspective, the pond should be located above the 100-year spring event on the drain.	JFSA completed additional hydraulic modeling of the development and pond's impact on water levels within the receiving water courses. The analysis has demonstrated technical feasibility. Noting that this is a unique feature of these lands in particular.	Appendix I	This rationale is inconsistent with the rationale being used to justify locating the pond within the regulatory (spring) event on the Jock River and sets a bad precedent, however, resolution of this matter is deferred to RVCA.	See discussion of the SWM Pond #1 operation in Section 7.1.2
21	5	Per previous comments, a 0.30m freeboard is required above the design high water level. This is a standard design criterion that must be incorporated into all pond designs – in this case, without filling in the floodplain. Therefore, the pond design must be revised accordingly to explicitly achieve this criterion.	As per the MOE SWMPD manual, a 0.3 m freeboard should be provided between the design high water level in the pond and surrounding grades. This qualification does not apply to that side of the pond adjacent to the Van Gaal Drain, which will be inundated by floodwaters during significant spring flood events. However, a 0.3 m freeboard should be provided on the sides of the pond adjacent to the proposed development (roads, residential lots, etc.).	MOE Design Guidelines - Section 4.2 Sitting of Facilities.	**Comment not addressed:** It appears that the purpose of providing freeboard for engineered works may not be fully appreciated. In this case, a key purpose of the pond is to provide flood control storage: providing the minimum standard freeboard around the pond ensures that a reasonable safety factor is provided that recognizes inherent modeling limitations, allows for construction tolerances, minor future settlement, etc., to ensure that the design flood control storage will be available for the life of the pond. It is not relevant whether the pond fronts on lots or onto the floodplain. Please revise the design accordingly such that the minimum standard freeboard of 0.3m is provided above the pond's design (100 year) water level for the entire pond.	JFSA / DSEL / City to meet and debate this point.
22	6	The draft plan indicates 2 sediment management areas are provided for Pond 1 (total area approximately. 2100 m2). The City requires that these areas be sized approximately equal to the size of the forebay(s) at the permanent pool elevation (total area approximately 6000 m2). Adjust the draft plan to provide for sufficient area for sediment storage purposes outside of the regulatory floodplain. These areas should also be clearly demarcated on the appropriate figures in the DSEL report.	The sediment pond drying area was sized employing the MOE Design guidelines for estimating sediment build-up in the forebays and a clean-out frequency of 20-years. This volume would be spread over the drying area at an average depth of 0.60m.	Calculation to be included in future submission	**Comment not addressed**: Please provide the calculations/documentation and identify the required sediment drying area on the drawings for review and approval.	As discussed at Sept 12/13 meeting, drying area is shown. Sediment loading calculations can be found in Appendix I and a summary is provided in Section 7.3.
23	7	Please indicate the location of the Pond 1 spillway on Drawing 3 and Figure 20. Also note that the scale on Drawing 3 should be 1:4000.	Duly noted		**Comment not addressed:** The pond 1 spillway is still not shown on Drawing 3.	See updated Drawing 3 and Figure 20.
24	8	Per previous comments, the outlet invert elevation for Pond 1 (92.35m) is lower than the 2 year flood elevation. This is not consistent with the criterion specified in the MOE SWM Planning and Design Manual.	The MOE recommendations reads that the SWMP should be higher than the 2-year floodline and the overflow must be above the 25 year floodline.	MOE Design Guidelines - Section 4.2 Sitting of Facilities.	**Comment not addressed.**	Agreed to at Sept 12/13 meeting that the pond can have its outlet lower than the 2-year event. Pond design remains as is.
		Pond 2				

25	9	As currently proposed, the quantity control weir elevation for Pond 2 (93.35m) is below the 25-year floodline (93.82 m). This is inconsistent with the design criteria in the MOE SWM Planning and Design Manual	The design criteria speak to the overflow elevation above the 25-year, not the outlet. Pond 2 emergency overflow is at 96.00m spilling toward Burke Street. Note that 100-year elevation is Pond 2 is 94.31m.	Figure 21.	Based on the proposed design of pond 2, the water quantity weir elevation is below the 25 year floodline. As shown in Table 5B, the elevation of 96.0 m corresponds to the top of berm: how has the design accounted for a controlled spill out of the pond? (based on the proposed grading, the pond will spill toward the existing lots located on Ottawa Street).	The water quantity weir will function as a partially-submerged weir once the water level in the SWM facility exceeds that of the Jock River at the proposed outlet. The performance of the facility under these conditions was verified in XPSWMM based on the 100-year spring flood level of 94.18 m on the Jock River (conservatively assuming initial conditions where it has backed up into the SWM facility such that it also has a water level of 94.18 m). Under these conditions, the maximum water level in the facility was simulated as 94.84 m - well below the top of berm at 96.0 m.
26	10	The block for Pond 2 has not accounted for the required sediment drying storage area – please revise/expand the block accordingly.	Please note that sediment drying area indicated on Figure 21.	Figure 21.	**Comment not addressed**: As for Pond 1, please provide the calculations/documentation supporting the sizing of the sediment drying area.	As discussed at Sept 12/13 meeting, drying area is shown. Sediment loading calculations can be found in Appendix I and a summary is provided in Section 7.3.
27	11	It is not clear where the emergency overflow from Pond 2 would be directed (should blockage or partial blockage of the outlet structure occur). Please confirm there is an appropriate emergency outlet/flow path to the river that will not impact existing and future homes and indicate this on the appropriate figures.	Pond 2 emergency overflow is at 96.00m spilling toward Burke Street. Note that 100-year elevation is Pond 2 is 94.31m, therefore there is 1.7m of available emergency storage.	Figure 21.	For 100 year conditions, the flow from the outlet of pond 2 is about 2.14 cms, which exceeds the capacity of the downstream trunk sewer: confirm that any surcharge does not exceed the road elevation and/or increase the pipe size accordingly.	As discussed at Sept 12/13 meeting, emergency outlet to demonstrate conveyance to the Jock River tributary (aka Arbuckle Drain). Due to road grading constraints, an outlet along rear yards is shown with a positive outlet to Queen Charlotte Street. The SWM facility outlet pipe has been updated to reflect a 1500 mm diameter pipe, with a capacity of 2.235 m^3/s.
		Figure 4, derived from the City's LiDAR data, indicates fill areas even without the required minimum freeboard provided.			**Comment not addressed**: As previously commented, please revise the design to ensure minimum freeboard is provided and filling in the regulatory floodplain is not required to provide the necessary quantity control storage.	As discussed at Sept 12/13 meeting, City defers to the RVCA.
	E. Storm sewer servicing					
28	1	Per standard design practice, sewer obverts should be matched at the crossing of the Moore Drain and minimum clearance provided between the Moore Drain crossing and the storm sewer.	Requesting deviation from Sewer Design Guidelines (6.2.10). In regards to standard engineering practices - invert to invert connections do not have a noticeable effect on hydraulics (See Haestad Methods Stormwater Conveyance Modeling and Design - Page 421/422.) Also note that invert to invert connections have been made numerous other circumstances when either depth of cover or volume of imported material is a problem.	Haestad Methods - Stormwater Conveyance Modeling and Design - Page 421/422	Matching obverts is a basic design requirement. Please revise the design accordingly. However, if a deviation is pursued further, rationale per the Sewer Design Guideline's exception criterion is to be provided for vetting by the Infrastructure Services and Environmental Services Departments.	Agreed to at Sept 12/13 meeting that deviation is acceptable. A section on deviations from Design Guidelines is provided in Section 6.2.1 regarding potenial use of multiple barrels with invert to invert connections.
29	2	As previously commented, the use of a 300mm deep x 1800mm wide box culvert is not acceptable. It is apparent that this is being proposed to provide clearance from the existing servicing on Fortune St. – but it is not acceptable. Revise the design to an appropriate storm sewer cross-section and provide the minimum clearance required between existing storm and sanitary services. Provide the confirmed invert on the existing sanitary sewer at the crossing (drawings indicate this is still to be confirmed). In the alternative, relocate the outlet further upstream and revise the pond design accordingly. Note: this is not a detailed design issue as it brings into question whether the proposed outlet location is feasible. There is also minimal cover where the outfall sewer crosses the Moore Drain (approx. 0.30m based upon a drain invert of approx. 93.38m from LiDAR data).	Two options exist to resolve the conflict at Fortune Street. One, provide a multiple barrel outlet versus a custom box culvert. Two, lower the existing sanitary service on Fortune Street and connect to new sanitary sewer on Martin Street. Moore Drain crossing to be reviewed with Drainage Engineer.		**Comment not addressed**: Please provide the details of an acceptable design solution so that it may be reviewed: it appears that a lowering of the existing sanitary sewer on Fortune St. may be unavoidable and address the lack of cover outfall under Moore Drain.	Solutions discussed at Sept 12/13 meeting. Minimal separation below the Moore ditch is still shown (with a 750mm pipe). A section on deviations from the Design Guidelines is provided in Section 6.2.1 for potential invert to invert connections using multiple pipes to provide clearance from the Moore ditch. In addition, the sanitary sewer conflict has been rectified with the new sanitary servicing solution shown in Drawing 7.
30	3	The extensive use of box culverts (close to 1400m in total) is inconsistent with Sewer Design Guideline. Operations staff have confirmed that in lieu of box culverts, multiple round conduits that provide equivalent flow capacity are required.	Duly noted, multiple barrels will be investigated at detailed design.		Comment addressed.	
31	4	Per previous comments, the report notes (p.21) that the Jock River Estates drainage (MAT-E) is proposed to be conveyed north of Ottawa Street through a new culvert to the redesigned Moore Tributary channel and then to the Van Gaal/Arbuckle drain. It is not clear how this is to be achieved with a culvert. Please clarify. The report and drawings also note that this drainage will be picked up and conveyed to Pond 2. Whatever the case, consistency is required between drawings and documentation – please revise as required.	Page 21 outlines a high level review of servicing strategies. It is proposed to collect and convey the Jock River Estates drainage through Pond 2.		Please clearly document in the report and on the drawings how this drainage is to be dealt with.	To be included with the RVCA submission for the Mattamy draft plan. Flows will be conveyed through Pond 2. Discussion on the proposed sequencing of closure of the existing ditch is included in updated text in Section 4.1.3 "Option 3".
32	5	Minimum cleansing velocities (0.80 m/s at the 5 year design flow) have not been met for all storm sewers: 600 mm diameter: 244 m (4 pipe sections); 675 mm Diameter: 264 m (6 pipe sections). Please revise the design accordingly.	City of Ottawa Guidelines Section 6.1.2.1 "Storm sewers must be designed to provide a minimum velocity o 0.80m/s when flowing full."		**Comment not addressed**: Please revise design accordingly to ensure that mimimum velocities are achieved. .	At the macro design level there are still some pipe segments that are shown marginally below the 0.8m/s cleansing velocity. These select pipe segments will be refined to meet the 0.8m/s at the detailed design stage.
33	6	Per previous comments: Given that the pond outlet crosses under (not into) the Moore Drain to connect to the Van Gaal, it is not apparent how cool "baseflow" can continue to be directed to the top of section 2 of the Moore. Is this still a requirement? If yes, how is it to be addressed?	Addressed with RVCA. Pond 1 to involve a bottom drain outlet to provide cool base flow to the Van Gaal.		Please provide the proposed concept on the drawing for review.	Coordinated with RVCA, see Section 7.1.1 for descriptions/discussion. Outlet shown on Drawing 3 and Figure 20.
34	7	Regarding the Pond 2 outfall along Ottawa St. – it appears there may be potential for conflicts between existing sanitary services connections toward the end of Ottawa St.	The proposed pond 2 outlet is contemplated to be on the south side of Ottawa Street, therefore the existing homes on the south side of the street would be in potential conflict. However, this is limited to one home where the existing sanitary sewer is only slightly above the proposed storm, indicating that there is no conflict, only the possibility that the service may be close.	Drawing 6	Given confirmation that a conflict(s) may exist, clearly identify this on the drawing and indicate it will be further confirmed/addressed during detailed design.	The potential conflict is noted on Drawing 6 for "Pond Storm Outfall 2" and that it is to be reviewed at the detailed design stage.
	F. Sanitary HGL					
35	1	Comments will be provided pending resolution of the foundation drainage servicing.			Foundation drainage servicing not yet resolved.	
City Comments Dated December 14, 2012 - Review of Updated Assessment of Subsurface Drainage and Analysis of 100 Year Flood Event						
From: Darlene Conway To: Cheryl McWilliams						
	Detailed Comments:					

36	1	The Golder memo of October 3, 2012 documents assumptions, conclusions and limitations but does not appear to provide specific recommendations with respect to engineering design. What are the recommendations related to the design of a foundation drainage system that can be offered from this modeling exercise? It would be of assistance for the author(s) of the memo to compare the utility of this modeling exercise to other technical work used to support the engineering design of this proposed development, for example, the recommendations proceeding from the Golder geotechnical memo of June 27, 2011 or the hydrotechnical modeling completed by JFSA (Appendix I in the DSEL November 2012 report).	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013	Defer to detailed comments provided by others (Dillon, Michel Kearney).	
37	2	From the Golder October 2012 memo (emphasis added): In order to establish an initial groundwater condition, the model recharge was adjusted until the simulated groundwater elevation was directly beneath the foundation drains (which occurred at a recharge rate of 90 mm/yr). Using this initial condition, the 100 year storm was simulated transiently over a 24 hour period, during which time groundwater elevations in the storm sewer trench and service stub were increased from 92.98 masl to 94.11masl. Water levels were assumed to increase instantaneously at the onset of the storm. To simulate the additional impact of the 100 year storm occurring concurrently with the spring freshet, the recharge was increased to 2000 mm/year during the same 24 hour period. This value of recharge resulted in an average head throughout the model domain that approximated the 100 year storm water level. The magnitude and duration of the spring freshet used for the modelling were assumed values; however, the selected parameters are considered to be conservative. The 100 year spring event used to simulate the regulatory (100 year) flood level on the Jock River is a 10day snowmelt +rainfall event having a volume of approximately 270mm. The 100 year 24 hour storm has a volume of approximately 107mm. Pond operating levels under this condition are also higher than in the receivers. Further clarification regarding the contention that the parameters used to model the spring 100year condition are conservative would be helpful.	The 100 year storm event simulation increased the water table to the elevation of the foundation drains, increased the water elevation in the pond and in the service trenches to the designed operating level (94.11 masl) and added additional infiltration (2000 mm/yr recharge) due to rainfall. Higher rates of recharge have been included in the revised model, to be described in a separate memorandum	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013	**Comment not addressed:** The analysis does not consider the impact of the extended duration of the event that generates the regulatory flood level (94.11m) on the Jock River which is a *10-day* rain on snow event - not a 24 hour event. Further, under the regulatory (spring, 10-day) event, the levels in the ponds are higher than 94.11m (the flood level in the river/drain). To further clarify: the regulatory event that generates the 100 year flood level of 94.11m is an extended event, resulting in days, not hours, of elevated flood levels that would presumably require sump pumps to operate for days in succession.	
38	3	The attached tables summarize pond operating levels and Jock River flood levels for various conditions and frequencies. Separate from the issue of lowering the existing water table, the numbers in this table indicate that, based upon the USF elevations proposed, sump pumps could be expected to operate very frequently and at length, considering the fractured bedrock conditions in the southern portion of the site. Every USF is located below the 2 year flood elevation (section 20686) on the Jock River. Has the worst case scenario been considered in this exercise? What is to prevent the need for continuous operation of sump pumps given the 2 year flood level in the Jock River and the fractured bedrock conditions?	DSEL / Golder reviewed site grading in the area adjacent to the River. If USF's are situated in rock their elevations have been increased to be above the 100-year water level in the Jock River.		Please provide an updated grading plan for review that confirms the response provided.	
39	4	From the Golder October 2012 memo: A service stub was specified using a constant head boundary from the storm sewer to towards the house. The stub was terminated at a distance of 3 m from the house. A 2 m width was assumed for the stub trench. The constant head boundary was assigned at the same elevation as the storm sewer trench; The modeling also assumed that the granular material within the service trenches will not directly connect to foundation drains. Inspection during construction, to ensure implementation of this design, is recommended. Given the above modeling assumptions, is the memo recommending that sufficient protection would be provided during the 100 year event (and per the tables above events as frequent as the 5 year) by a width of some 2m of non-granular backfill material (assuming 1m of granular backfill adjacent to the foundation wall) that is further dependent on stringent inspection during the backfilling process?	Yes, the technical memorandum is recommending that sufficient protection would be provided.		Based upon typical construction practices and the extent of site supervision available, it is not apparent that this is an acceptable approach on which to base the protection of basements from flooding due to backwatering of the service trenches.	
40	5	Can examples from other municipalities be provided where all basements have been built well below the existing water table elevation on the basis of a groundwater modeling exercise with similar assumptions, along with a list of contacts such that the City may follow up on the performance of any such examples?	Please note that not all basements are situated below the existing high groundwater table. Where basements are lower than the recorded high groundwater table, it should be noted that the proposed USF's are higher than the adjacent existing properties on Fortune and Queen Charlotte where it is expected that pre-development conditions were similar.		**Comment not addressed**: No examples from other municipalities where all (or a significant number of) basements have been built well below the existing water table elevation on the basis of a groundwater modeling exercise with similar assumptions have been provided. To clarify, this request was referring to relatively recent developments from municipalities with relatively current/comparable design guidelines. If such examples cannot be pointed to, please indicate this.	
City Comments Dated December 14, 2012 - Sump Pump Strategy and Numerical Modeling						
From: Michel Kearney To: Darlene Conway						
	Fundamental Considerations					
41	1	These comments are to be read in conjunction with the peer review performed by Dillon Consulting, dated November 29, 2012. There has been an effort to avoid, as much as possible, the duplication of comments, but where deemed appropriate a comment made by Dillon may be reiterated, either for emphasis or for contextualization.	No comment.			

42	2	The use of sump pumps, as proposed, is a deviation from the Ottawa Sewer Design Guidelines. DSEL(285)3 refers to §§ 3.2.2, 5.7.3 and 5.9 of the Guidelines; however, §3.2.2 refers to infill developments and requires that sump pumps drain to the surface, §5.7.3 refers to sewers with limited capacity constraints and recommends discharge of the sump pump to the ground surface (this section mentions slab on grade as a means to eliminate the need for a sump pump), and § 5.9 refers to ditch pipes with storm sewer systems, and even though it does not refer to where the sump pump should discharge, in view of the shallowness of the ditch pipes and considering the other references in the Guidelines regarding sump pump discharge, it is understood that discharge is to be directed to the surface in this instance as well.	Is the City suggesting that the Sump pumps could be approved if they were directed to the surface? In our previous discussions regarding sump pump, the City had expressed concerns regarding this arrangement due to the possibility of future home owners creating illegal connections. Please advise.		**Comment misinterpreted:** The comment from the City was addressing the assertion by DSEL that the use of sump pumps in this development falls within the City of Ottawa Design Guidelines. DSEL quoted §§3.2.2, 5.7.3 and 5.9; however, as pointed out by the City, these sections of the Guidelines all refer to circumstances other than those found in the Richmond Western Development Lands (i.e., is the discharge of sump pumps into storm sewers). The possibility of sump pumps discharging to the surface of the ground (as an option for the subject lands) is not something that was being discussed in the comment.	
43	3	DSEL continues to use a matrix evaluation procedure, even though it has been pointed out that the use of a weighted matrix is not appropriate at the subdivision level (see DSEL(14)). The use of a matrix may be justified for the first three options only, i.e. not the sump pump option (see DSEL 308, 309), as sump pumps can only be considered once it has been determined that other options are not viable.	Noted, it was determined by the client that the first three options are not financially viable. As described in Section 5.0 of the report, Option 1 exceeds grade raise recommendation throughout the subdivision.		**Comment not addressed:** There are other components to viability besides financial considerations. We reiterate that a matrix evaluation procedure is not appropriate at the subdivision level.	
44	4	In view of the lower level of service provided by sump pumps, compared to the other three options, it has not been justified why future residents should be subjected to the lower level of service provided by sump pumps. The level of service must be addressed on its own merit and not based on a decision matrix.	Please note that Section 5.0 of the SWM report describes each option in its own light. The matrix provides a comparison table for the options and was not the sole vehicle for arriving at the sump pump conclusion.		**Comment not addressed:** As stated by the City, the matrix presented by DSEL is inappropriate as a comparison tool, even if the matrix was not the sole vehicle for arriving at the sump pump conclusion.	
45	5	Although most of Richmond is currently on sump pumps, the vast majority of properties are historical and have sump pumps that discharge to the surface. There is a smaller and more recent subset of Richmond properties where sump pumps discharge into a storm sewer, but these developments are not as dense as the proposed Western Development Lands, and our review indicates, to the best of our knowledge, that the foundations were kept above the pre-development water table condition.	No comment.		No further comment required	
46	6	The City has never considered approving the Western Development Lands on the basis of footing elevations that are lower than the existing water table4. In fact, it has been stated numerous times that footing elevations have to be above the water table (DSEL 15, 270, 271, 273, 274, 276, 290, 294, 314, 316). Although it is recognized that the proposal before the City is for the lowering of the existing water table, based on a predictive assessment of the effect of the storm sewer network and service stubs, this proposed solution is unprecedented at the City. Rather, it has been the practice of the City to only approve developments on sump pumps where it can be demonstrated that the foundation is above the pre-development high water table5. That being the case, it would have been prudent to liaise with the City prior to commencing a modelling exercise that looks at lowering the water table.	The City's reference to the determination of the high water table elevation by the Ottawa Septic office demonstrates that the high water table is typically determined after draft approval, and after other servicing (roads and storm drains) have been constructed. It also demonstrates that the City is typically satisfied with an estimation of the water table elevation, based on one-time observations. The City suggests that, "it would have been prudent to liaise with the City prior to commencing a modelling exercise that looks at modelling the water table." However, Golder participated in a meeting with the City, on February 17, 2010, and in a memorandum by the City dated April 29, 2010, the City states, "It was agreed at the meeting [of February 17th] that more fieldwork would be required in order to obtain better quality information on water table elevations and soil hydraulic conductivities. In conjunction with the fieldwork, Golder would perform a numerical modeling exercise in order to investigate some theoretical scenarios, which could later be fine-tuned with the new field information." It is our recollection that it was the City, not Golder, that suggested the use of groundwater modelling in order to assess the sump pump issue.		**Comment not addressed:** The City's reference to the Ottawa Septic System Office (OSSO) was in the context of an individual lot in the rural area, not in the case of a subdivision, where the City requires that the water table be determined as part of the design of the subdivision. Also the estimate of the water table by the OSSO is not a "one-time observation," but is rather based on the experience of the inspector in that particular area. The model referenced by Golder is not the same model as the one currently proposed. It was a much simpler model, and it was to be used to estimate a parameter under review at the time, which was dubbed by Golder "time to flood." The model was not suggested by the City and it had a fundamental flaw that rendered it unusable for the purpose intended by Golder at the time. When Golder recognized that the sump pit would be a direct conduit for groundwater accumulating around the foundation, the option of using the granulars under the floor slabs as storage was abandoned. This preliminary and abandoned model is not relevant to the current discussion, where Golder is proposing to use a model to predict the lowering of the water table for the purpose of setting the foundations at a lower elevation (than the existing monitored water table).	
47	7	The relationship between basement elevations and the water table is of crucial importance for future homeowners within the subdivision, and it is evident that a high comfort level needs to be provided against the possibility of basement flooding. There is considerable doubt whether such a comfort level can be achieved through a predictive groundwater model. There is ample literature discussing the role of models in decision-making. It is widely recognized that groundwater numerical models have an important role to play in decision-making, as analytical solutions are available for only the simplest of problems; however, great caution needs to be exercised when relying on model predictions. In a recent issue of Ground Water7, Hunt and Zheng state that "models can never be considered crystal balls predicting the future, no matter how well constructed". They go on to say that this does not mean that models do not have utility today, but that it must be kept in mind that models have a "societal decision-making context", and that models are "better thought as heuristic8 science-based tools to assess what has happened, what is in the realm of possible for the future, and how uncertain the conjectures are". Golder is very familiar with these concepts, but they are nevertheless presented here for the non-specialist who is reading these comments and seeking understanding.	Golder has the necessary expertise and experience regarding the appropriate application of groundwater models. Golder has completed many projects involving groundwater modelling for many clients, including the City of Ottawa. In Golder's opinion, the models we have developed for this project reasonably approximate the real systems that they were developed to simulate, to the extent that they provide reasonable predictions of future groundwater levels. With regards to third party reliance, Golder's professional work produce was produced for the use of our client for the stated purpose only, as we have a contractual relationship with our client, only. Golder would not provide reliance to any third party in the absence of a contractual relationship. It is important to note that contracts and limitations statements address specific non-technical issues that are not related to quality of work or professional standards. The City should not confuse issues of a contractual and technical nature. We recommend that the City investigate the degree to which the City can rely on the services provided by the consultant it has retained in relation to this project (Dillon Consulting).		**Comment not addressed:** Godler's expertise is not being questioned by the City. The comment provided by the City was to highlight for the non-specialist the normal use of models, and that they have to be used with great caution when making predictions. The City does not agree with Golder that setting foundations based on a modeling exercise is appropriate. Dillon, in their July 11, 2013 review letter have also expressed a similar concern. In addition to this concern, the City is also concerned that it is excluded from the parties that can rely on the Golder report (see Dillon on this matter also).	

		Golder has properly outlined the limitations of the model, stating that "[h]ydrogeological investigations and groundwater modelling are dynamic and inexact sciences" and that "groundwater systems are complicated beyond human capability to evaluate them comprehensively in detail". Golder also seems to acknowledge the heuristic use of the model when they state that "the behaviour of a valid groundwater model reasonably approximates that of the real system". It is however a large step to go beyond system behaviour to then predict future water table elevations with sufficient confidence to set basement elevations. Recognizing these limitations, Golder states that third parties9 can only rely on the model at their own risk. Dillon has pointed out that "[g]iven the significant limitations listed in the report, it is unclear if the assessment approach used is an adequate method to predict the impacts caused by the development." The City shares this concern. There is a need to couch the model within the "societal decision-making context", which in this case we understand to consist, at least partly, in protecting future homeowners who would have to deal with potential flooding problems should the model predictions be incorrect. It must therefore be recognized that the utility of a modelling exercise is limited when trying to establish basement elevations.				
	Specific Comments on the Golder Technical Memorandum (TM)					
48	8	On page 1 the TM states that its contents represent a summary of the results; however, as pointed out by Dillon, additional documentation is required for a complete review. This review is therefore preliminary.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013	**Comment partially addressed:** Additional information has been provided by Golder, but some information is still missing (see Dillon review letter, July 13, 2013).	
49	9	There is a width of 2 m assumed for the service trench (page 2); however, this seems to be significantly wider than typical. Is a 2 m trench required in the model? (Also see Comment 18 below).	A 2 metre trench is not necessary. The width of the trench in the model has been revised to 1 m. The revised model is described under separate cover.		**Comment partially addressed:** The width of the service trench was reduced to 1 m, but no explanation was provided (as to the need for a specified trench width).	
50	10	On page 3 there is an assumption that initial groundwater drainage will occur through the granular backfill material within service trenches; however, sewer trench bedding consists of Granular "A"10, which is a fairly well-graded material with substantial amounts of fines, and hence does not appear to exhibit the drainage characteristics assigned in the model. This premise of the model therefore needs to be reviewed. The hydraulic conductivity of the granular bedding material must be taken into account, and the longitudinal flow component of the dewatering regime—i.e. along the storm sewer bedding to the outlet—needs to be quantified.	Granular A was not specified in the Golder memo. We would recommend Granular O, which is Granular A with the fines removed, and thus is free draining.			
51	11	On page 3 weathered bedrock is assigned a hydraulic conductivity of 5x10-5 m/s, which appears to be an average value based on the measured values; however, since the fractured rock areas may be the worst case areas, it is important to assess the sensitivity of the model to the higher hydraulic conductivity (see also Comment 13 below).	The hydraulic conductivity of the upper bedrock was assigned a value that was the average of the measured values. This value is considered reasonable, and, based on the results of the subsurface investigation, is representative of the average conditions across the site		**Comment not addressed:** It is standard practice to provide a sensitivity assessment (see also Dillon's July 11, 2013 review letter).	
52	12	On page 3 an anisotropic ratio of 10:1 is applied to all layers; however, justification should be provided for assigning this ratio to the overburden. The sensitivity of the model to isotropic conditions in the overburden needs to be investigated.	The anisotropy of the overburden in the model has been revised. The revised model is described under separate cover.		Comment addressed.	
53	13	On page 3 the worst case scenario is said to occur at the location shown on Figure 1; however, it would appear that a worst case may be at a location near the Jock River, where excavation is proposed in fractured bedrock, with fractures possibly connected to the river and where house excavations may open up more fractures. The entire area south of Ottawa Street therefore needs to be investigated separately. More field investigations may be required in order to better assess the connectivity between the fractures in the bedrock and the Jock River. All the foundations are below the 2-year flood level in the Jock River and many of these foundations are likely below even more frequent events. In houses constructed into the bedrock the sump pumps may not be able to keep up with the possible high flows from some parts of the fracture network. A work plan should be devised as to how this issue can be investigated.	DSEL / Golder reviewed site grading in the area adjacent to the River. USF's have been increased to either be situated above the 100-year water level in the Jock River or the rock elevation, which ever is lower.		**Comment partially addressed:** Please provide the revised Grading Plan.	
54	14	The second bullet on page 4 states that the drain boundary elevations were specified at an elevation of 0.05 m above the storm sewer invert elevation to represent the potential flowing water depth in the sewer; however, the water in the sewer is not connected to the water in the trenches. Clarification is required, especially in view that the sixth bullet on the same page seems to indicate the opposite.	As stated by the City, water in the sewer is not to be connected to water in the trench bedding. This is clarified in the memorandum prepared by Golder regarding the revised model.		Comment addressed.	

55	15	On page 5 there is a rationale for a 24-hour, 100-year storm; however, this rationale seems to miss the issue raised by the City. It appears that Golder is attempting to address the issue represented in DSEL(184). The City was raising the concern about the effect of the regulatory flood event, and also other events that might impact the basements, on houses located to the south of Ottawa Street. As stated above, there may be a hydraulic connection between the river and the fractured bedrock and the concern is that sump pumps may not keep up where basements are located below the water table and/or below the regulatory flood event (all underside of footing elevations south of Ottawa Street are located well below the regulatory flood elevation). The hydrograph for the regulatory flood will peak and recede relatively slowly. Assessing the effect of a 24-hour storm at the north portion of the subdivision does not address the City's concerns expressed in DSEL(184). See also Comment 13 (above) on this issue.	DSEL / Golder reviewed site grading in the area adjacent to the River. USF's have been increased to either be situated above the 100-year water level in the Jock River or the rock elevation, which ever is lower.		**Comment partially addressed:** Please provide the revised Grading Plan.	
56	16	Page 5 states that it will take 475 days for the water table to lower to a level where most basements are at or higher than the predicted water table. It should be specified whether it is intended to wait 475 days before the construction of the first house, after it has been confirmed in the field that the water table lowering has been achieved. Also, in view of Comment 9 (above), the lowering of the water table may not be achieved if the trenches are not free draining as is assumed in the model.	The modelling results provide general guidance regarding the timing of the lowering of the water table, under average conditions. Many factors may increase or decrease the actual time to achieve the required water table lowering. As such, field confirmation of the water table elevation during construction is recommended. Active dewatering during construction will likely be necessary (as is typical) for construction of the buried services, so it is likely that a combination of active and passive dewatering methods will achieve the water table lowering necessary to enable house construction.		**Comment misinterpreted:** The City was inquiring whether the developer will be waiting 475 days (now 400 days) to ensure that the water table has been lowered (we assume that this would be confirmed through monitoring). The City was not inquiring about the construction methodology.	
57	17	On page 7 there is a predicted peak inflow into the sump pit of 1.07 m3/house/day based on the model. It would be useful to look at actual studies to see if this value is in fact an upper range for similar conditions. For example, there is a study by the Ministry of the Environment12, where rain and pumping volumes were measured for a number of houses, and where flows exceeding 15 m3 were measured from a single house foundation drain over a period of approximately 25 hours.	Noted. Golder's analysis was specific to the study area.		**Comment not addressed:** The City is not necessarily asking Golder to provide data from studies in other areas; however, the City is suggesting that since the proposed solution is of a unique nature it would be useful to make use of data from studies where flows from foundations have been measured. The City provided a reference for one such study.	
58	18	Page 6 mentions that inspection is recommended in order to ensure that the service trench granular material is not connected to the house; however, this is not an area that is currently closely inspected. There may not be a mechanism to ensure that this gets inspected and therefore this particular inspection point should not be assumed as part of the solution. Under the proposed scheme, there is in effect only approximately 2 m of soil between the granular in the house service trench and the free draining material along the perimeter of the foundation (typically around 1 m).	Clay seals are commonly designed for City sewers and watermains, and presumably the City carries out inspections to ensure that they are installed as designed. Our recommendation in this regard is that inspections be conducted at all service stubs, to ensure that the seals are installed as per the final designs.		**Comment not addressed:** The clay seals that Golder is referring to are normally in the main sewer trench, where continuous inspection is provided by the developer's consultant and supplemented by inspection by the City. The service trenches are not inspected to this degree.	
59	19	Although the model is based on measured hydraulic conductivities, there is still a need to bracket parameters within possible ranges and run the model for several scenarios in order to determine the sensitivity of the model to variable inputs.	Golder has revised the groundwater model in response to questions and comments from the City's consultant. Description of the revised model and modelling results are presented under separate cover. In our opinion, the model is appropriately representative of the site conditions, based on site specific data, and as such, the model results present our best estimates of the groundwater conditions that will occur, in accordance with the scenarios as described. Further refinement or adjusting of model parameters is unlikely to provide better, or more accurate results.		**Comment not addressed:** It is standard practice to provide a sensitivity assessment (see also Dillon's July 11, 2013 review letter).	
60	20	The Technical Memo by Hatch Mott MacDonald refers to an attached sketch showing a check valve, an isolation valve and a union; however the supporting drawing provided to the City (Figure 18, Dec 2012, "SUMP PUMP – DETAIL") does not show any valve(s). Notwithstanding the sketch, the memo states that the "[u]se of a backwater valve ahead of the 100 mm Y, is not considered necessary as potential backflow would be inhibited by the rise of the pump discharge coupled with the sump pump check valve", and further adds that "[g]iven that the pump discharge will be from a 40 or 50 mm pipe to a 100 mm pipe, there will be free air surface at the junction of the Y connection, therefore siphoning will not be an issue"; however, this would only happen if the 100 mm storm service was free draining, which will not be the case under 100-year conditions (and more frequent events as well), since the sewer in the street and the entire length of the storm service will be surcharge, and hence under a pressurized condition—i.e. there will be no air in the service connection. That being the case, the premise of relying on a check valve to protect against siphoning must be properly vetted and a case made for review by the City.	Duley noted, however there will be air in the outlet line at the top of the 'U' where it is exhausted above. Note that this is a typical arrangement employed throughout the GTA.		**Comment not addressed:** How will air enter the pipe at the top of the "U"? If the check valve in the sump pit (assuming that there is a check valve at this location) malfunctions, then the siphoning action will begin almost immediately after the sump pump shuts off. There is no time for air to come in (presuming that some kind of air entry device is designed—see question above). Please provide the details of the GTA arrangement (ensure that the level of detail is of "shop-drawing" level).	DSEL to provide shop drawings of the proposed arrangment. Refer to Goulbourn standard for the proposed sump pump arrangement.

61	21	In view of the assumptions used, the uncertainties inherent in the modelling exercise and the limitations in the TM, the City does not have a comfort level that the water table will lower as predicted. Also, it would appear that a significant number of houses will not have any freeboard between the predicted water table and the foundation; in fact, some houses will be in the water table even after 475 days (page 5 of the TM). Notwithstanding the comments provided on the Golder model, it is anticipated that given the inherent risks with sump pumps in this particular setting, the requirements of the deviations/exceptions Section (§ 1.3) of the Ottawa Sewer Design Guidelines will not be met using this method of foundation drainage13. Consideration may have to be given to constructing homes without basements, or including a foundation drain collector system, should increased filling of the property be deemed not viable.	Please note that the sump pumps are expected to operate normally during the spring time where the ground water level is highest. The long term model predicts that the high ground water level will be lower than existing due to the presence of the development.		**Comment not addressed:** The City does not concur with Golder that a groundwater model can used as the basis for setting foundation elevations where the current water table is above the underside of footings.	
Third Party Review Comments Dated November 29, 2012 - Review of Oct 3, 2012 Technical Memorandum on Assessment of Drainage and Analysis of 100 year flood event.						
From: Dillon Consulting To: Darlene Conway						
62	1	Dillon is only in a position to provide limited comments at this time on the modeling work completed as the level of documentation is currently insufficient to support a full review. Additional documentation required to support a more complete review should include the following:	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013	For comments on proponent responses provided, please refer to: Village of Richmond, Western Development Lands, Review of June 5, 2013 Technical Memorandum on Assessment of Drainage and Analysis of 100 Year Storm Event, Dillon Consulting, July 11, 2013	
		• A figure showing the boundary conditions including assigned head elevations.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
		• A figure showing hydraulic conductivity zones in both plan and section.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
		• A figure showing actual water level distribution (in plan) and a comparison figure of modelled water level distribution.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
		• A figure showing the predicted (modelled) water level distribution in plan after water levels have decreased after development and comparison to foundation elevations.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
		• A figure showing the amount of drawdown (decrease in water levels) predevelopment versus post-development.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
		• Similar documentation for the 100 Year Storm Event Model.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
		• An assessment of the sensitivity in the model to assumed and variable input parameters including hydraulic conductivity assumptions and recharge rates.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		

63	2	Overall, further description should be provided to allow a better understanding of the methodology, assumptions and results of the modeling exercise. This should include supporting information on the basis and justification for the approach and assumptions applied. Given the significant limitations listed in the report, it is unclear if the assessment approach used is an adequate method to predict the impacts caused by the development. Further description and justification of the overall approach should be provided, specifically with respect to the level of reliance that should be placed upon the model predictions.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
64	3	The use of constant head boundaries to simulate the storm sewer infrastructure requires further description and justification. Constant head boundaries assume that the sewer trench is completely free-draining. The basis of this assumption should be discussed in greater detail. Also it is unclear if the elevations used as the basis of the constant heads take into account the rise in the sewer invert elevation away from the storm ponds.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
65	4	The 100 year storm event simulation methodology is unclear and would be better supported by a graphic. What is the basis for assuming a 24 hour period for the storm event effect on water levels? Likewise what is the basis for the assumption that water levels will instantaneously decrease after 24 hours?	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
66	5	Similarly, it is assumed that the constant heads simulating the foundation drains remained active during the 100 year storm event simulation. This should be confirmed and the requirements for ensuring that the storm sewer remains a viable outlet discussed.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
67	6	More explanation should be provided to support the conclusion "… would not adversely effect baseflow to adjacent water courses, …" Are the expected flow rates greater than 50,000 L/day and is a Permit to Take Water required from the MOE?	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
68	7	Other potential complicating factors should be considered and commented upon, whether implicitly as part of the modeling exercise or through discussion of their potential effects. This should include: a. the presence of upward gradients in the bedrock at some locations; b. variable hydraulic conductivity in both the overburden and bedrock, as observed in the estimates provided in the TM; c. whether underside of footing (USF) elevations at some locations may encroach upon or be in contact with the more conductive bedrock materials (either within or beyond the modeled domain).	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
69	8	As noted previously, further discussion should be provided with respect to the sensitivity of the model results to variations in model assumptions and a discussion of specific sources of uncertainty should be included with the overall conclusions of the assessment.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
70	9	Potential settlement issues associated with the water table lowering should be explicitly addressed.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		
71	10	In the limitations section, the City of Ottawa and other review agencies should be included as third parties that can rely on the Golder report.	Addressed by Golder in updated technical memorandum.	Golder Associates - Updated Assessment of Subsurface Drainage and Analysis of 100-year storm event, Proposed Village of Richmond Development, June 5, 2013		

1.3 Public and Agency Consultation

Public consultation is an integral part of the preparation of Mattamy Homes Official Plan Amendment and supporting studies including the Stormwater Management and Drainage Plan exercise. A transparent process in which members of the public, community groups, residents, City of Ottawa, public agencies and other stakeholders can express their issues and concerns and obtain timely information on the study as it progresses are key components of this study. Regulatory public meetings are also a requirement of the Class Environmental Assessment Process which is being followed, in spirit, for the Stormwater Management and Drainage Plan exercise. The following consultation points are outlined under the Class EA Process for Phases 1 and 2:

Phase 1 – Problem or Opportunity
- Discretionary public consultation to review problem or opportunity

Phase 2 – Alternative Solutions
- Mandatory public and agency consultation on the identified problem or opportunity and identified and evaluated alternative solutions to the problem

The City of Ottawa initiated the Richmond Village Community Design Plan (CDP) process in March of 2008. Through the Ward Councillor, a Steering Committee made up of representatives from the Village was established to facilitate a community based approach to prepare and develop the Community Design Plan for the Village of Richmond. The Steering Committee is comprised of residents, farmers, the Richmond Village Association, business people, and individuals/companies with a development interest. The community based Steering Committee allows the Richmond Village Community Design Plan to be developed by the community, for the community. Mattamy Homes is a member of the Richmond Village Steering Committee.

A collaborative public consultation approach has been undertaken that informs both Mattamy Homes Official Plan Amendment process and the Richmond Village CDP. A number of public events have taken place either lead by the City, the Richmond Village Steering Committee or Mattamy Homes that have assisted with the preparation of the technical documents supporting Mattamy's OPA as well as the preparation of the Village CDP. These public consultation events are briefly described below with supporting documentation in ***Appendix B***.

April 12th Public Open House

The City of Ottawa held a public open house at the Richmond Public School from 9:00 a.m. to Noon on Saturday, April 12, 2008. The purpose of the open house was to introduce the commencement of the Richmond Village Community Design Plan process. As well, a number of information stations were displayed with City staff on hand to answer questions and share information related to: the existing village plan, heritage buildings, natural environment, groundwater and servicing. Participants were asked to identify “Places We Like Most” and “Places We Like Least” in the Village of Richmond through red (least) and green (most) dots placed on a mounted aerial photo of the Village. Participants were also asked to place yellow dots on the aerial photo to identify traffic problems and pedestrian safety “hot spots” in Richmond. This event was well attended with approximately 125 people participating in the event.

April 19th Visioning Workshop

The City of Ottawa and Richmond Village Steering Committee hosted a visioning workshop to bring together the residents and stakeholders in Richmond to see what they wanted the Village to be in 20 years. Participants were also asked to identify “greatest opportunities” and “greatest challenges” to meet the vision for the Village. The workshop was held at the Richmond Public School from 9:00 a.m. to Noon. The “Dot-mocracy” map was on display and those who had not attended the April 12th session were asked to place their red, green and yellow dots on the aerial photo of the Village. The workshop had discussion tables set up for the following topic areas: Village/Heritage Character; Future Development; Transportation and Pathways; Environment, Drainage and Floodplains; Servicing and Groundwater; Building Richmond as a sustainable community; and Recreation, Community Facilities and Open Space. Each participant could participate in 5 topic areas within the time period established for the break out sessions. City staff and Richmond Village Steering Committee facilitated the discussions at each of the topic area tables. Approximately 75 people attended and participated in the visioning workshop.

Community Visioning Principles

Based on the feedback from the April community visioning exercise, City staff and the Richmond Village Steering Committee drafted a primarily community vision for the Richmond Village which comprised of six main community principles:

- Create a Livable and Sustainable Community
- Protect and Enhance Richmond’s Historic Village Character
- Protect the Natural Environment and Incorporate Constraints in the Plan
- Expand and Maintain Transportation Infrastructure
- Create and Protect Open Space, Recreation and Community Services
- Ensure Sustainability of Servicing (Groundwater, Wastewater and Stormwater Systems)

On June 4, 2008, the Richmond Village Steering Committee hosted a Strategic Direction Workshop at the Richmond Library from 7:00 p.m. to 9:00 p.m. The public was invited to participate in a small working group session to provide input on the draft community visioning principles on: Village Character and Development; Environment, Recreation and Sustainability; Transportation and Facilities; and Servicing and Groundwater. Based on the input received, the Village of Richmond Community Vision Workbook was prepared and circulated for comment to all residents in the Village in July 2008. A total of 2461 booklet were distributed directly to residents in the Village and additional copies were available at the Richmond Library and the Valu-Mart. A total of 246 booklet responses were returned to the City. More than two-thirds of respondents agreed with the community visioning principles. The response suggests that the principles are in line with the areas that Richmond residents feel important in the planning process. The Richmond Community Visioning Principles are contained in ***Appendix B***.

September 2008 Design Workshop

The Richmond Village Steering Committee endorsed Looney Ricks Kiss (LRK), architects and community planners, to undertake a four-day design workshop in the community to define the Village Core based on the established visioning principles. The Councillor along with the Richmond Village Steering Committee invited residents to attend the four-day workshop held from September 22 to September 25 at the vacant storefront situated at 3480 McBean Street. The workshop was set up as a drop-in centre – day or night – to participate in defining and designing the village core plan.

In preparation for this workshop, LRK conducted site visit investigations to 16 villages in Eastern Ontario called benchmarking. This exercise involved measuring and documenting local built precedents and historic and contemporary examples that could be used as the base line for defining architectural character as well as urban design and landscape patterns. This process documented landscape/streetscape treatments and patterns and architectural buildings that could provide examples to draw from when preparing the Richmond Village plan.

As well, Mattamy Homes consultant team set up and manned display boards in the store on the existing conditions information related to planning, design, natural environment, stormwater management, transportation, water, groundwater and sanitary servicing. The public visiting the workshop could view the display information and asked questions to the consultant on existing information and the overall process being undertaken to support Mattamy Homes Official Plan Amendment.

This workshop was very well attended with over 250 individuals attending at least one of the four days, with many persons making repeated visits. A summary of the four day design workshop is provided below:

Monday, September 22

Focus groups were set up for the morning inviting representatives from the businesses, real estate agents, recreational groups, residents along McBean and Perth, community groups, schools, churches, and farmers. The public was invited to participate in a benchmarking tour of other Eastern Ontario villages in the afternoon to investigate comparable communities and the design examples that may be applicable to Richmond. In the evening, the Councillor sponsored a barbeque which attracted around 100 residents. Participants were asked to participate in a community design survey developed by LRK. Participants were asked through a PowerPoint presentation to vote for the image that best represents their vision for Richmond associated with: residential building design, commercial building design, parks and open space, Perth Street, McBean Street, river corridor, local streetscape. The survey was available during all four days so that all attendees could participate. A total of 120 surveys were completed.

Tuesday, September 23

On Tuesday morning, residents and stakeholders were invited to take part in a walking tour of the village core. Participants were encouraged to talk about McBean Street, Perth Street, the Jock River and the surrounding area and how these spaces can be improved.

Roundtable discussions took place in the afternoon where City staff, agencies and the public participated in several topic areas hosted by Mattamy Homes and the consultant team including: transportation, servicing, natural environment and open space, as well as the design of McBean Street. The Master Servicing Study presented the preliminary list of alternatives being considered for water and sanitary servicing for the Village.

On Tuesday evening, LRK presented the results of the visioning survey which would serve as the foundation to create the design for the Village Core. Through public input, the Village Core was defined the Perth Street and McBean t-intersection extending along McBean Street to the Jock River bridge.

Wednesday, September 24 and Thursday, September 25

The last two days of the workshop focused on the design of the Village Plan. Participants could view the progress being made on the plan and the street and building designs. The technical information was also on display for the public to review, ask questions and provide comments. On Thursday evening, LRK presented the Village Core Plan at the South March High School. Over a hundred people attended the presentation.

Mattamy's December 2008 Design Workshop

Following the September workshop, Mattamy Homes conducted a similar workshop in December to prepare a conceptual land use plan for Mattamy's lands in Richmond based on the vision developed for the Village. Looney Ricks Kiss facilitated this three-day workshop that took place at the same storefront on McBean Street from December 8-10, 2008. As well, the findings of the technical studies was available through a series of display boards related to planning, design, transportation, stormwater management, natural environment, hydrogeology, water and sanitary servicing. Mattamy Homes consultant team attended the three day event to allow the opportunity for the public to ask questions, provide input and comments on the technical findings and preliminary recommendations.

A total of 52 persons attended the event on one of the three days. This workshop was conducted to assist Mattamy Homes in preparing an Official Plan Amendment application for our future development lands in the Village of Richmond. The workshop was also advertised as a formal meeting (Phase 1) under the Municipal Engineers Association Class Environmental Assessment Process as the Master Servicing Study being prepared by Mattamy Homes is being planned as a Schedule C undertaking.

The storefront opened on Monday, December 8th at 4:00 p.m. There were two open house sessions held from 4:00 p.m. to 6:00 p.m. and again from 6:00 p.m. to 8:00 p.m. to present three different land use options for public review, input and comment. These sessions were conducted as an open house/workshop format to understand the public preferences related to the amount, distribution and type of land use for Mattamy Homes lands. On Tuesday, December 9th the doors opened at 1:00 p.m. to present to the public the consolidated land use plan based on the input heard from the public the previous evening. Mattamy's planning, design and technical studies were also presented through a series of display boards related to planning, design, transportation, stormwater management, natural environment, hydrogeology, water and sanitary servicing. Roundtable discussions took place with the public, city staff and stakeholders on various technical aspects including water and sanitary servicing. The Master Servicing study evaluation criteria were displayed and a roundtable discussion took place on the evaluation process, criteria and weighting. As well the alternatives for water and sanitary servicing were also presented and discussed.

On Tuesday evening, LRK presented the preferred concept plan for Mattamy lands based on the input received on Monday and earlier in the day. Based on the public response, LRK then finalized the concept plan. On Wednesday, December 10th, the open house started at 10:00 a.m. where the public could drop in and visit the design, planning and technical displays as well as see the land use plan that had resulted over the two day workshop. On Wednesday evening, the evolution of the concept plan for Mattamy's land was presented by LRK with design examples associated with different aspects of the plan. The plan was well received by the participants in attendance.

February 12, 2009 Open House

Mattamy Homes held a Public Open House on Thursday, February 12th from 5:00 p.m. to 8:00 p.m. at St. Phillips Catholic Church in the Village of Richmond. A total of 80 persons attended the public open house. The purpose of the open house was to present the land use concept plan for Mattamy's lands, the results of Phase 1 and 2 of the water and sanitary Master Servicing Class EA Study and the findings to date on the planning, natural environment, stormwater and transportation studies supporting Mattamy's Official Plan Amendment. The workshop was also advertised as a formal meeting (Phase 2) under the Municipal Engineers Association Class Environmental Assessment Process as the Master Servicing Study being prepared by Mattamy Homes is being planned as a Schedule C undertaking. The preferred solutions for water and sanitary were presented to the public along with the natural environment constraints, the preliminary stormwater management options and the recommended transportation solutions for the Village.

This meeting provided stormwater management and drainage information associated with the problem, existing conditions, and identification of alternatives satisfying Phase 1, discretionary consultation point of contact.

September 12, 2009 Open House

The City of Ottawa hosted a public open house on Saturday, September 12, 2009 for Mattamy's Official Plan Amendment application. The meeting was held at the Richmond Memorial Community Centre situated at 6095 Perth Street from 9:00 a.m. to Noon. The meeting was well attended with 103 signed-in attendees. The purpose of the meeting was to present the recommendations of the planning and technical studies supporting Mattamy's Official Plan Amendment application. The meeting started with a presentation on the concept plan. An "Ask the Experts" session was then available for participants to visit each of the display stations and ask questions to the consultants. Display materials were exhibited for the concept plan, transportation, stormwater management, natural environment, land use planning, water and wastewater servicing. An open "Question and Answer" period followed to allow additional questions to be asked to Mattamy, the consultant team and City staff. A list of the questions asked by attendees is contained in Appendix B.

Village of Richmond Planning Project Steering Committee

Ward Councillor Glenn Brooks established a Steering Committee to guide the development of the Community Design Plan for the Village of Richmond in concert with City Planning Staff. The Steering Committee was established in April of 2008 and meets once a month at the Richmond Library. These meetings are open to the public and all documentation is filed at the library. Mattamy Homes is a non-voting member of the Steering Committee. Updates on Mattamy's planning and technical studies are provided at these meetings.

Technical Advisory Committee

Mattamy Homes is the proponent of the Village of Richmond Water and Sanitary Master Servicing Study. The Master Servicing Study will identify preferred infrastructure projects that will ultimately be owned and operated by the City of Ottawa. As such, City input along with approval agencies and the public is required throughout the process. At the request of Mattamy Homes, a Technical Advisory Committee was established by the City of Ottawa to provide technical input and advice throughout the preparation of the MSS. As a Stormwater Management and Drainage Plan was also being prepared for Mattamy Homes lands, the TAC was broaden to include stormwater as well. Infrastructure Planning in the City's Infrastructure Services and Community Sustainability Department has been assigned the lead at the City. The Technical Advisory Committee (TAC) is comprised of City staff from various sectors of the City related to water, wastewater and stormwater in areas of policy, planning, approvals, operating and maintenance. As well representatives from the Ministry of Environment and Rideau Valley Conservation Authority participate as members of the TAC. Members of the Richmond Village Steering Committee and interested public were extended an invitation to attend the TAC meetings following the first TAC meeting.

Two meetings have taken place with the TAC through Phases 1 and 2 of the EA process. The first meeting took place on September 28, 2008 with City and agency staff that focused on the existing servicing setting in the Village of Richmond, the workplan for the MSS as well as to introduce the stormwater management and drainage plan study. The evaluation criteria were presented and distributed to the TAC for input and comments. Comments were later received by the TAC requesting that the Operation and Maintenance criterion weight be increased and equal to the capital cost weighting to make sure the maintenance of the system is sustainable in order to protect the City's infrastructure.

As per the MEA Class EA process, it is urged that the proponent contact the Regional Coordinator of the Environmental Assessment and Approvals Branch to discuss the approach being considered for the Master Plan. As well, it is recommended that First Nations and Aboriginal Peoples be recognized as a stakeholder and notified of the Class EA process being undertaken, early in the process. Mattamy Homes wrote to both of these parties in December 2008 notifying them of the Village of Richmond Master Servicing Study and stormwater management and drainage plan being undertaken by Mattamy Homes. The Regional Coordinator responded indicating no concerns with the Master Plan or stormwater management approach. The representative for the First Nations indicated an interest in the Master Servicing Study and Archeological information. This information was sent to the First Nation representative on October 5, 2009. At this time of writing this report, a reply has not been received.

A meeting was held on January 28, 2009 with the RVCA and City staff from Infrastructure Planning to discuss the Stormwater Management and Drainage Plan study as well as the floodplain mapping update being conducted for the Van Gaal Drain by the RVCA. The minutes of this meeting are contained in **Appendix B**.

The second Technical Advisory Committee Meeting was held on February 4, 2009 at City Hall. The focus of this meeting was to present the results of the evaluation of water and sanitary alternatives applying the evaluation criteria developed for the study. As well, the three preliminary stormwater management options were presented by David Schaeffer Engineering Limited. The advertisement for the February 12, 2008 open house was distributed to attendees encouraging their attendance at the meeting. The TAC minutes are contained in **Appendix B**.

January 22, 2009 Agriculture and Rural Affairs Committee

At the January 22, 2009 meeting of the Agriculture and Rural Affairs Committee, City staff presented a report on the status of the Richmond Village Community Design Plan and the processes associated with the future development lands. The report acknowledges the technical studies Mattamy Homes is leading and funding including the Village-wide Master Servicing Study. It states that the City will be using these studies to assist with completing the Richmond Village Community Design Plan that will be provided through Mattamy Homes Official Plan Amendment submission.

Technical Circulation Comments

Mattamy Homes Official Plan Amendment application was circulated for technical and public review and comment in June 2009. The various studies were posted on the City of Ottawa website, Mattamy Homes Richmond website as well hard copies of the reports were made available at the Richmond library. A number of comments were received on the various reports submitted as part of this application. Time has been spent on resolving the relevant issues with City staff, agencies and the public. Revised reports addressing the comments were submitted back to the City beginning in February 2010 for review and concurrence.

The comments received on the DSEL Stormwater Management and Drainage Plan report (March 2009) are contained in ***Appendix B***. This report now replaces the 2009 report originally submitted with the application. Subsequent meeting(s) maybe required with City and RVCA staff following their review of this report.

2.0 RELEVANT STUDIES, GUIDELINES AND POLICIES

2.1 Policies and Guidelines

The following provides a brief summary of the policy, standards and guidelines that are applicable to stormwater management, drainage and sewers that need to be considered when preparing this Stormwater Management and Drainage Plan.

Ottawa 20/20 Official Plan (OP) (Consolidated, 2007)
The Official Plan (OP) provides a framework for future growth in the City of Ottawa. The OP also serves as a basis for a wide range of municipal services, including water and sewage servicing requirements, as well as surface drainage.

Of particular relevance to the Richmond Village project are the determinations of water and sanitary service areas:

Drainage and Stormwater Management:
The Official Plan states that planning to be done on the basis of natural systems to protect and enhance natural processes and ecological functions (e.g. watershed planning, groundwater and surface water protection and green space policies).

Ottawa 20/20 Infrastructure Master Plan (IMP) (June 2003)
The IMP focuses on many aspects related to the planning of infrastructure systems. It is intended to direct the management and extension of public works systems related to

APPENDIX B

Geotechnical Report

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PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

Proposed Residential
Subdivision
Perth and Ottawa Streets
Richmond Area
Ottawa, Ontario

Mattamy Homes Ltd.

PROJECT NO. 1026929

1.0 INTRODUCTION

This report presents the results of the Geotechnical Investigation carried out for the proposed residential subdivision to be located to the west of the Village of Richmond in Ottawa, Ontario. The work was carried out in general accordance with our Proposal Number 1026824, dated June 7, 2007. Authorization to carry out the work was received on June 8, 2007, from Mr. Matthew Kingston of Mattamy Homes – Ottawa Division.

This report has been prepared specifically and solely for the project described herein. It presents the factual results of the Geotechnical Investigation and provides preliminary geotechnical recommendations for the design and construction of the residential subdivision.

2.0 PROJECT DESCRIPTION

The project site location is shown on the Key Plan, Drawing No. 1 in Appendix A. The property boundaries are indicated on Drawing No. 2 in Appendix A, and is based on Drawing Reference No. 07-10-724-00 prepared by J.D. Barnes Surveying Limted and dated April 21, 2007.

Although the layout and configuration of the proposed development has not yet been completed, it is anticipated that it will include numerous single family homes and townhouses on a new street network and that the development will be connected to municipal sewer services. Water supply, it is anticipated, will be through individual private wells.

3.0 SITE DESCRIPTION AND BACKGROUND

Geological maps, of the area indicate that the soil conditions vary significantly across the site. Southeast of Ottawa Street, maps indicate that a layer of glacial till is present overlying bedrock at shallow depth. Northwest of Ottawa Street the soil map profile includes sand (beach and reworked glaciofluvial origin) over silts and clays (marine origin) over glacial till. The thickness of the silt and clay layer increase along with depth to bedrock from southeast to northwest with total overburden thickness estimated to be as much as 10 m at the northwest limit of the site.

The subject site is approximately 300 acres in area and includes nine parcels of land. The site is roughly rectangular, 600 m wide and 2500 m long, with the long axis oriented northwest to southeast. The eight parcels include PINS 0062 and 0061 to the north of Perth Street, 0285, 0286, 0287 and 0714 between Perth and Ottawa Street and the Jock River and PIN 0075 on the southeast side of the Jock River.

The site is primarily used as an agricultural field. The Jock River crosses the southern end of the site. The layout of the site is shown on Drawing No. 2 in Appendix A.

The slopes adjacent to the Jock River are indicated in Ontario Geological Society Paper MP68 to have a Factor of Safety of 2.0 to 2.5 which indicates that only routine inspection will be necessary to determine the need and extent of a geotechnical investigation.

4.0 SCOPE OF WORK

Due to time constraints the scope of work for the project is divided into two phases; the Due Diligence Phase (Preliminary Phase) and Detailed Design Phase. The work completed for this report was conducted as part of the Due Diligence Phase. The scope of for each phase is summarized below:

Due Diligence Phase (Preliminary Phase)

- Field investigation
- Laboratory testing
- Prepare an initial due diligence report which will includes a test hole location plan, draft test hole logs and discussion of the geotechnical constraints to residential development of the site.

Detailed Design Phase

- Design discussions with the project team.
- Prepare a detailed geotechnical design report for the project. The report will include test hole location plan, test hole logs and recommendations for the detailed design of the development. The report will reflect the discussions from the other members of the design team including the proposed grading and layout of the subdivision.

The present report is the due diligence report. Further discussion and analysis will be carried out prior to presenting a final report for the proposed development.

Jacques Whitford is also conducting a Phase I Environmental Site Assessment (ESA) for the site. The results of the Phase I ESA are reported separately.

5.0 METHOD OF INVESTIGATION

5.1 Geotechnical Field Investigation

Prior to carrying out the investigation, Jacques Whitford staked out the proposed borehole locations and made arrangements to identify and clear the locations of underground utilities. The drain tile system that is present on the site could not be accurately located in the field. The drain tile system was encountered and may have been damaged at test pit locations TP07-10, TP07-22 and TP07-23.

Between June 14 and 20, 2007, test holes numbered BH 07-1 to TP 07-72 were advanced at the approximate locations shown on Drawing No. 2 in Appendix A. Several parcel's of the originally planned project site were eliminated from the scope of work, therefore test holes at locations BH 07-4, CPT 07-8, TP 07-12 and BH 07-16 were dropped after initiation of the project.

The test holes consisted of boreholes, Seismic Piezocone Penetration Tests, test pits and hand auger holes.

Boreholes

The boreholes were advanced using a track mounted CME power drill and hollow stem augers. Soil samples were collected at close intervals using a split spoon sampler while conducting standard penetration testing (SPT). Several relatively undisturbed samples of cohesive materials were acquired by pushing thin-walled samplers (Shelby tubes). The undrained shear strength of cohesive soils was measured in the boreholes by carrying out vane shear and pocket penetration tests. The boreholes were backfilled with auger cuttings.

Seismic Piezocone Penetration Test (SCPTu)

The Seismic Piezocone (SCPTu) tests consisted of pushing an instrumented rod with a cone-shaped tip into the ground at a controlled rate (20 mm/s) using the hydraulics of the CME power auger drill. The device contains electronic sensors that measure tip resistance, sleeve friction and pore .water pressure at very short intervals in order to provide a near continuous profile. These measurements can be used to assess a wide variety of engineering properties of the subsurface soils. The equipment standards and test procedures are outlined in ASTM D5779-95. Seismic cone tests were also conducted at 1.0 m intervals to estimate the shear wave velocity of the soil.

When performing a seismic evaluation, the cone contains a built in geophone to measure compression and shear wave velocities in addition to the standard piezocone parameters. The seismic soundings were performed approximately at 1 m intervals.

During the seismic soundings, shear waves are generated by striking the seismic beam using a sledgehammer. The seismic beam is placed under the rear stabilizers of the drill rig and secured to the ground surface by using the weight of the drill. Seismic triggers are attached to this seismic beam, which initiate the recording of the seismic wave trace.

Geophones in the body of the piezocone recognize the arriving waves generated at the ground surface. Any waves received by the geophones on the cone penetrometer are sent back up to the control unit to be displayed on an oscilloscope. On site software then plots the wave amplitude versus time to calculate wave velocities.

For this assignment the piezocone testing also included several pore water pressure dissipation tests.

Test Pits

The test pits were excavated with rubber tired backhoes. Samples of all soil types were acquired from test pit walls. The test pits were backfilled with the excavated materials.

Hand Auger Holes

Access restrictions in the south east corner of the site prevented the use of mechanized equipment. Soil conditions were evaluated based on hand auger holes.

The subsurface stratigraphy encountered in each test hole was recorded in the field by our personnel. All samples recovered were stored in moisture-proof bags and were returned to our laboratory for detailed classification and testing.

Monitoring wells and standpipes were installed in several locations to allow for the measurement of groundwater levels. Groundwater levels were measured on June 20, 2007.

5.2 Survey

Prior to the fieldwork, the borehole locations were established in the field by Jacques Whitford personnel. The ground surface elevations at the test hole locations were surveyed relative to a temporary benchmark with an assigned elevation of 100.00 m Local. The temporary benchmark was established on the top of the concrete base of a light pole located on the north side of Perth Street. The location of the temporary benchmark is shown on Drawing 2 in Appendix A.

5.3 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual examination and additional classification by a geotechnical engineer. Selected samples were tested for moisture content, organic content, particle size gradation and Atterberg Limits. Results of this testing are shown on the Borehole Records and in Appendix D. Consolidation testing on two relatively undisturbed samples and some classification testing is currently underway. The results will be forwarded when they become available.

Samples will be stored for a period of one (1) month after issuance of this report unless we are otherwise directed by the client.

6.0 RESULTS OF INVESTIGATION

6.1 Site Description

The site is primarily used as an agricultural field. Some trees and brush were noted at localized areas throughout the site. Denser trees and brush are present in the vicinity of Jock River. A fill pile was observed near the south side of Ottawa Street. The approximate location of the fill pile, trees and brush are indicated on Drawing No. 2 in Appendix A.

A large drainage ditch is located near the northeast corner of the site. Steel corrugated pipe culvert provides access over the drainage ditch. A second drainage ditch which outlets to the Jock River runs through the southern parcel of land. The approximate locations of the ditches are indicated on Drawing No. 2 in Appendix A.

The slopes adjacent to the Jock River are heavily covered in trees and brush. The slopes heights were visually estimated to be less than 3 m in height and are sloped at angles shallower than 2H:1V.

6.2 Subsurface Information

In general, the subsurface profile changes significantly across the site.

Drawings 3 and 4 in Appendix A present plan summaries of the Estimated Thickness of the Clay Deposit and the Estimated Depth to Bedrock Respectively. Drawing No. 5 in Appendix A presents two stratigraphic plots across the site.

PIN 04437-0062, 0061, PIN 03933-0285, 0286

Within these parcels of land, the soils consist of a thick deposit of clay overlying a till deposit overlying inferred bedrock. Bedrock is anticipated at depths in excess of 6 m below ground surface to the north of Perth Street and becoming shallower to the south of Perth Street. The estimated thickness of the clay deposit and the depth to inferred bedrock are shown on Drawings No. 3 and 4, respectively.

PIN 03933-0287

Within this parcel of land the soils consist of a thin deposit of clay overlying a sandy silt deposit over a till deposit over inferred bedrock. Bedrock is anticipated at depths between 3 m to 4 m below ground surface. The estimated thickness of the clay deposit and the depth to inferred bedrock are shown on Drawings No. 3 and 4, respectively.

PIN 03933-0714 and PIN 03933-0746, 0047, 0075

Within these parcel's of land the soils consist of a deposit sandy silt over a till deposit over inferred bedrock. Bedrock is anticipated at depths greater than 4 m to less than 1 m below ground surface. The depth to inferred bedrock is shown on Drawing No. 4.

The subsurface conditions observed in the test holes are presented in detail on the Test Hole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided. The result of the Seismic Piezocone Tests is provided in Appendix C.

A summary of the observed subsurface conditions is presented below.

6.2.1 Surficial Materials

Topsoil was encountered in the majority of the test pits and boreholes and was observed to range from 0 mm to 300 mm in thickness. Two organic content tests were conducted on grab sample from test pits the results will be provided when testing is complete.

6.3 Clay (CL)

Borehole and Test Pit Results

Beneath the surficial materials a clay deposit was encountered across the majority of the northern half of the site. The upper 2 m to 3 m of clay was greyish brown and had a stiff to firm consistency. This upper clay layer is commonly referred to as a "crust". Several pocket penetration tests and in-situ shear tests were conducted on the clay crust; the result indicated shear strength between 48 kPa to greater than 160 kPa. The moisture content of the crust samples tested ranged from 23% to 53%.

Beneath the crust, the clay was grey and firm as indicated by the measured in-situ shear strength which varied from 20 kPa to 53 kPa. The moisture content of the clay samples tested ranged from 38% to 65%. Soil gradation test results are presented in Appendix D.

The clay can be classified as a lean clay (CL) in accordance with the Unified Soil Classification System (USCS).

The results of consolidation testing are not yet available and will be forwarded upon completion of the tests.

Seismic Piezocone Penetration Test (SCPTu)

The SPCTu tests classify the deposit as clays and organics. The results are presented in Appendix C.

Pore pressure dissipation tests were conducted within the SCPTu. The results of the pressure dissipation tests will be presented at the detailed design stage. The results of the SCPTu tests are presented in Appendix C.

6.4 Sandy Silt (ML)

Borehole and Test Pit Results

Beneath the surficial materials or clay a deposit of sandy silt with variable quantities of gravel was encountered. The deposit was generally encountered to the south of Perth Street. The color of the deposit was brown changing to grey with depth. The deposit was generally loose to compact as indicated by the SPT 'N" values which ranged from 1 to 37.

The moisture content of the samples tested ranged from 13 % to 37%. Soil gradation test results are presented in Appendix D. This material can be classified as sandy silt (ML) in accordance with the Unified Soil Classification System (USCS).

Seismic Piezocone Penetration Test

The SCPTu test classifies the deposit as sands and silt mix. The test results are presented in Appendix C.

6.5 Silty Sand (SM)

Borehole and Test Pit Results

A deposit of silty sand with variable quantities of sand and silt was encountered. The deposit was generally encountered near the Jock River. The color of the deposit was brown changing to grey with depth. The deposit was generally compact based on visual observation during the excavation of the test pits.

Soil gradation test results are presented in Appendix D. This material can be classified as sandy silt (SM) in accordance with the Unified Soil Classification System (USCS).

Seismic Piezocone Penetration Test

The SCPTu test was not conducted in this deposit.

6.6 Glacial Till

Borehole and Test Pit Results

Beneath the sandy silt and silty sand deposits a glacial till was observed. The till consists of a variable mixture of silt, sand and gravel with frequent cobbles and boulders. The deposit was generally compact to dense based on visual observation during the excavation of the test pits.

The moisture content of the samples tested ranged from 8 % to 20%. Wash sieve analysis are presented in Appendix D. This material can be classified as sandy silt (ML) in accordance with the Unified Soil Classification System (USCS).

Seismic Piezocone Penetration Test

The SCPTu was not conducted in this deposit.

6.6.1 Inferred Bedrock

Inferred bedrock was encountered at multiple test locations. The inferred bedrock was based on auger refusal, refusal of the back hoe bucket and refusal of the SCPTu. Table 6.1 summarizes the depth to inferred bedrock and Drawing No. 4 in Appendix A visually presents the bedrock depths inferred across the site.

Table 6.1: Summary of Depth to Bedrock

Test Hole Location	Depth to Inferred Bedrock Measured from Ground Surface (m)
MW 07-18	5.9
BH 07-21	5.3
TP 07-23	4.2
TP 07-24	4.3
MW 07-25	4.8
TP 07-26	4.4
BH 07-27	3.6
BH 07-28	3.9
TP 07-29	3.8
MW 07-30	3.9
TP 07-31	3.8
TP 07-32	3.5
TP 07-33	4.0
TP 07-34	3.6
MW 07-37	3.4
TP 07-36	2.9
TP 07-39	2.9
TP 07-40	1.9
TP 07-41	2.2
TP 07-42	3.3
TP 07-44	2.3
TP 07-45	2.9
TP 07-46	4.0
TP 07-48	3.6
TP 07-49	2.0
TP 07-50	3.8
TP 07-57	0.2
TP 07-58	0.9
TP 07-59	0.5
TP 07-60	1.9
TP 07-61	0.6
TP 07-62	0.7
TP 07-63	0.4
TP 07-64	3.5
TP 07-65	2.9
TP 07-67	1.6
TP 07-71	0.6
TP 07-75	3.5

Table 7.1: Preliminary Grade Raise Restrictions

Site Area	Maximum Grade Raise above Existing Site Grades (m)
0062, 0061; north of Perth Street	1.0
PIN 0285, 0286; parcel to the south of Perth Street	1.5
PIN 0287; parcel north of Ottawa St.	2.0
PIN 0714, 0746, 0047, 0075; parcel north and south of Ottawa St.	4.0

- Shallow bedrock was encountered within the southern half of the site. It is anticipated that bedrock excavation for underground services and building foundations will be required in the area south of Ottawa Street. Estimated depths to inferred bedrock are presented on Drawing No. 4 in Appendix A.
- The soil conditions encountered are suitable for the use of conventional spread and strip footings for the support of structures. Table 7.2 provides Geotechnical Bearing Resistance Values for Preliminary Design purposes. The values have been calculated assuming no modifications to existing grades, a footing depth of 2.4 m below finished grade with an embedment of 0.5 m and a footing width of 0.8 m.

Table 7.2: Geotechnical Resistance for Shallow Foundations

Site Area	Founding Material	ULS (kPa)	SLS (kPa)
PIN 0062, 0061	Firm Clay	125	100
PIN 0285, 0286	Firm Clay	125	100
PIN 0287	Dense sandy silt	300	150
PIN 0714, 0746, 0047, 0075	Dense sandy silt, glacial till	300	150
PIN 0714, 0746, 0047, 0075	Bedrock	500	NA

The Ultimate Limit State (ULS) bearing resistance includes a resistance factor of 0.5. The Serviceability Limit State (SLS) bearing resistance corresponds to total settlement of 25 mm. Differential settlements between footings are expected to be less than 19 mm.

Note that settlement estimates are highly dependent on grade changes. The geotechnical resistance values will need to be reviewed in conjunction with the proposed grading plan.

- The groundwater table was observed to be relatively high. Grade reductions may lead to drainage concerns.

- Several test pits were terminated between 3 m to 4 m below ground surface within the sandy silt deposit due to side wall collapse and groundwater seepage. Excavations below the groundwater level may require shoring protection or special dewatering treatments.
- Several ditch's may require backfilling and the water courses may need diverting.
- In accordance with Table 4.1.8.4.A. of the 2006 Ontario Building Code, the project site is considered to be classified under several site classifications for seismic site response. Table 7.3 summarizes the appropriate site classification for the various parcels of land.

Table 7.3: Site Classification for Seismic Site Response

Parcel of Land	Ground Profile Name	Site Class
PIN 0062, 0061,0285, 0286	Soft Soil	E
PIN 0287, 0714	Stiff Soil	D
PIN 0746, 0047, 0075	Very Dense Soil and Soft Rock	C

- Table 7.4 summarizes the possible re-uses of site generated materials.

Table 7.4: Suitable Re-uses for Site Generated Materials

Soil Deposit	Exterior Foundation Wall Backfill	Subgrade Fill	Landscaping Fill
Clay Crust		X	X
Clay			X
Sandy Silt, Silt,			X
Glacial Till, Silty Sand	X	X	X
Crushed Bedrock	X	X	X

8.0 CLOSURE

This report has been prepared for the sole benefit of Mattamy Homes Ltd and its agents, and may not be used by any third party without the express written consent of Jacques Whitford Limited and Mattamy Homes Ltd. Any use which a third party makes of this report is the responsibility of such third party.

The recommendations made in this report are in accordance with our present understanding of your project. We request that we be permitted to review our recommendations when your drawings and specifications are complete.

This report is based on the site conditions encountered by Jacques Whitford at the time of the work, and at the specific testing and/or sampling locations, and can only be extrapolated to a limited extent around these locations. The extent depends on the variability of soil and groundwater conditions as influenced by geological processes, construction activities and site use. Should any conditions at the site be encountered which differ from those at the test locations, we require that we be notified immediately in order to permit reassessment of our findings.

We trust the above meets your present requirements. Should you require any further information please do not hesitate to contact us.

Yours very truly,

JACQUES WHITFORD LIMITED

DRAFT

Christopher McGrath, P.Eng.
Project Manager

DRAFT

Fred J. Griffiths, Ph.D., P.Eng.
Principal and Group Leader
Geotechnical and Materials Engineering

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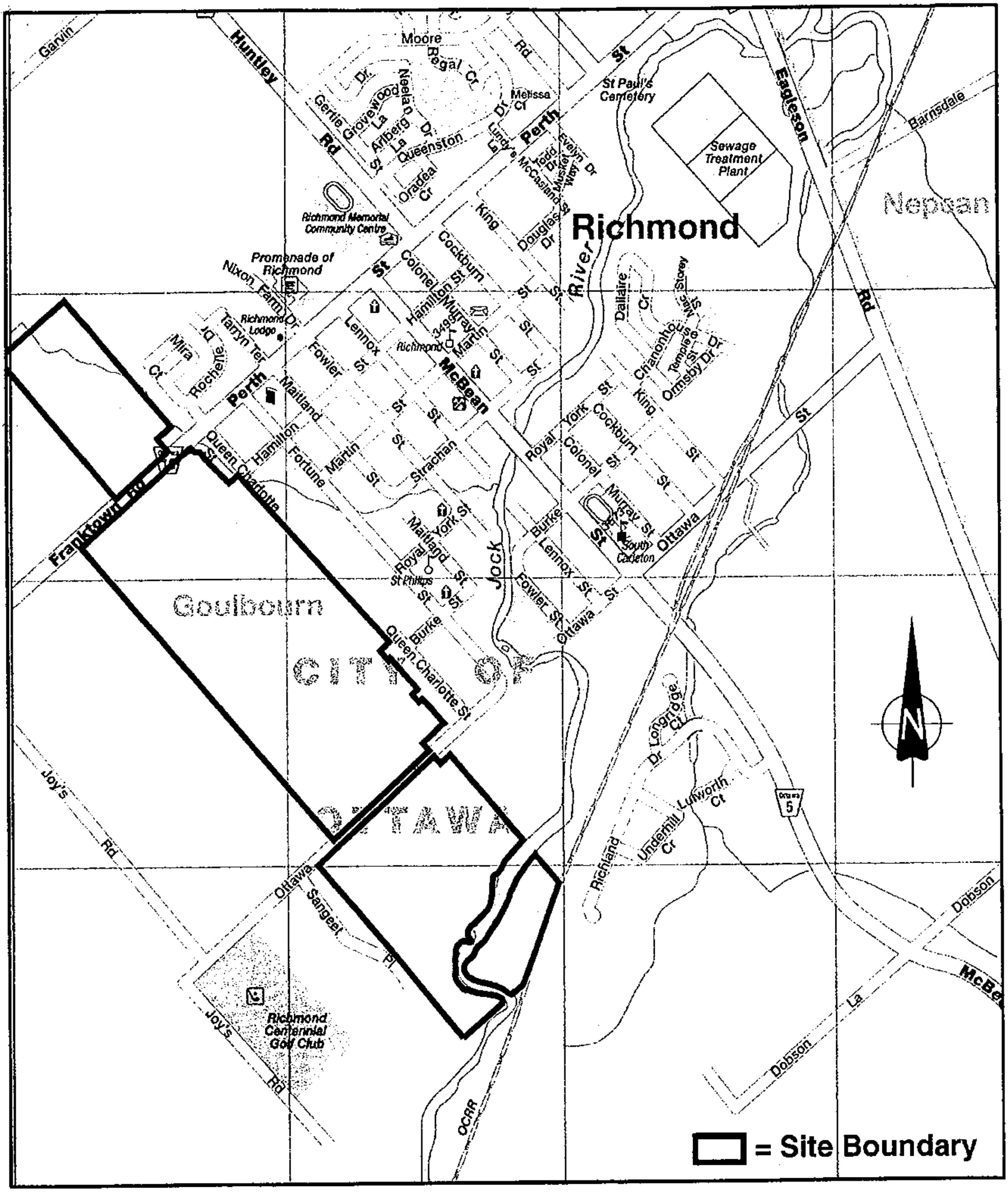
APPENDIX A

Drawing No. 1 - Key Plan
Drawing No. 2 - Test Hole Location Plan
Drawing No. 3 - Estimated Thickness of Clay Deposit
Drawing No. 4 - Inferred Depth to Bedrock
Drawing No. 5 – Stratigraphic Plot
Drawing No. 6 – Stratigraphic Plot



REFERENCE: MAPART

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A JACQUES WHITFORD LIMITED REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

KEY PLAN

GEOTECHNICAL INVESTIGATION FOR RESIDENTIAL SUBDIVISION
PERTH & OTTAWA STREETS, OTTAWA, ON.

Client: MATTAMY HOMES

Job No.: 1026929
Scale: 1 : 20 000
Date: 07/06/21
Dwn. By: GBB
App'd By:

Dwg. No.: 1

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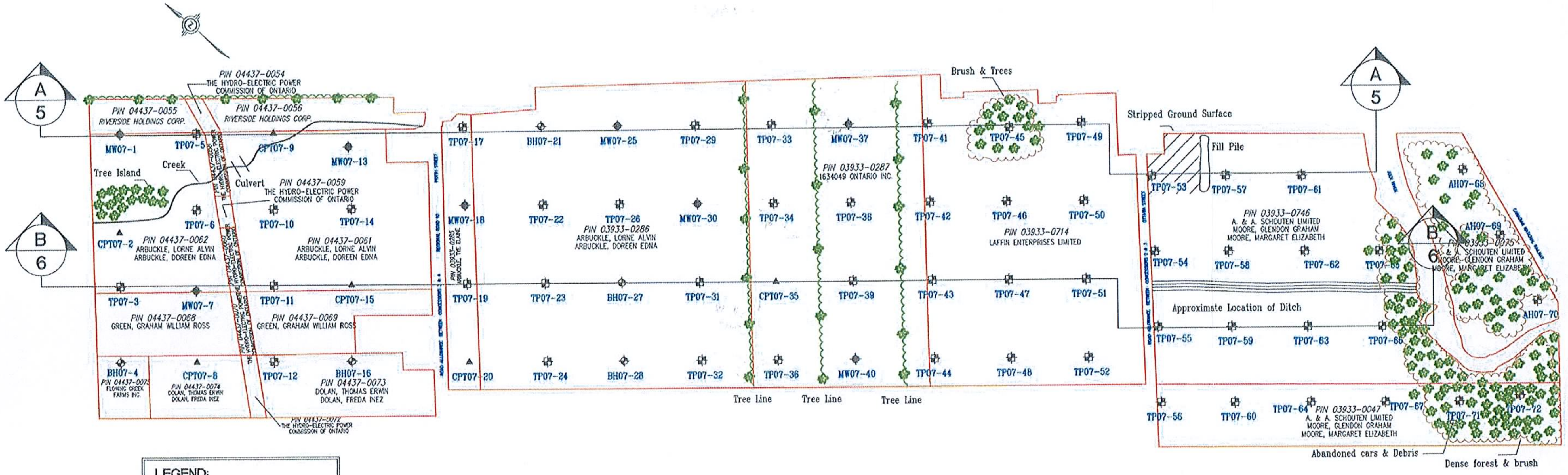

LEGEND:

━	PROPERTY BOUNDARY
▲	SCPTu HOLE
⊕	BOREHOLE
●	MONITORING WELL
⊞	TEST PIT
△ BM	BENCHMARK ASSUMED EL.=100.00 m
━	TREELINE
━	BUSH/VEGETATION

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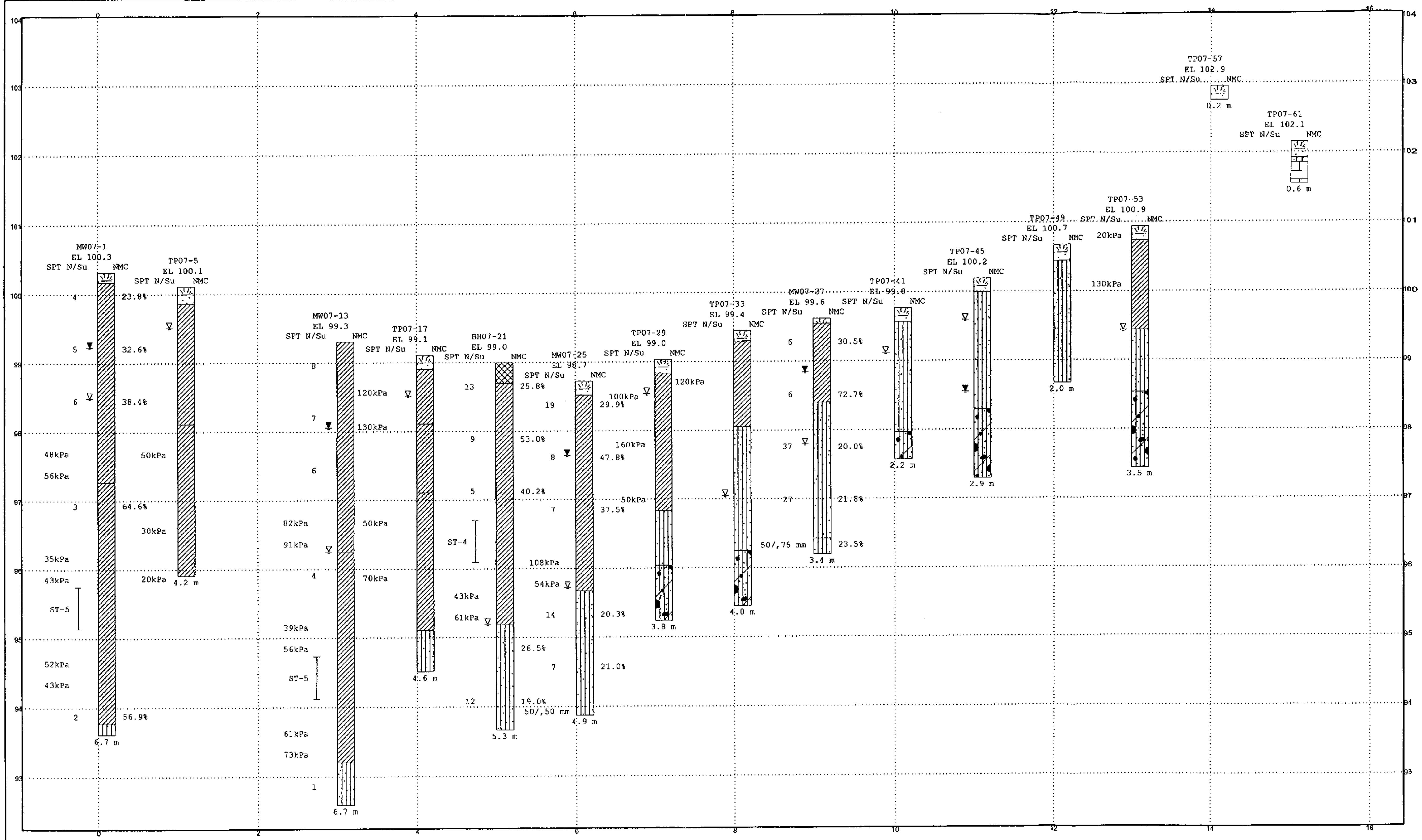
NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A JACQUES WHITFORD LIMITED REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference: Baseline Provided by J.D. Barnes Ltd. Dwg. No. 07-10-724-00 Date: April 21, 2007	Job No.:	1026929	Client: MATTAMY HOMES	GEOTECHNICAL INVESTIGATION FOR RESIDENTIAL SUBDIVISION	TEST HOLE LOCATION	Dwg. No.: 2
	Scale:	1 : 8000				
	Date:	07/06/22	Site Address VILLAGE OF RICHMOND OTTAWA, ONTARIO			
	Dwn. By:	EAG				
	App'd By:					

Jacques Whitford



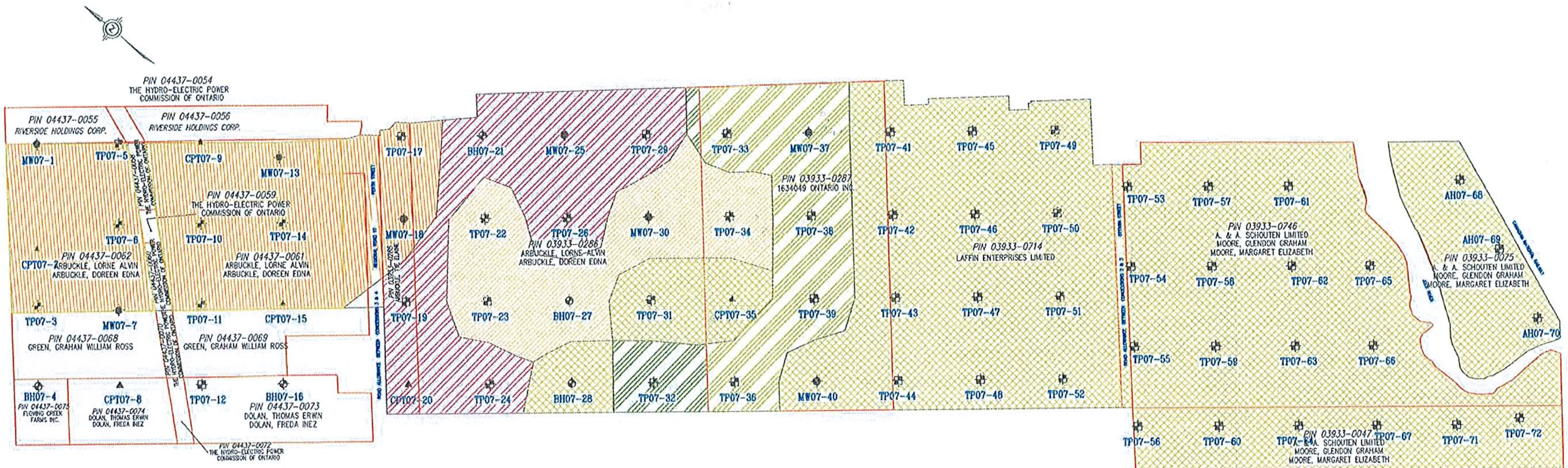
SOIL & ROCK KEY
- BOULDERS, COBBLES & GRAVELS
- SAND & GRAVEL
- SAND
- SILTY SAND
- SILT
- CLAY
- SILTY CLAY
- TILL
- FILL
- ORGANICS
- SHALE BEDROCK
- LIMESTONE BEDROCK
- SANDSTONE BEDROCK

Jacques Whitford
2781 Lancaster Road,
Ottawa, Ontario K1B 1A7

Title: Summary of Test Hole Data
Mattamy Homes, Village of Richmond
Cross Section A

Figure No. 5

Drawn: BDG
Approved: FG
Date: 07/06/22
Job No. 1026929



LEGEND:

ESTIMATED THICKNESS OF CLAY DEPOSIT	
> 4 m in thickness	▥
3 m to 4 m	▨
2 m to 3 m	▦
1 m to 2 m	▨
No clay	▦

PROPERTY BOUNDARY	—
BCPTu HOLE	▲
BOREHOLE	⌖
MONITORING WELL	●
TEST PIT	⌗
BENCHMARK (TOP OF FIRE HYDRANT) ASSUMED EL.=100.00 m	△ BM

Notes :

-The thicknesses are inferred from the test hole results.

-Conditions between test holes could very from those encountered at the test hole.

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A JACQUES WHITFORD LIMITED REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference:
Baseline provided by J.D. Barnes Ltd.
Dwg No. 07-10-724-00
Date: April 21, 2007

Job No.: 1026929
Scale: 1 : 8000
Date: 07/06/22
Dwn. By: EAG
App'd By:

Client: MATTAMY HOMES

Site Address: VILLAGE OF RICHMOND
OTTAWA, ONTARIO

GEOTECHNICAL INVESTIGATION
FOR RESIDENTIAL SUBDIVISION

ESTIMATED THICKNESS
OF CLAY DEPOSIT

Dwg. No.: 3

Jacques Whitford

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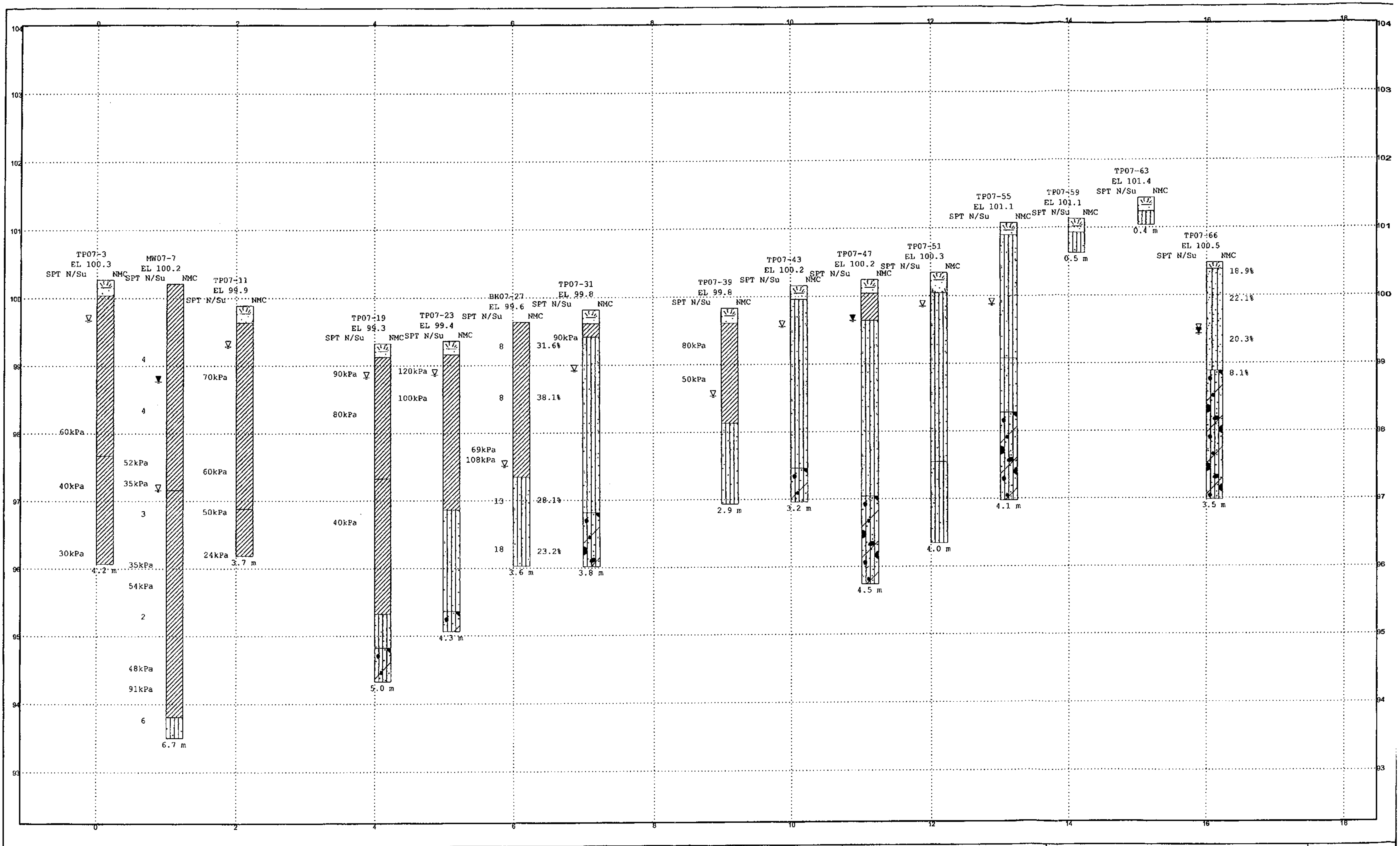


SOIL & ROCK KEY

- BOULDERS, COBBLES & GRAVELS
- SAND & GRAVEL
- SAND
- SILTY SAND
- SILT
- CLAY
- SILTY CLAY
- TILL
- FILL
- ORGANICS
- SHALE BEDROCK
- LIMESTONE BEDROCK
- SANDSTONE BEDROCK

Jacques Whitford
2781 Lancaster Road,
Ottawa, Ontario K1B 1A7

Title: Summary of Test Hole Data
Mattamy Homes, Village of Richmond
Cross Section B

Figure No. 6

Drawn: BDG
Approved: FG
Date: 07/06/22
Job No. 1026929

APPENDIX B

Symbols and Terms Used on the Borehole Records
Borehole and Test Pit Records

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
Very Soft	<0.25	<12.5
Soft	0.25 - 0.5	12.5 - 25
Firm	0.5 - 1.0	25 - 50
Stiff	1.0 - 2.0	50 – 100
Very Stiff	2.0 - 4.0	100 - 200
Hard	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor*
25-50	*Poor*
50-75	*Fair*
75-90	*Good*
90-100	*Excellent*

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

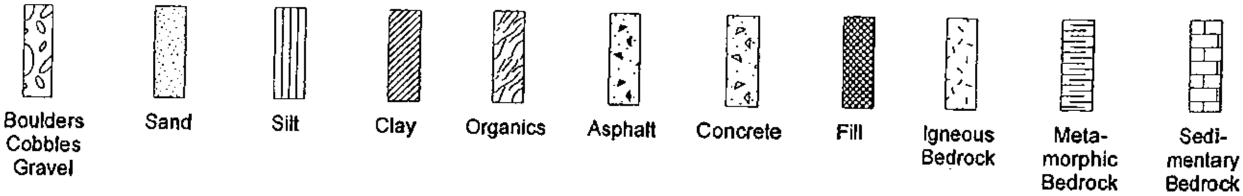
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

Term	Description
Fresh	No visible signs of rock weathering. Slight discolouration along major discontinuities
Slightly Weathered	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
Moderately Weathered	Less than half the rock is decomposed and/or disintegrated into soil.
Highly Weathered	More than half the rock is decomposed and/or disintegrated into soil.
Completely Weathered	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼	measured in standpipe, piezometer, or well
▽	inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE / RQD

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log. RQD is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability. Soil type may be inferred from adjacent boreholes and test pits.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

Jacques Whitford

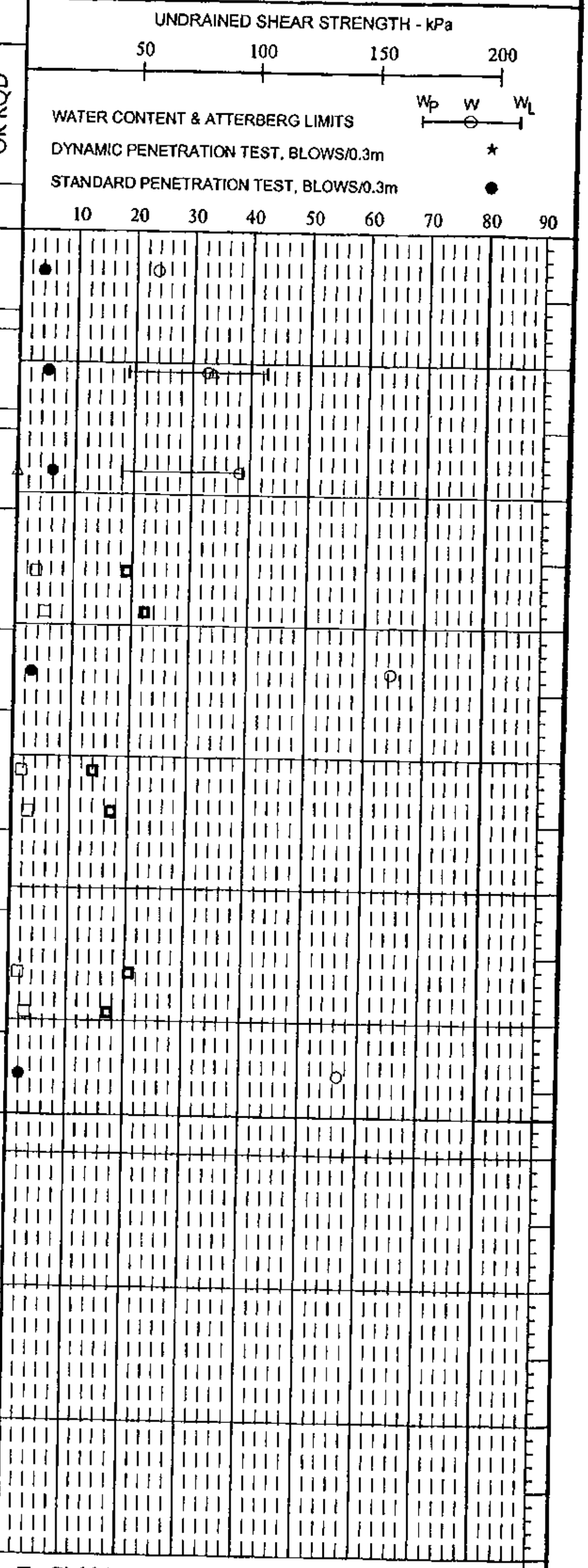
MONITORING WELL RECORD

MW07-1

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 18, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. MW07-1
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**100.32**							
	100.2	150 mm TOPSOIL						
		Firm to stiff, greyish brown lean CLAY (CL)			SS	1	300	4
1				▼	SS	2	610	5
2				▽	SS	3	610	6
3	97.3	Firm to stiff, grey lean CLAY			SS	4	610	3
4								
5					ST	5	610	
6								
	93.8				SS	6	610	2
7	93.6	Very loose, grey SANDY SILT (ML) End of Borehole Monitoring Well Installed						
8								
9								
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ________
Date ________

Jacques Whitford

TEST PIT RECORD

TP07-3

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

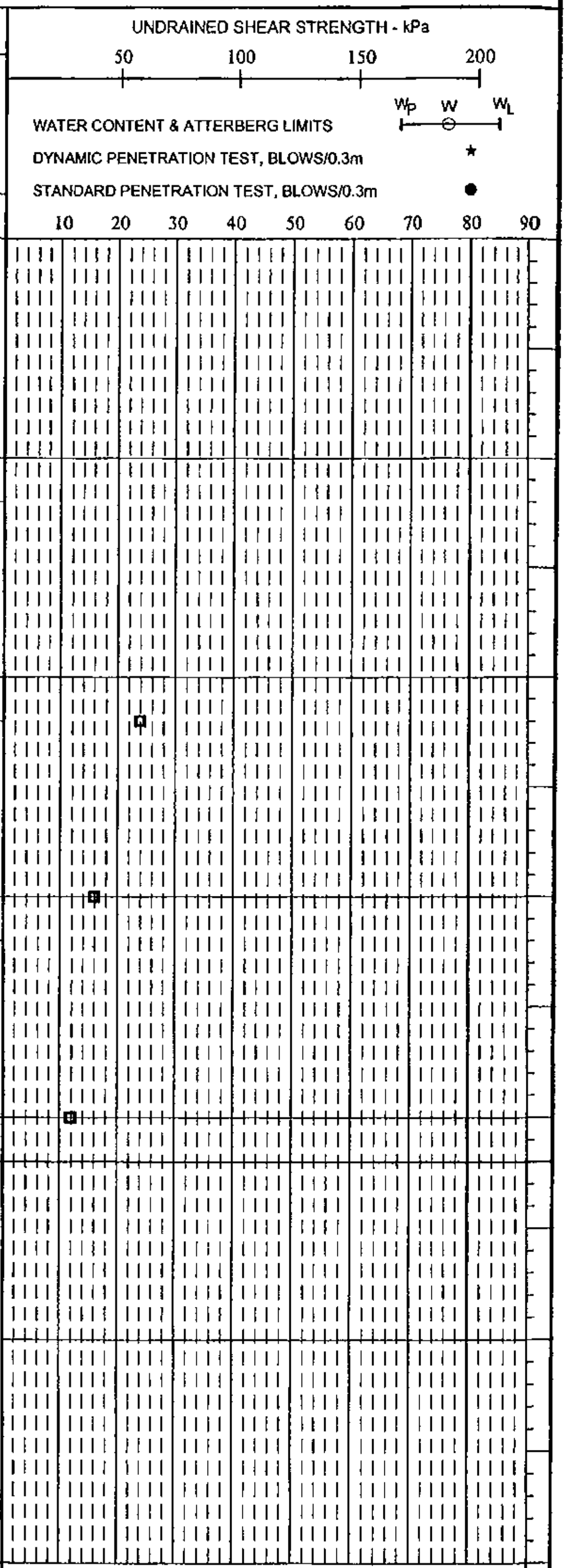
DATES: BORING June 16, 2007 WATER LEVEL

BOREHOLE No. TP07-3

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.27	230 mm TOPSOIL						
	100.0	Stiff, greyish brown lean CLAY (CL)		∇				
1					BS	1		
2					BS	2		
	97.7	Firm, grey lean CLAY (CL)						
3					BS	3		
4					BS	4		
	96.1	End of Borehole						
5								
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

BOREHOLE RECORD

BH07-4

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING ______ WATER LEVEL ______

BOREHOLE No. BH07-4

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD	UNDRAINED SHEAR STRENGTH - kPa
0		Borehcle was not drilled							
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									

UNDRAINED SHEAR STRENGTH - kPa: 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level

$\blacktriangledown$ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

Jacques Whitford

TEST PIT RECORD

TP07-5

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

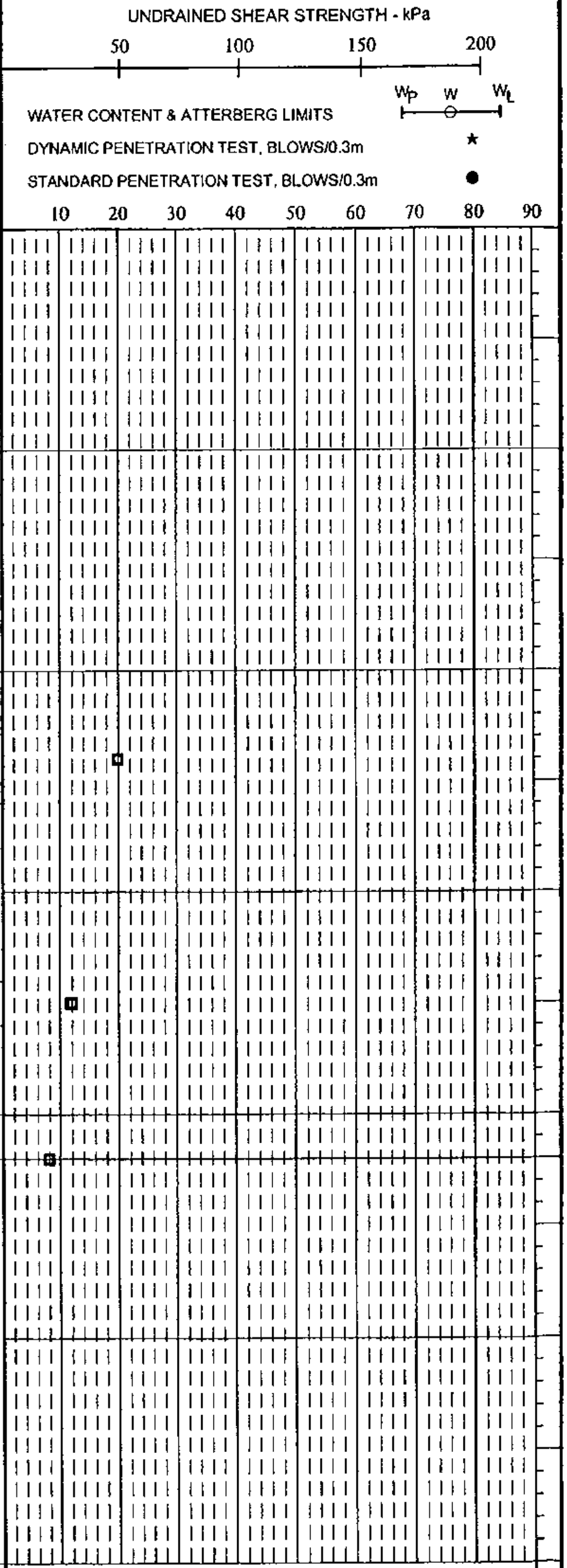
DATES: BORING June 16, 2007 WATER LEVEL

BOREHOLE No. TP07-5

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.11	250 mm TOPSOIL			BS	1		
	99.9	Stiff, greyish brown lean CLAY (CL)	▨	⊻				
1			▨		BS	2		
			▨		BS	3		
2	98.1	Firm, grey lean CLAY (CL)	▨					
			▨		BS	4		
3			▨					
			▨		BS	5		
4			▨		BS	6		
	95.9	End of Borehole						
5								
6								



⊻ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

TEST PIT RECORD

TP07-6

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

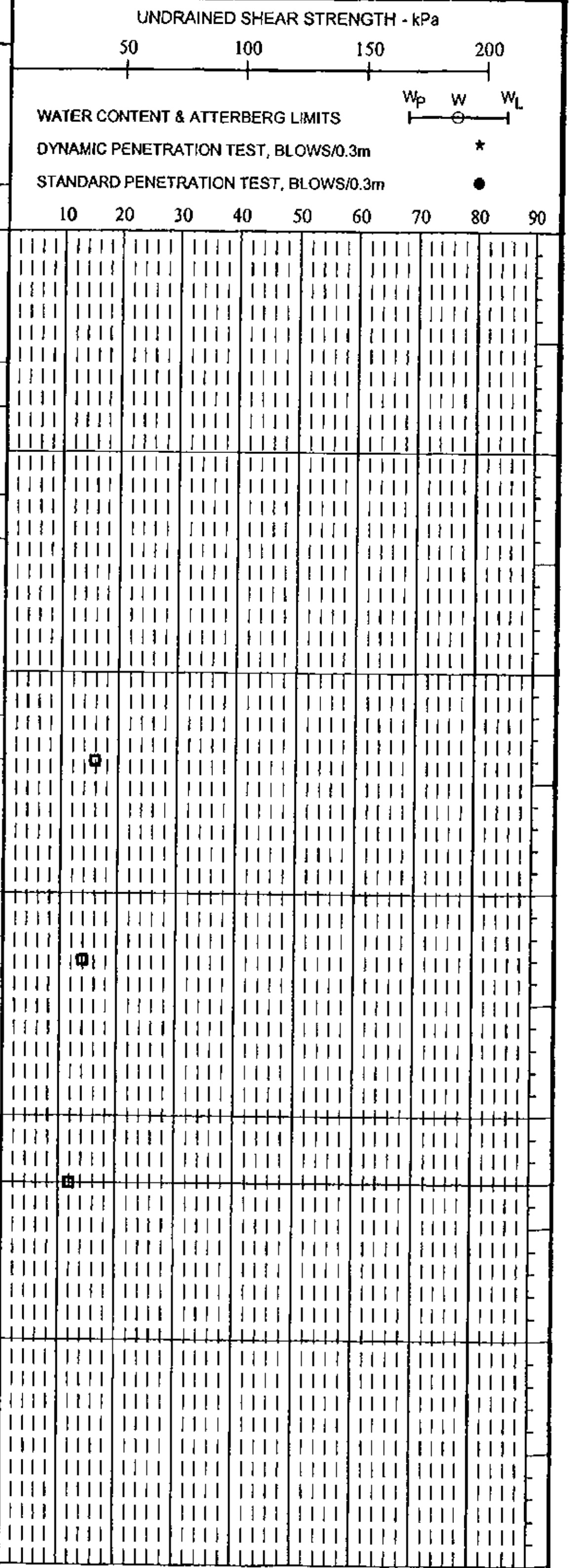
DATES: BORING June 16, 2007 WATER LEVEL

BOREHOLE No. TP07-6

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.85	250 mm TOPSOIL						
	99.6	Compact to dense, brown and grey SANDY SILT (ML)						
				∇	BS	1		
1					BS	2		
2	97.9	Firm, grey lean CLAY (CL)	▨					
					BS	3		
3					BS	4		
4	95.6				BS	5		
		End of Borehole						
5								
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

1 of 1

Jacques Whitford

MONITORING WELL RECORD

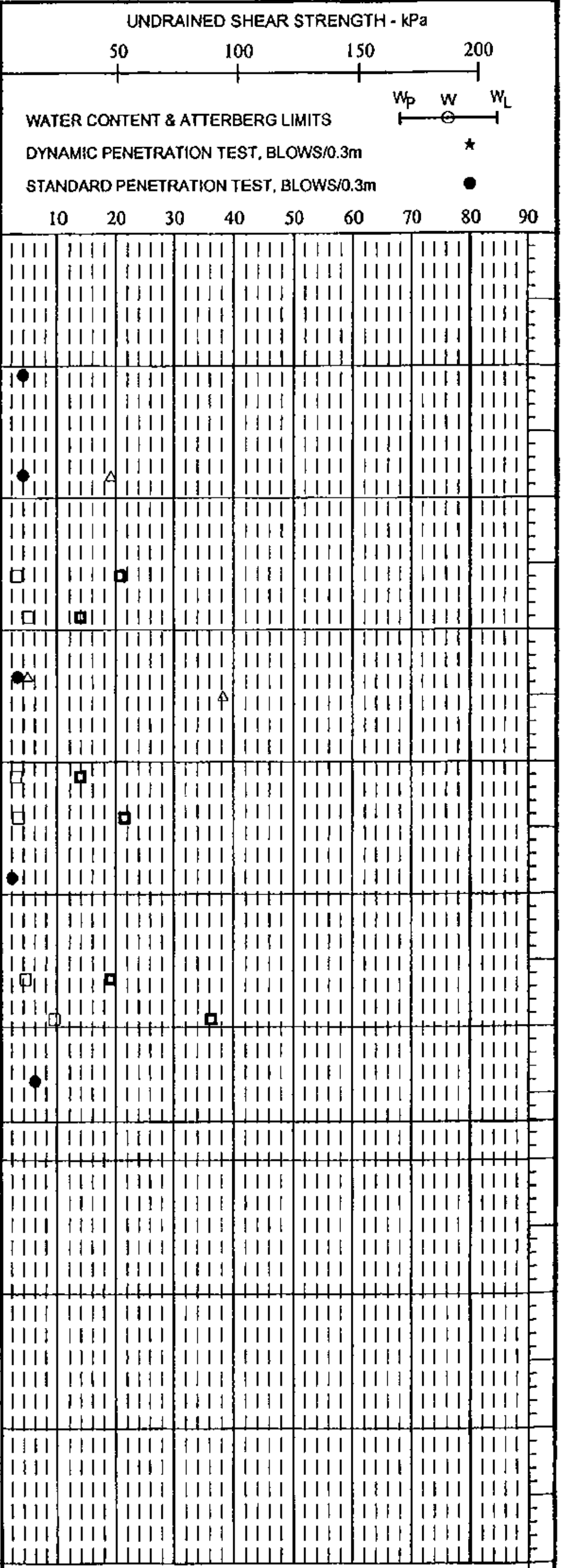
MW07-7

CLIENT Mattamy Homes — BOREHOLE No. MW07-7

LOCATION Proposed Subdivision, Richmond, ON — PROJECT No. 1026929

DATES: BORING June 18, 2007 WATER LEVEL June 20, 2007 — DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	100.21	Firm to stiff, greyish brown lean CLAY (CL)	▨					
1			▨		SS	1	610	4
			▨	▼				
2			▨		SS	2	610	4
3	97.2	Firm to stiff, grey lean CLAY	▨	▽	SS	3	610	3
4			▨					
5			▨		SS	4	610	2
6	93.8	Loose, grey SANDY SILT (ML)			SS	5	150	6
7	93.5	End of Borehole Monitoring Well Installed						
8								
9								
10								



▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

Jacques Whitford

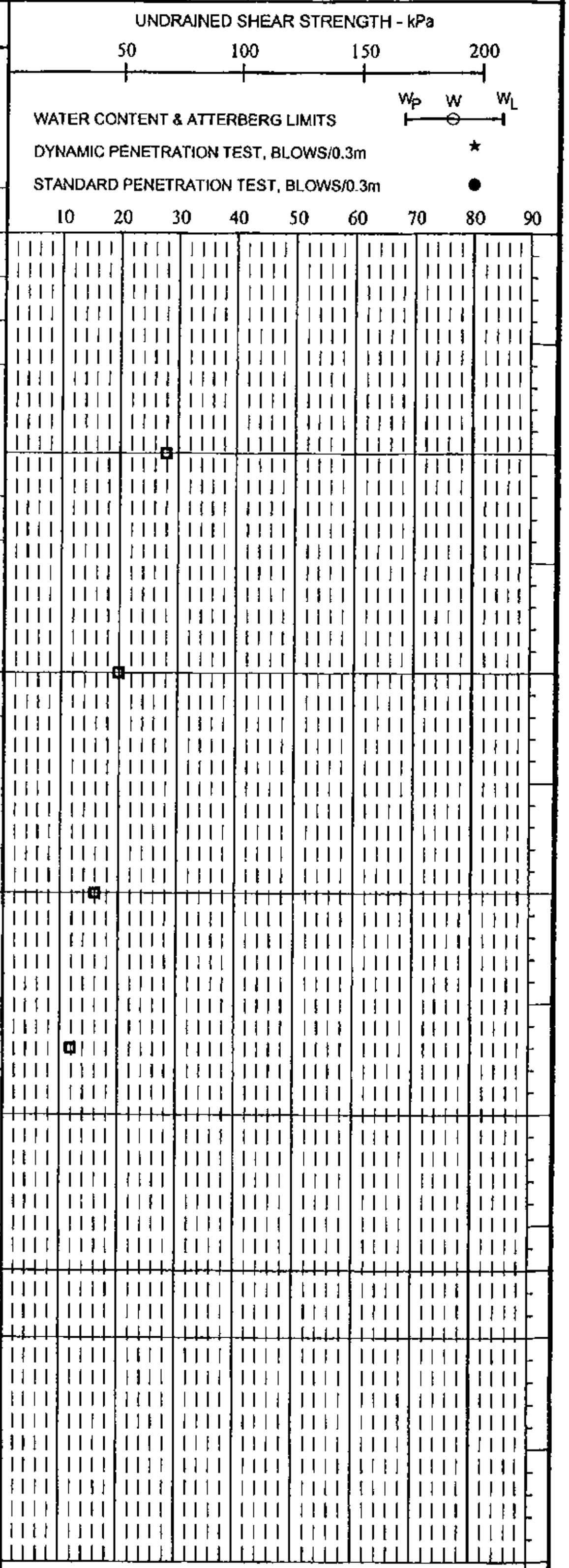
TEST PIT RECORD

TP07-10

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 16, 2007 WATER LEVEL
BOREHOLE No. TP07-10
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.69	250 mm TOPSOIL			BS	1		
	99.4	Stiff, greyish brown to grey lean CLAY (CL)		▽	BS	2		
1					BS	3		
2	97.7	Firm, grey lean CLAY (CL)			BS	4		
3					BS	5		
					BS	6		
4					BS	7		
	95.0	End of Borehole						
5								
6								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd

Date

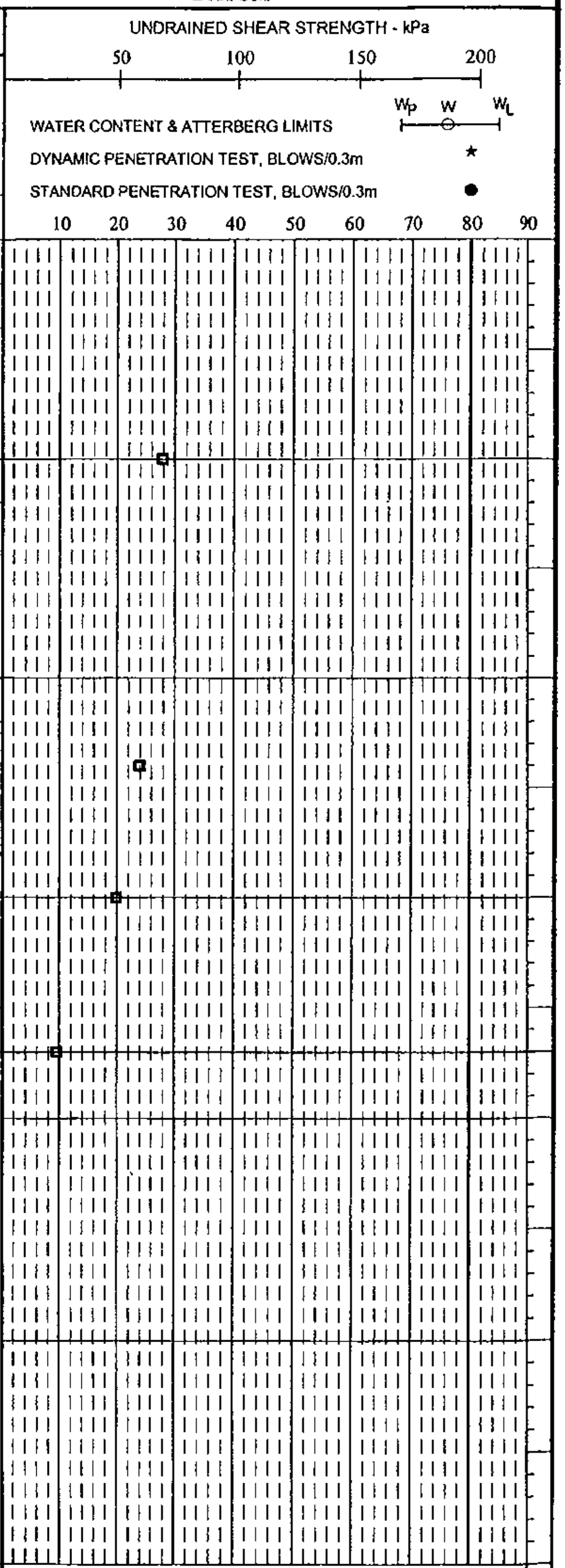
Jacques Whitford

TEST PIT RECORD

TP07-11

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 16, 2007 WATER LEVEL
BOREHOLE No. TP07-11
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.89	250 mm TOPSOIL						
	99.6	Stiff, brown and grey lean CLAY (CL)			BS	1		
				⊻				
1					BS	2		
2					BS	3		
					BS	4		
3	96.9	Firm, grey lean CLAY (CL)						
					BS	5		
	96.2	End of Borehole						
4								
5								
6								



⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-12

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING ______ WATER LEVEL ______

BOREHOLE No. TP07-12

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0		Test pit was not excavated						
1								
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

Jacques Whitford

MONITORING WELL RECORD

MW07-13

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

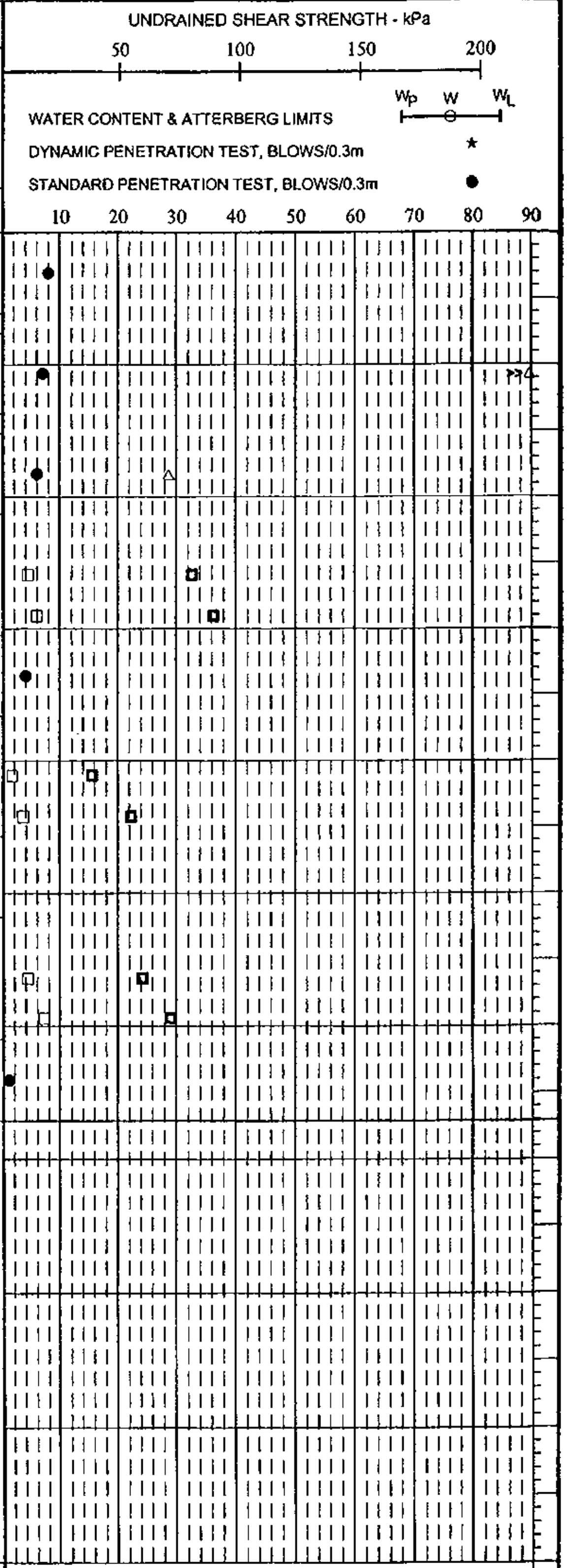
DATES: BORING June 18, 2007 WATER LEVEL June 20, 2007

BOREHOLE No. MW07-13

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.30	Stiff, greyish brown lean CLAY (CL)	SS	1	120	8
1			SS	2	75	7
2			SS	3	610	6
3	96.3	Firm to stiff, grey lean CLAY	SS	4	40	4
5			ST	5	610	
6	93.2	Very loose, grey SANDY SILT (ML)	SS	6	300	1
7	92.6	End of Borehole Monitoring Well Installed				
8						
9						
10						



▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

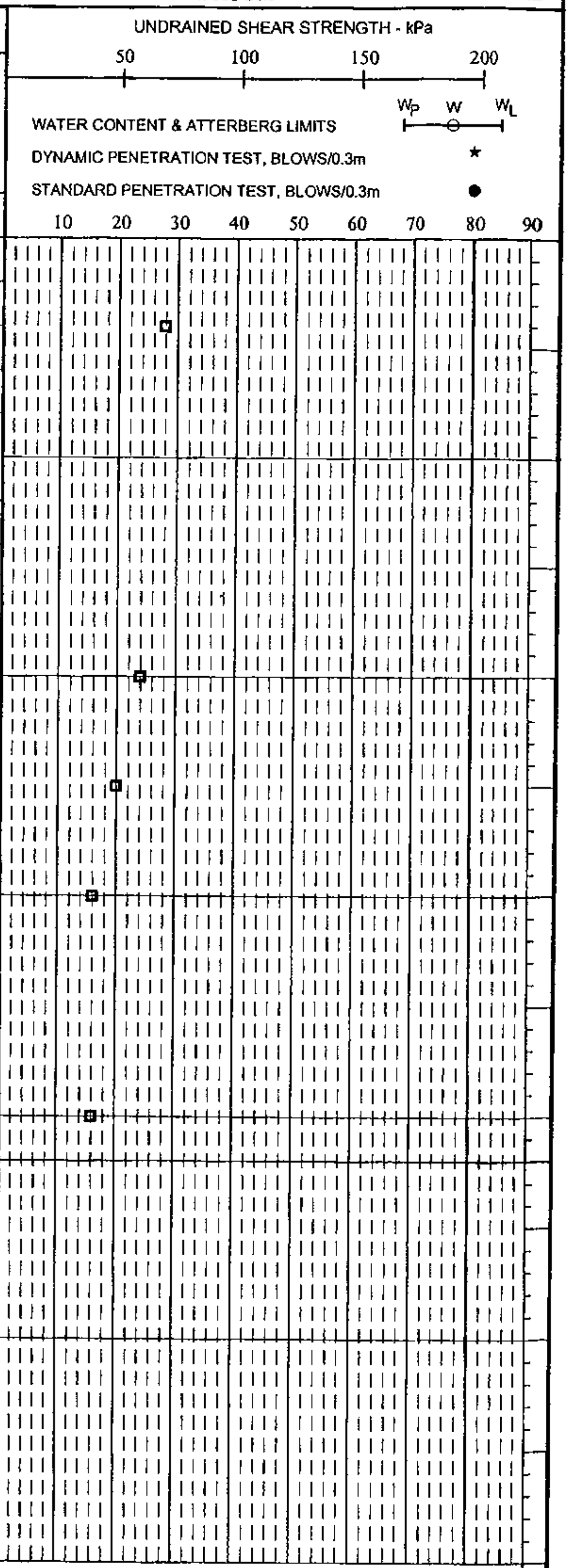
Jacques Whitford

TEST PIT RECORD

TP07-14

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 16, 2007 WATER LEVEL
BOREHOLE No. TP07-14
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.65	230 mm TOPSOIL						
	99.4	Compact to dense, greyish brown clayey silty sand		∇	BS	1		
	99.3	Stiff, greyish brown lean CLAY (CL)			BS	2		
1					BS	3		
2	97.7	Firm, grey lean CLAY (CL)			BS	4		
3					BS	5		
4					BS	6		
	95.5	End of Borehole						
5								
6								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

BOREHOLE RECORD

BH07-16

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING

WATER LEVEL

BOREHOLE No. BH07-16

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES				UNDRAINED SHEAR STRENGTH - kPa 50 100 150 200 WATER CONTENT & ATTERBERG LIMITS Wp W WL DYNAMIC PENETRATION TEST, BLOWS/0.3m * STANDARD PENETRATION TEST, BLOWS/0.3m ● 10 20 30 40 50 60 70 80 90
					TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD	
0		Borehole was not drilled							
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

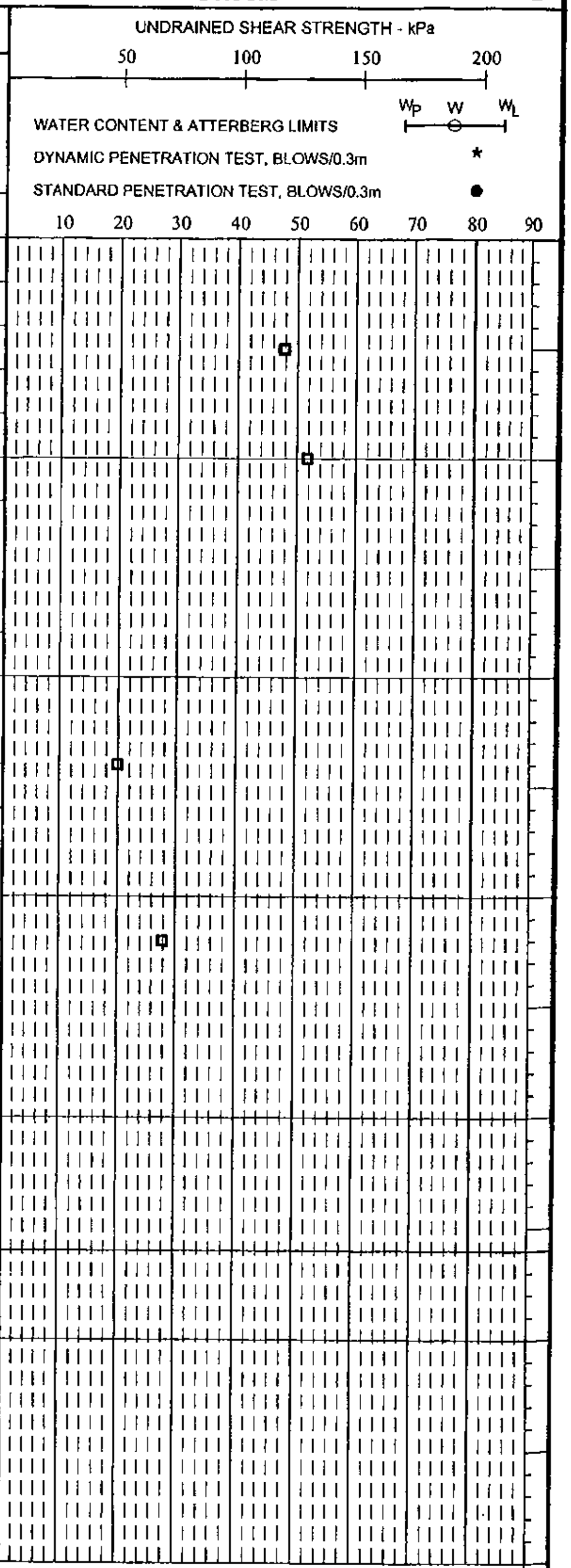
Jacques Whitford

TEST PIT RECORD

TP07-17

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 16, 2007 WATER LEVEL
BOREHOLE No. TP07-17
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**99.11**							
	98.9	200 mm TOPSOIL						
		Very stiff, brown lean CLAY (CL)			BS	1		
				∇				
					BS	2		
1	98.1	Stiff, greyish brown lean CLAY (CL)			BS	3		
					BS	4		
2	97.1	Firm, grey lean CLAY (CL)						
					BS	5		
3								
					BS	6		
4	95.1	Dense, grey SANDY SILT (ML)						
					BS	7		
	94.5	End of Borehole						
5								
6								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
▢ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ________
Date ________

Jacques Whitford

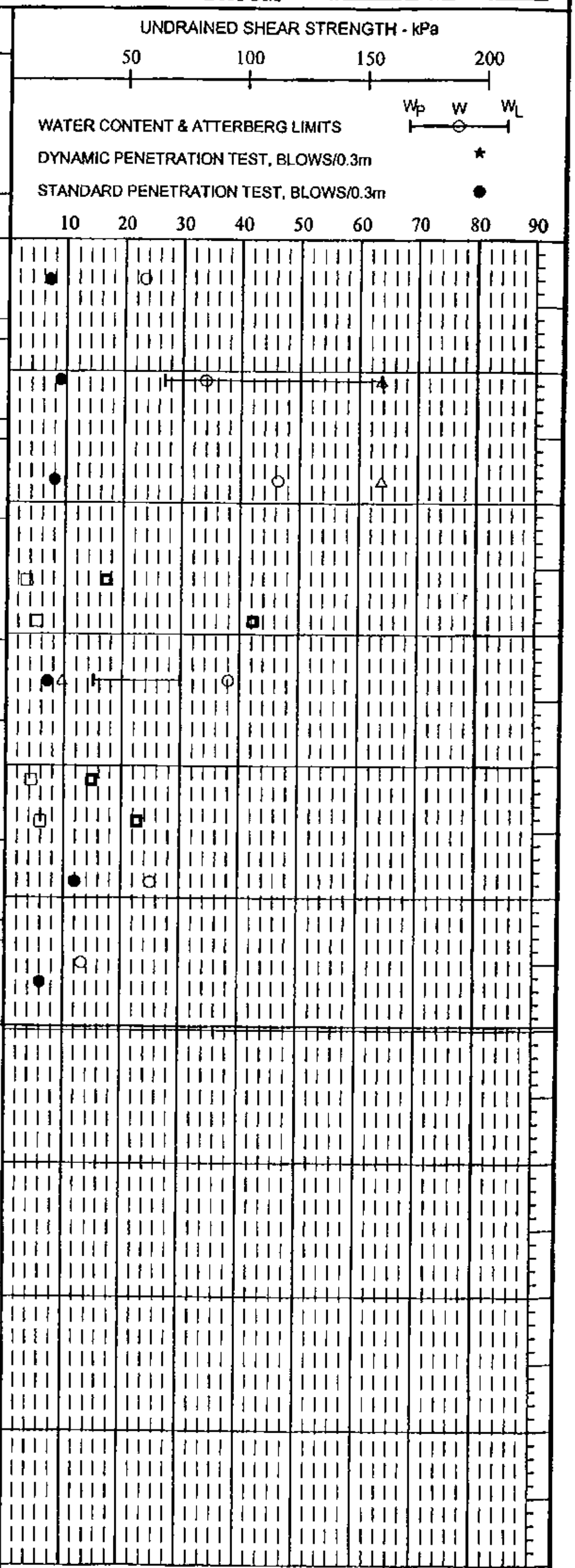
MONITORING WELL RECORD

1 of 1

MW07-18

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. MW07-18
PROJECT No. 1026929
DATUM Local

ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
99.19	Firm to stiff, greyish brown lean CLAY (CL)	SS	1	190	7
		SS	2	420	9
		SS	3	520	8
97.1	Firm to stiff, grey lean CLAY (CL)				
		SS	4	610	7
94.6	Loose to compact, grey SANDY SILT (ML)	SS	5	520	12
		SS	6	350	6
93.4	Loose, grey silty sand with gravel: TILL (SM)				
93.2	End of Borehole Auger Refusal on Inferred Bedrock Monitorng Well Installed				



DEPTH (m): 0 – 10

¥ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

TEST PIT RECORD

TP07-19

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

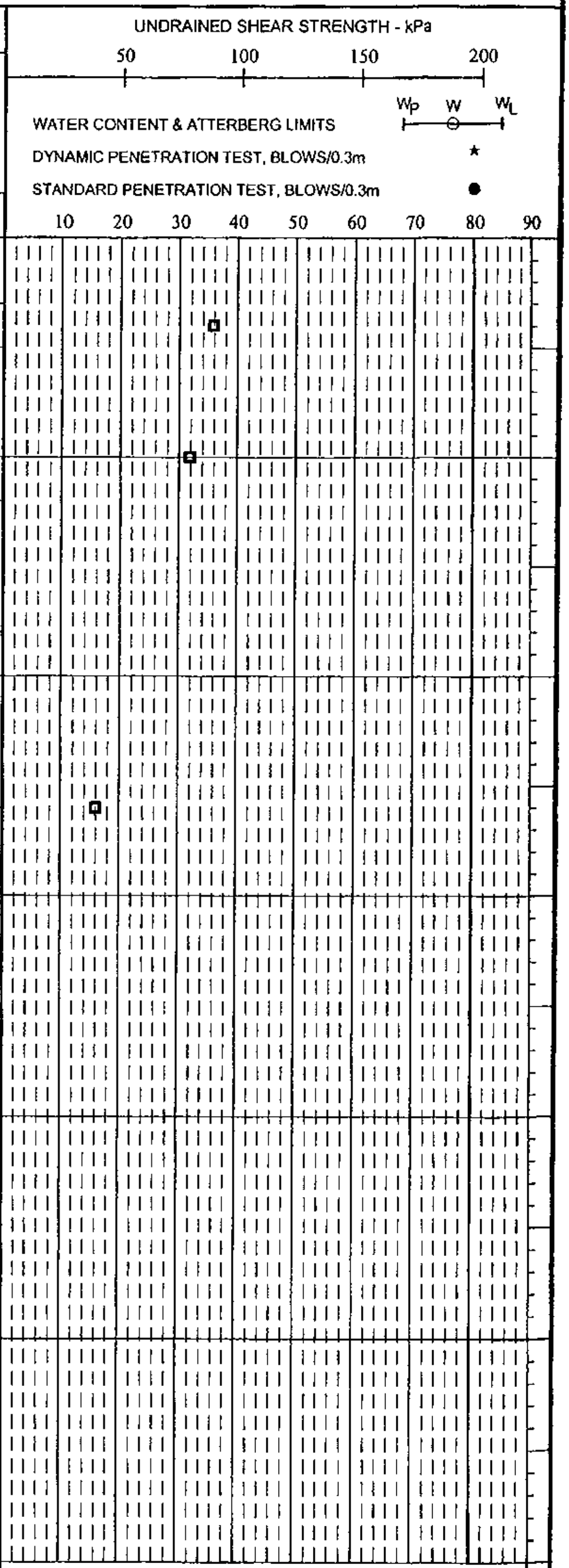
DATES: BORING June 16, 2007 WATER LEVEL

BOREHOLE No. TP07-19

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.33	200 mm TOPSOIL						
	99.1	Stiff, greyish brown lean CLAY (CL)	▨	∇	BS	1		
1			▨		BS	2		
			▨		BS	3		
2	97.3	Firm, grey lean CLAY (CL)	▨					
			▨		BS	4		
3			▨					
			▨		BS	5		
			▨		BS	6		
4	95.3	Dense, grey SANDY SILT (ML)			BS	7		
	94.8	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	8		
5	94.3	End of Borehole						
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

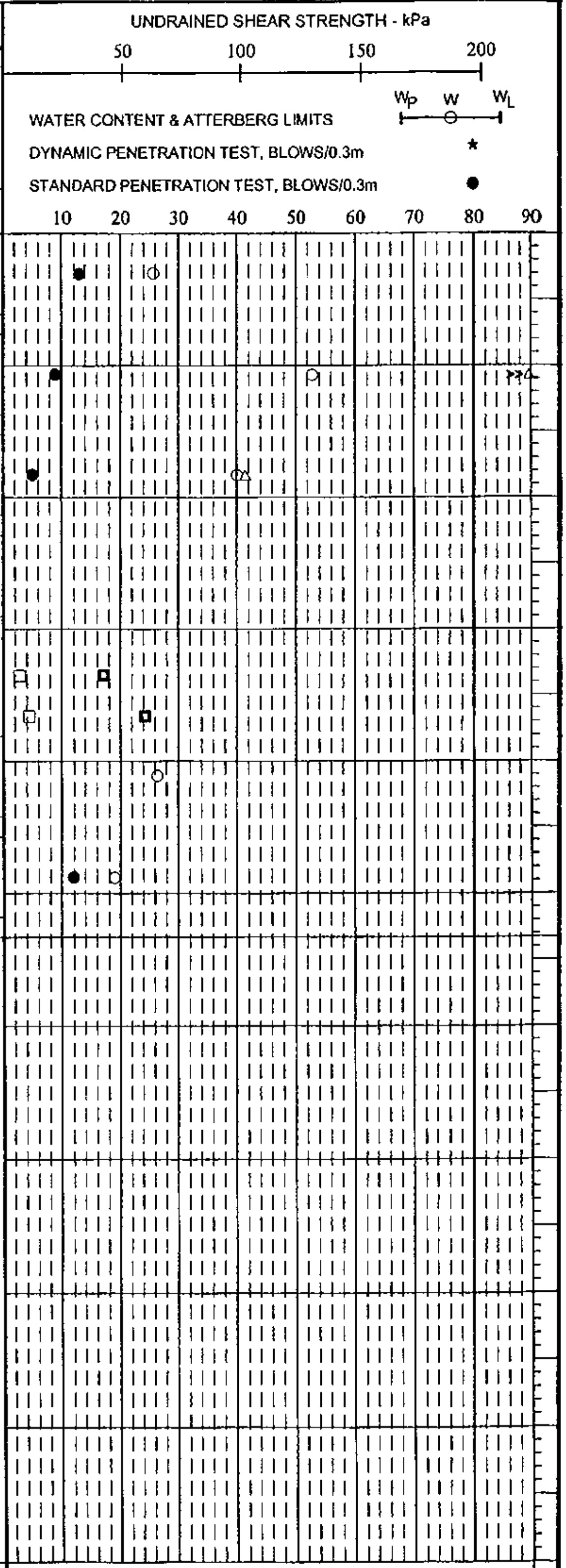
Jacques Whitford

BOREHOLE RECORD

BH07-21

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. BH07-21
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	98.99							
	98.7	Loose brown sandy silt, some gravel, occasional cobbles: FILL			SS	1	100	13
1		Firm to stiff, greyish brown lean CLAY (CL)			SS	2	280	9
2					SS	3	610	5
3					ST	4	620	
4	95.2	Compact, grey SANDY SILT (ML)		▽	SS	5	150	
5					SS	6	360	12
	93.7	End of Borehole Auger Refusal on Inferred Bedrock						



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

Jacques Whitford

TEST PIT RECORD

TP07-22

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

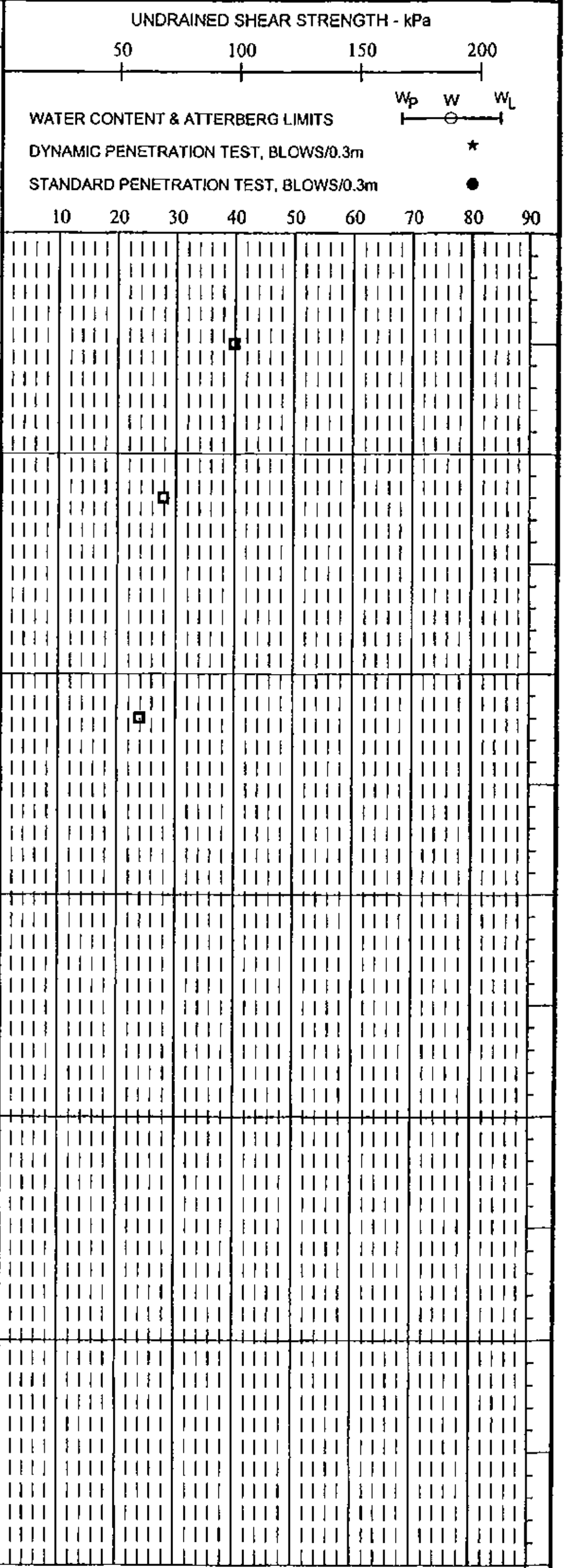
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-22

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.23							
	99.0	200 mm TOPSOIL			BS	1		
		Stiff, greyish brown lean CLAY (CL)			BS	2		
1					BS	3		
2				∇				
	97.0	Dense, grey SANDY SILT (ML)			BS	4		
3					BS	5		
					BS	6		
4	95.2	End of Borehole						
5								
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

1 of 1

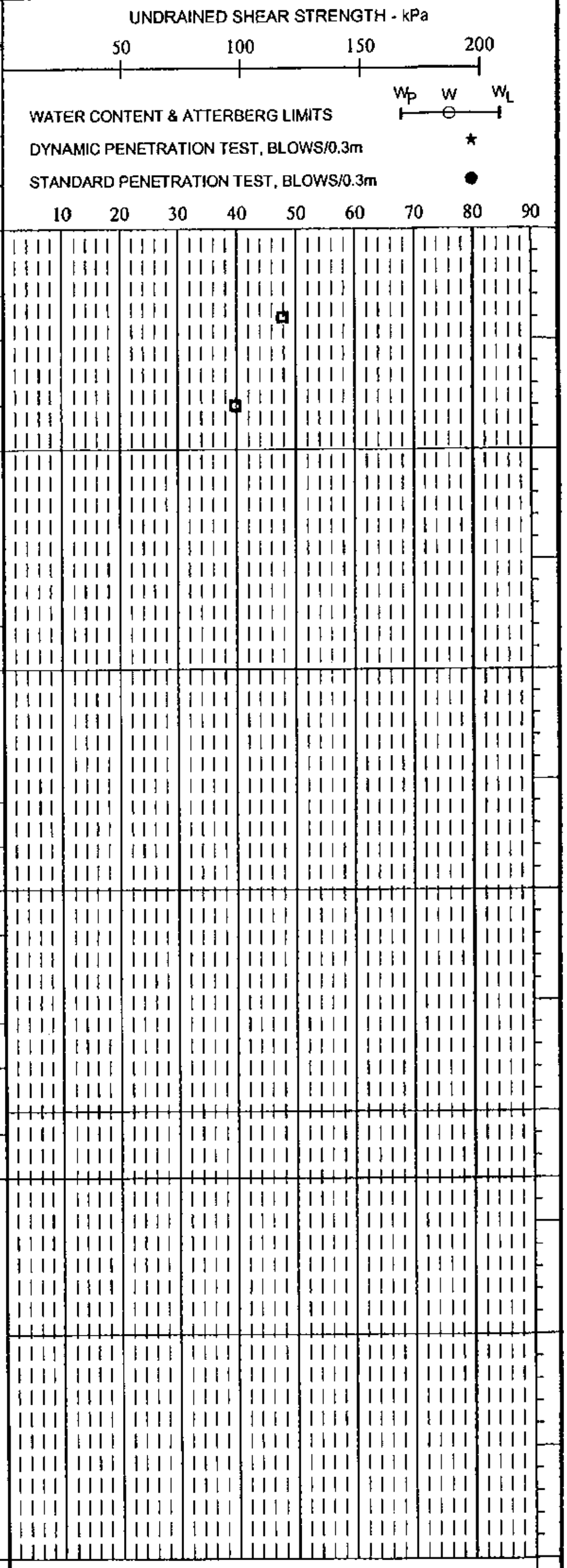
Jacques Whitford

TEST PIT RECORD

TP07-23

CLIENT Mattamy Homes
BOREHOLE No. TP07-23
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 15, 2007 WATER LEVEL
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.36							
	99.2	200 mm TOPSOIL						
		Very stiff, greyish brown lean CLAY (CL)		∇	BS	1		
1					BS	2		
2					BS	3		
	96.9	Compact to dense, grey and brown SANDY SILT (ML)			BS	4		
3					BS	5		
					BS	6		
4	95.4	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	7		
	95.1	End of Borehole Refusal on Inferred Bedrock						
5								
6								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ________
Date ________

1 of 1

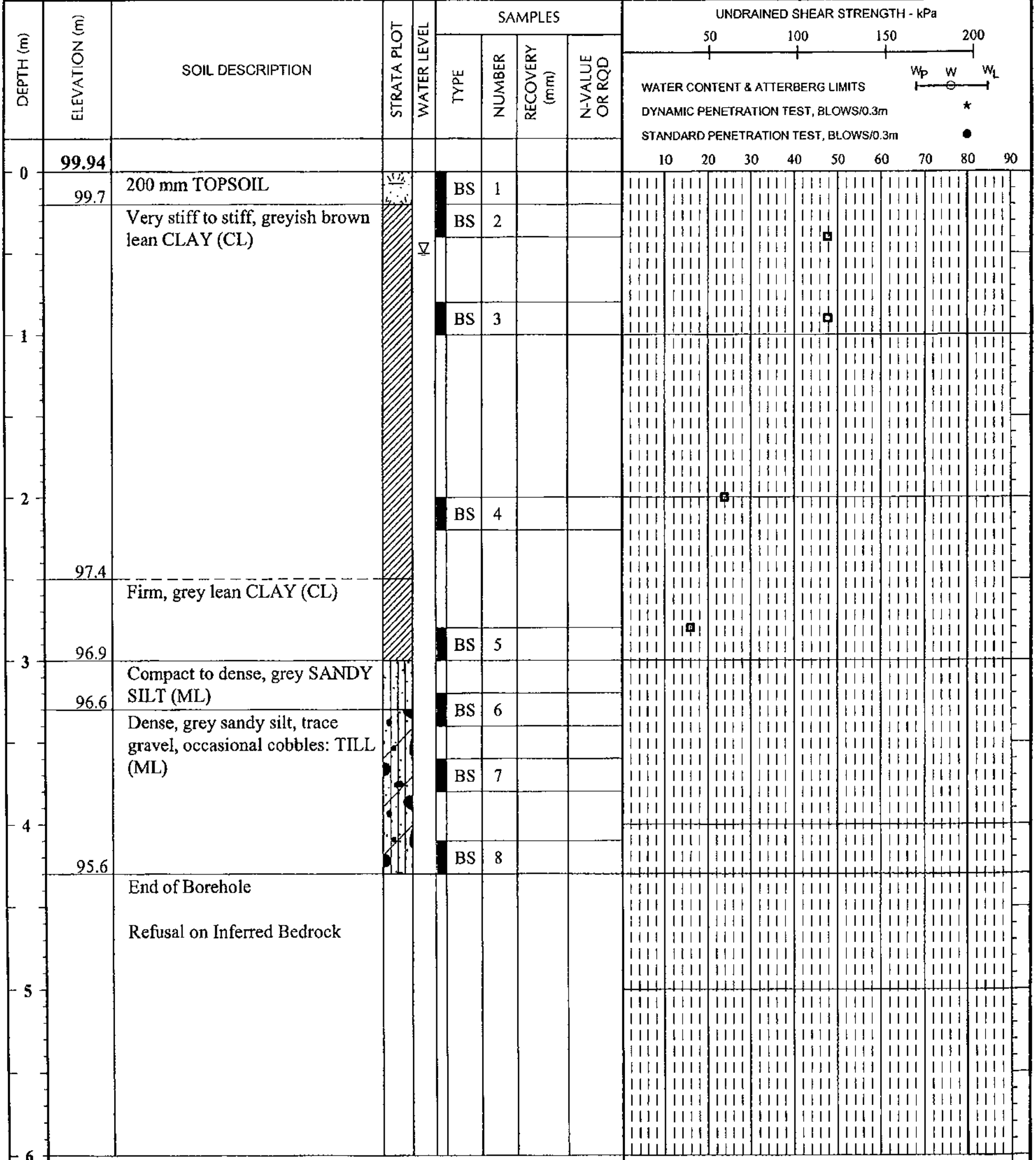
TEST PIT RECORD

TP07-24

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-24
PROJECT No. 1026929
DATUM Local

ELEVATION (m)	SOIL DESCRIPTION
99.94	
99.7	200 mm TOPSOIL
97.4	Very stiff to stiff, greyish brown lean CLAY (CL)
96.9	Firm, grey lean CLAY (CL)
96.6	Compact to dense, grey SANDY SILT (ML)
95.6	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)
	End of Borehole Refusal on Inferred Bedrock

SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
BS	1		
BS	2		
BS	3		
BS	4		
BS	5		
BS	6		
BS	7		
BS	8		



⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

Jacques Whitford

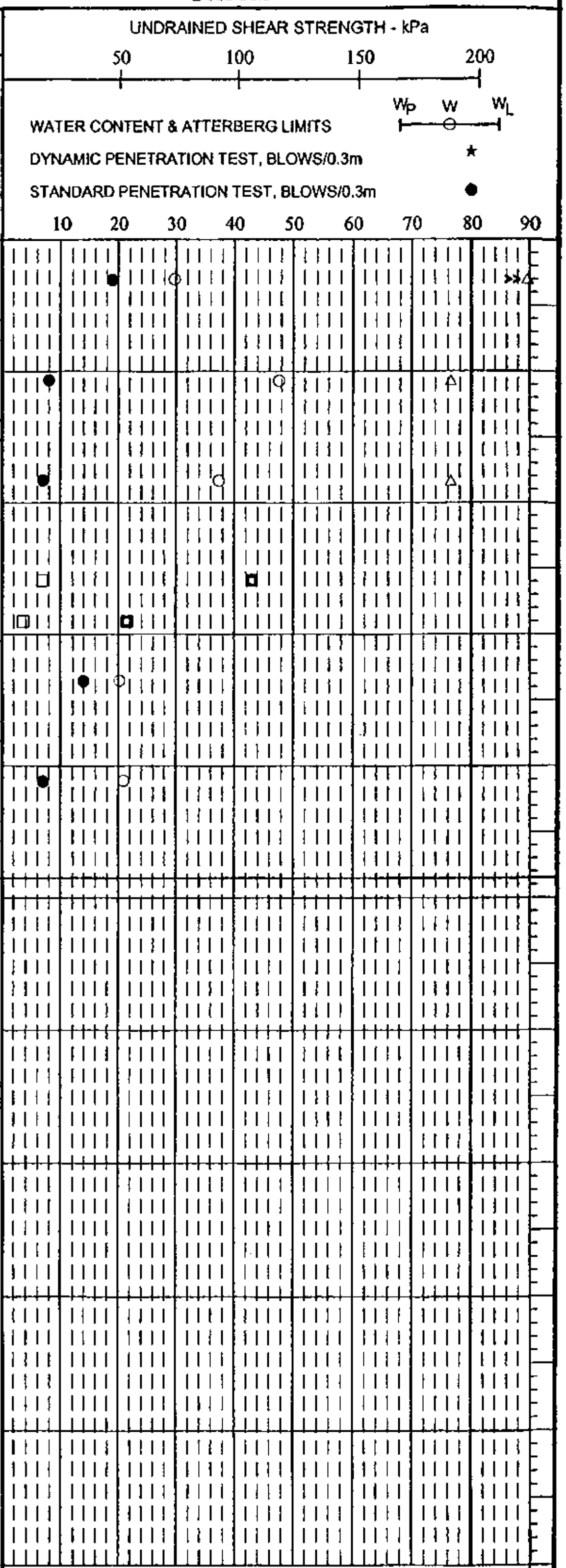
MONITORING WELL RECORD

MW07-25

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. MW07-25
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	98.72							
	98.5	200 mm TOPSOIL						
		Firm to stiff, greyish brown lean CLAY (CL)			SS	1	320	19
1				▼	SS	2	610	8
					SS	3	610	7
2								
3	95.7			∇				
		Loose to compact, SANDY SILT (ML)			SS	4	610	14
4					SS	5	450	7
	93.9				SS	6	150	50/ 50 mm
5		End of Borehole Auger Refusal on Inferred Bedrock Monitoring Well Installed						
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

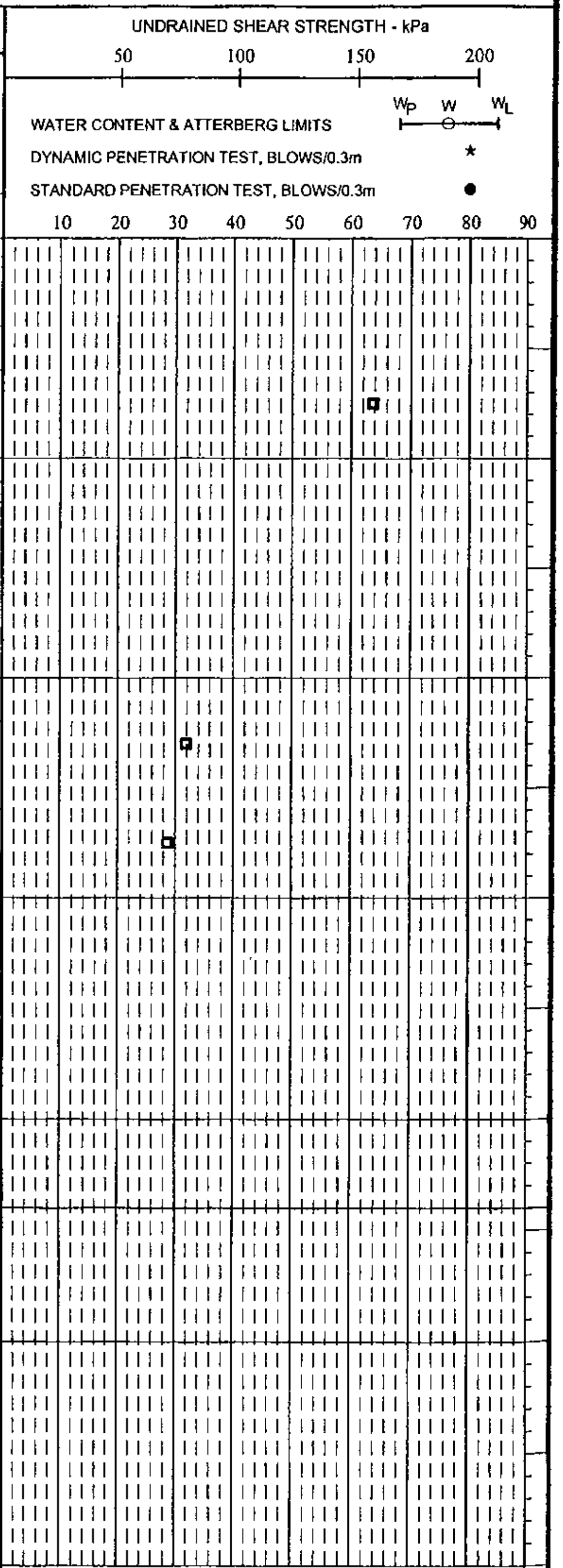
App'd
Date

TEST PIT RECORD

TP07-26

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-26
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.46							
	99.3	200 mm TOPSOIL						
		Very stiff to stiff, greyish brown lean CLAY (CL)			BS	1		
1				∇	BS	2		
2					BS	3		
					BS	4		
	96.6							
3		Dense, grey SANDY SILT (ML)			BS	5		
4	95.5							
		Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
	95.1	End of Borehole Refusal on Inferred Bedrock						
5								
6								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

BOREHOLE RECORD

BH07-27

CLIENT Mattamy Homes

BOREHOLE No. BH07-27

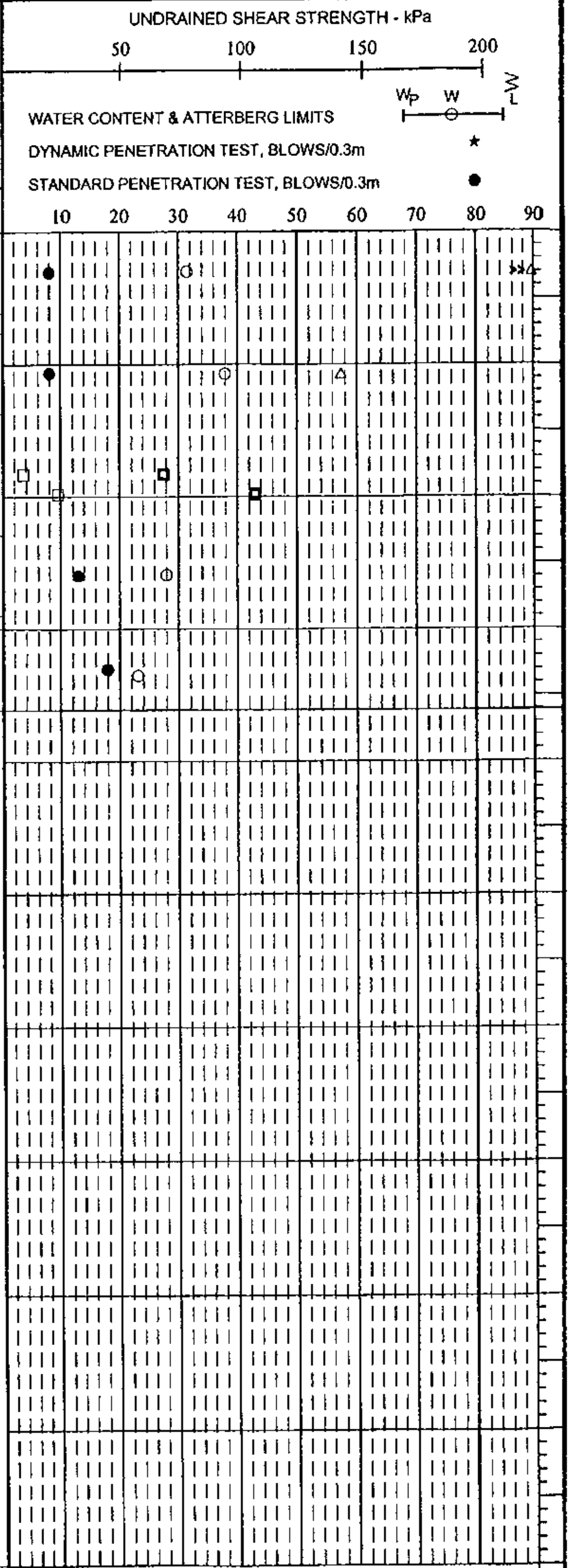
LOCATION Proposed Subdivision, Richmond, ON

PROJECT No. 1026929

DATES: BORING June 15, 2007 WATER LEVEL

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.64	Stiff to very stiff, greyish brown lean CLAY (CL)	▨		SS	1	280	8
1			▨		SS	2	30	8
2	97.3		▨	∇				
3		Compact, brown SANDY SILT (ML)			SS	3	320	13
	96.0				SS	4	350	18
4		End of Borehole Auger Refusal on Inferred Bedrock						
5								
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

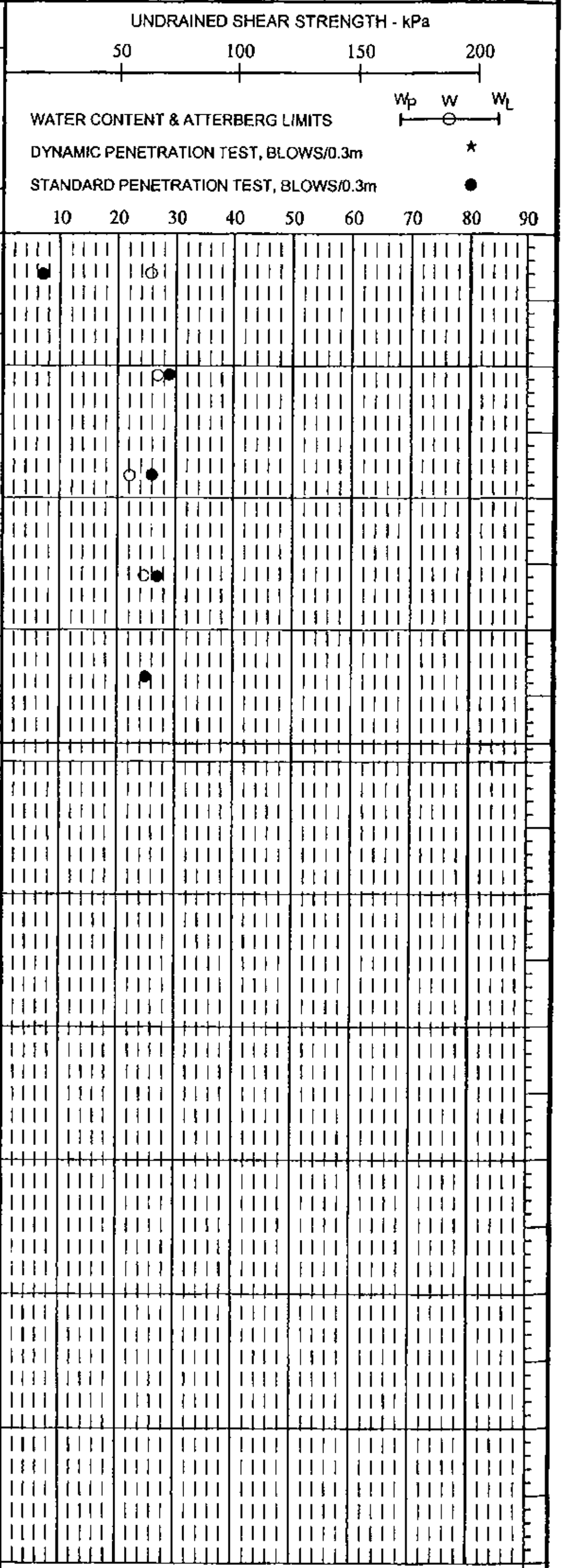
Jacques Whitford

BOREHOLE RECORD

BH07-28

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. BH07-28
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.04							
	99.9	100 mm TOPSOIL						
		Compact, greyish brown SANDY SILT (ML)			SS	1	150	7
1					SS	2	420	29
2				∇	SS	3	520	26
	97.8	Compact, brown SILTY SAND (SM)			SS	4	500	27
3	97.0	Compact, grey SILTY SAND (SM)			SS	5	420	25
	96.2				SS		100	50/ 0 mm
4		End of Borehole Auger Refusal on Inferred Bedrock						
5								
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

1 of 1

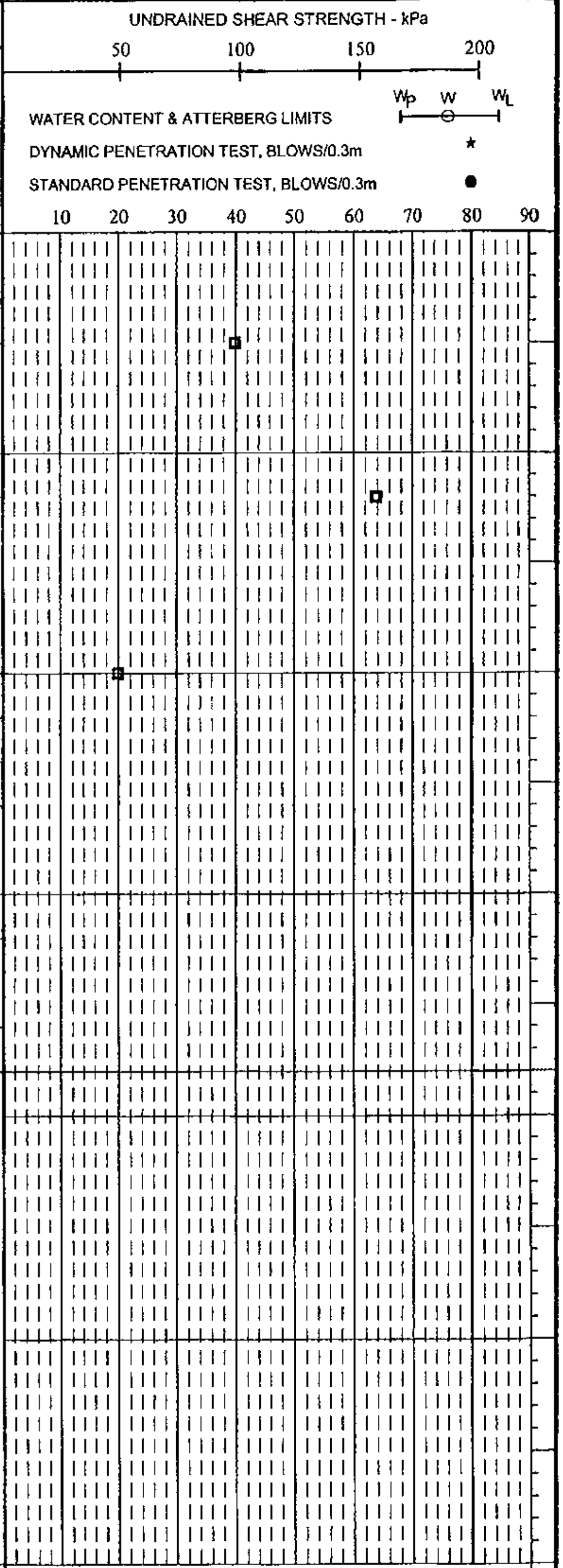
Jacques Whitford

TEST PIT RECORD

TP07-29

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-29
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES			
					TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.04							
	98.8	200 mm TOPSOIL						
		Very stiff to stiff, greyish brown lean CLAY (CL)		∇	BS	1		
1					BS	2		
2					BS	3		
	96.8	Dense, grey SANDY SILT (ML)			BS	4		
3	96.0	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	5		
					BS	6		
	95.2	End of Borehole Refusal on Inferred Bedrock						
4								
5								
6								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ________
Date ________

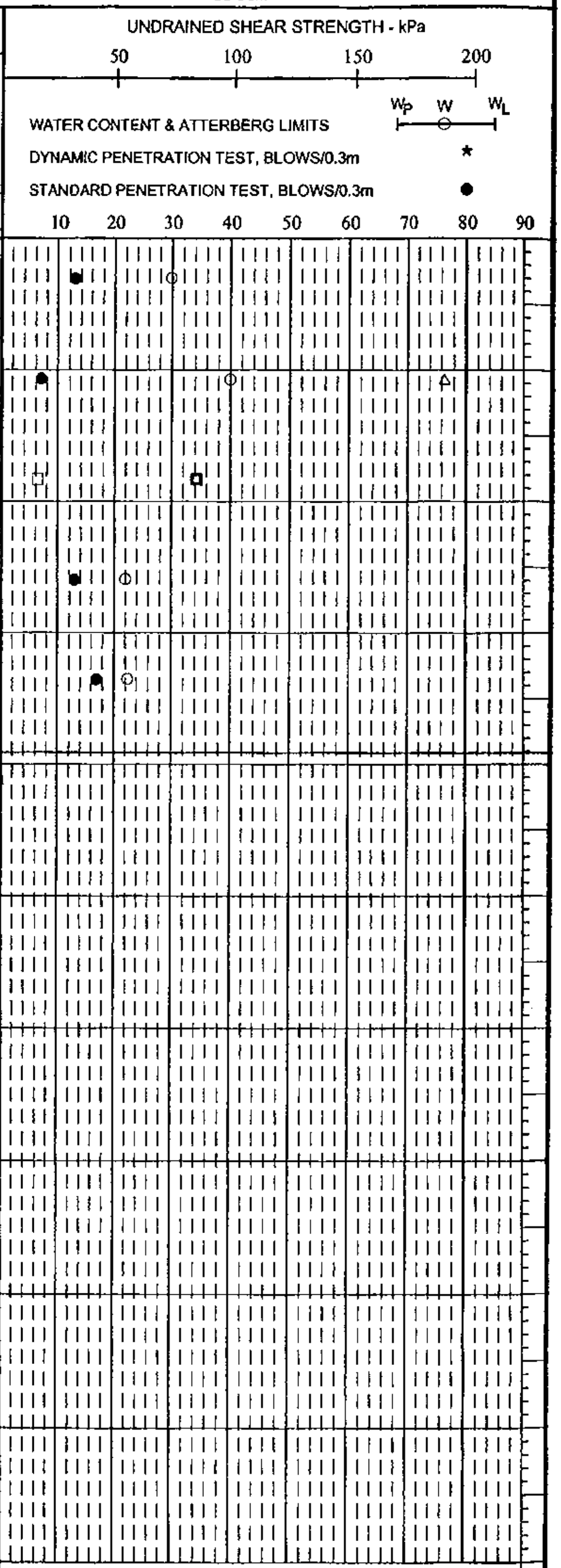
Jacques Whitford

MONITORING WELL RECORD

MW07-30

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. MW07-30
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.52							
	99.4	150 mm TOPSOIL						
		Firm to stiff, greyish brown lean CLAY (CL)			SS	1	100	13
1				▼	SS	2	500	7
2	97.5	Compact, brown to grey SANDY SILT (ML)						
					SS	3	320	13
3				▽	SS	4	420	17
4	95.6	End of Borehole Auger Refusal on Inferred Bedrock Monitoring Well Installed			SS	5	150	50/ 100 mm
5								
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-31

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

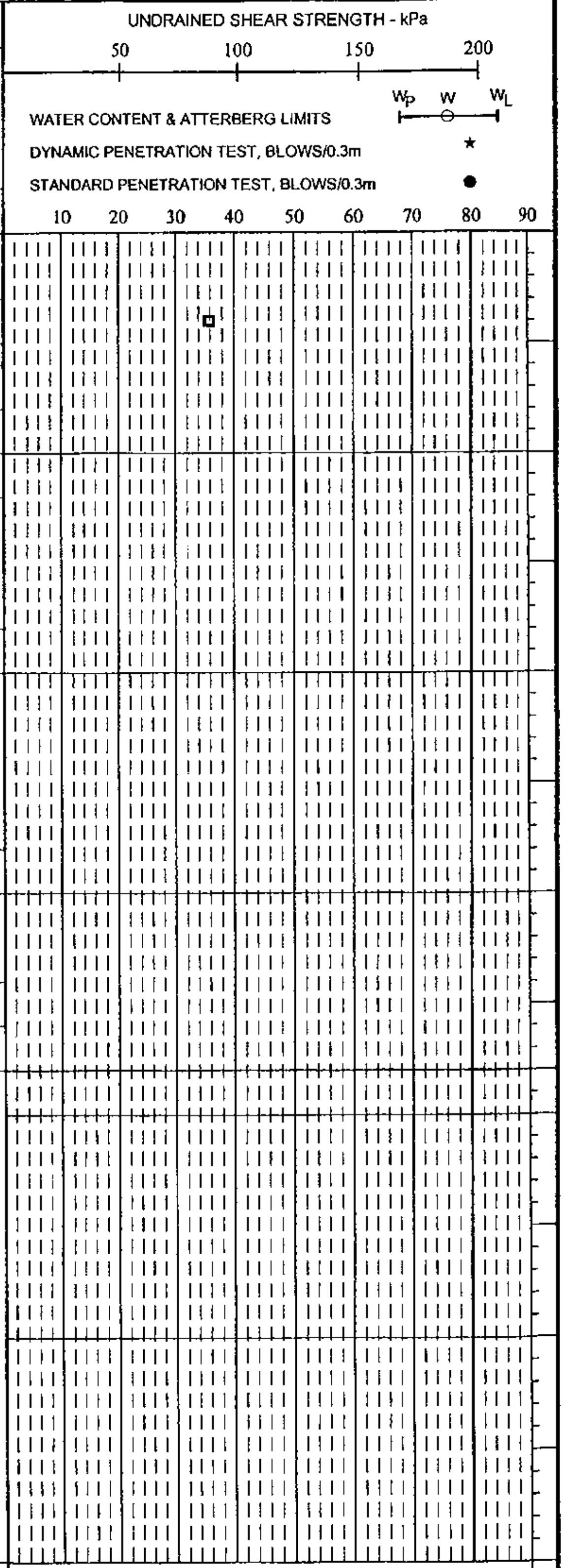
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-31

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.82	200 mm TOPSOIL						
	99.6	Stiff, greyish brown lean CLAY (CL)						
	99.4	Dense, grey SANDY SILT (ML)			BS	1		
					BS	2		
				⊽				
1					BS	3		
2					BS	4		
3	96.8	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	5		
					BS	6		
4	96.0	End of Borehole Refusal on Inferred Bedrock						
5								
6								



⊽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

1 of 1

TEST PIT RECORD

TP07-32

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

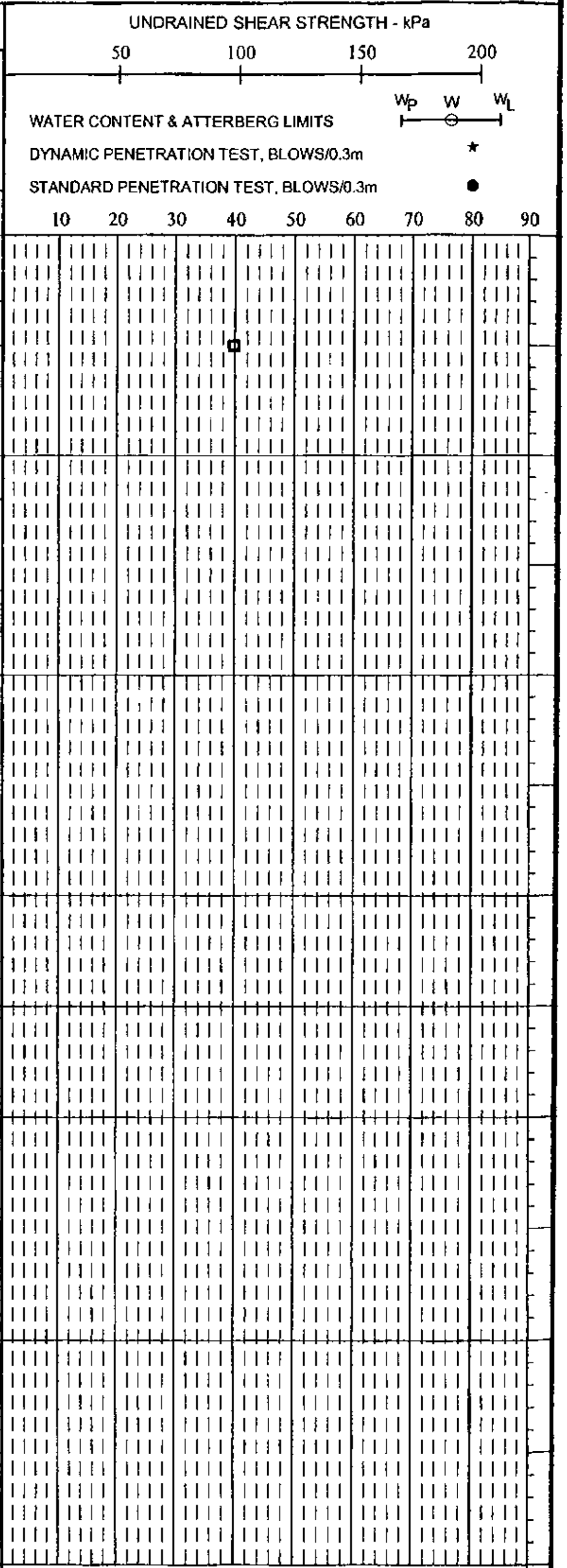
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-32

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**99.92**							
	99.8	150 mm TOPSOIL						
	98.1	Stiff, greyish brown lean CLAY (CL)		∇	BS BS	1 2		
	96.9	Compact to dense, grey SANDY SILT (ML)			BS BS BS	3 4 5		
	96.4	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
		End of Borehole Refusal on Inferred Bedrock						



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

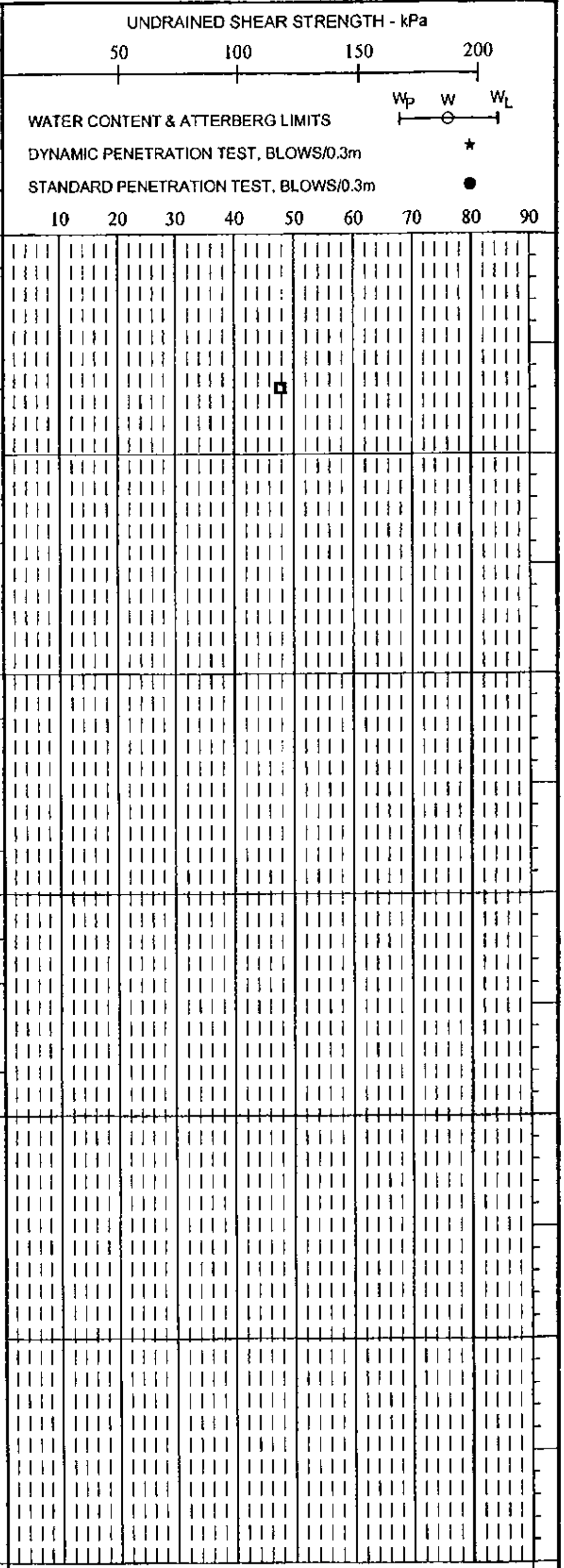
1 of 1

TEST PIT RECORD

TP07-33

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-33
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.45						
	99.3	150 mm TOPSOIL		BS	1		
		Stiff, brown lean CLAY (CL)		BS	2		
1				BS	3		
	98.0	Dense, brown SANDY SILT (ML)					
2				BS	4		
			∇				
3				BS	5		
	96.2	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)		BS	6		
				BS	7		
4	95.4	End of Borehole Refusal on Inferred Bedrock					
5							
6							



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-34

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

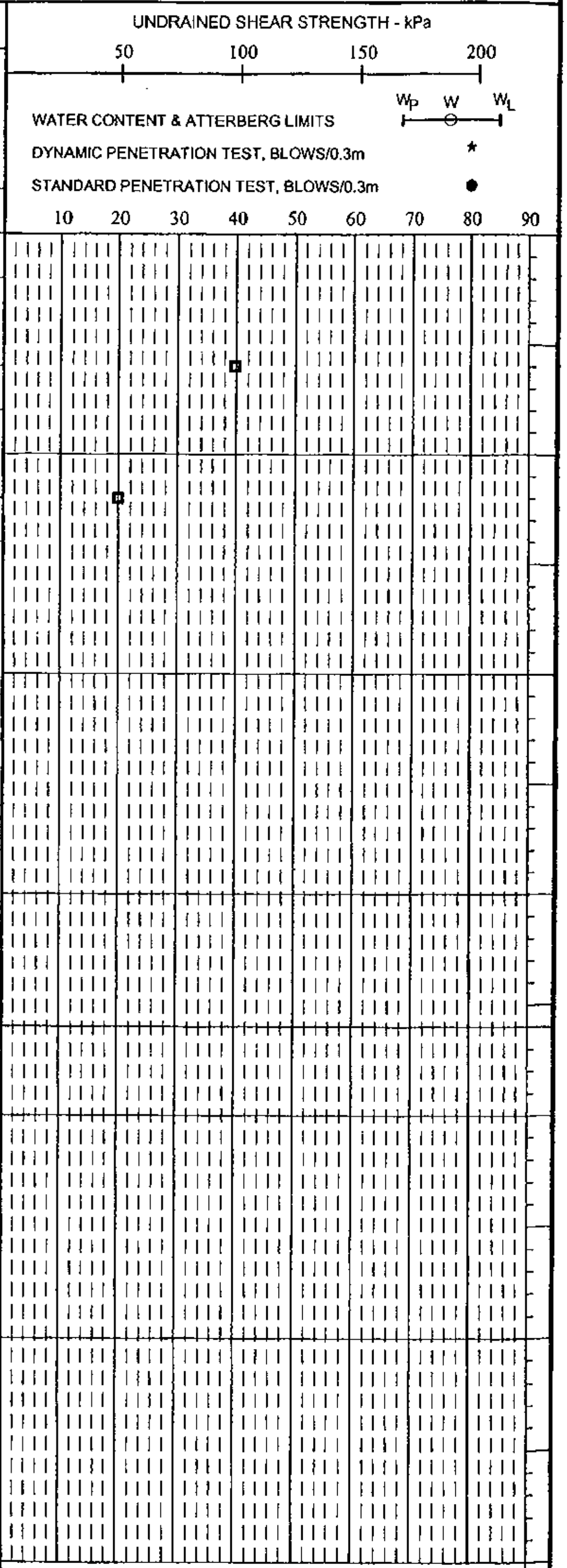
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-34

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.56	250 mm TOPSOIL		BS	1		
	99.3	Stiff, brown lean CLAY (CL)		BS	2		
1			∇	BS	3		
2	97.6	Dense, brown to grey SANDY SILT (ML)		BS	4		
				BS	5		
3	96.3	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)		BS	6		
	96.0			BS	7		
		End of Borehole Refusal on Inferred Bedrock					
4							
5							
6							



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

Jacques Whitford

TEST PIT RECORD

1 of 1

TP07-36

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

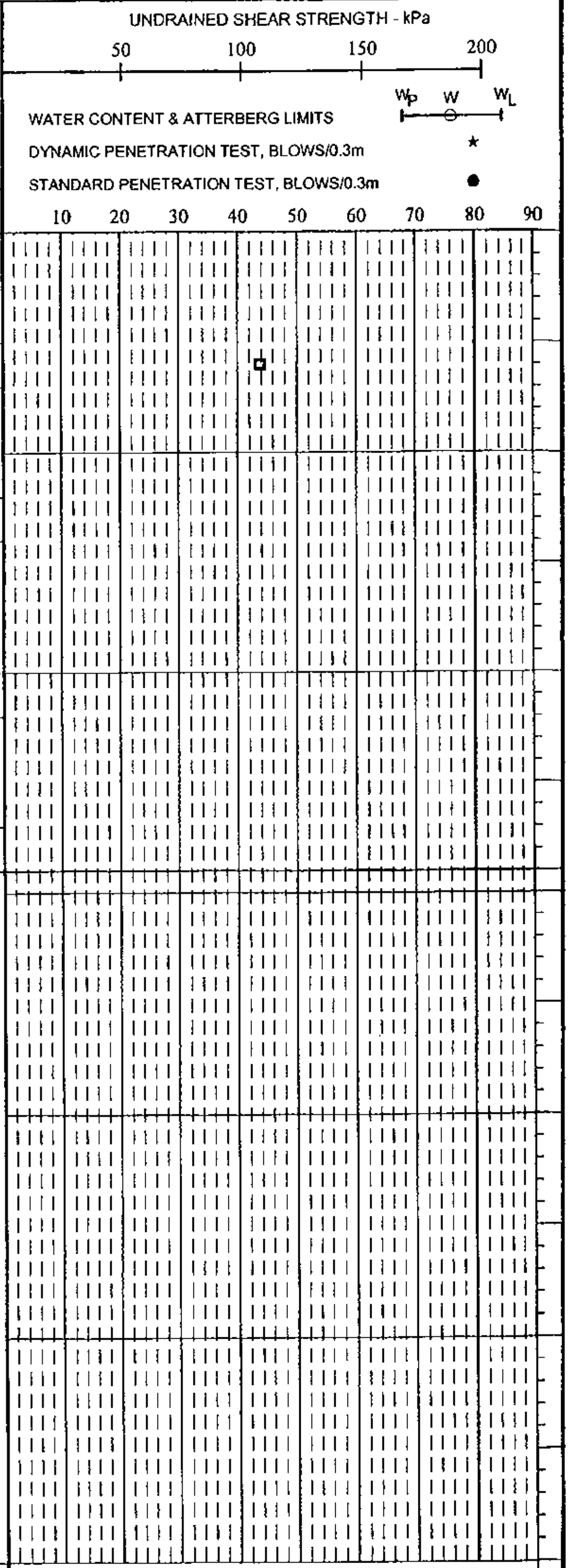
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-36

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.16							
	100.0	200 mm TOPSOIL			BS	1		
		Stiff, brown lean CLAY (CL)			BS	2		
1	99.1	Dense, brown SANDY SILT (ML)		∇	BS	3		
2					BS	4		
					BS	5		
3	97.3	End of Borehole Refusal on Inferred Bedrock						
4								
5								
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

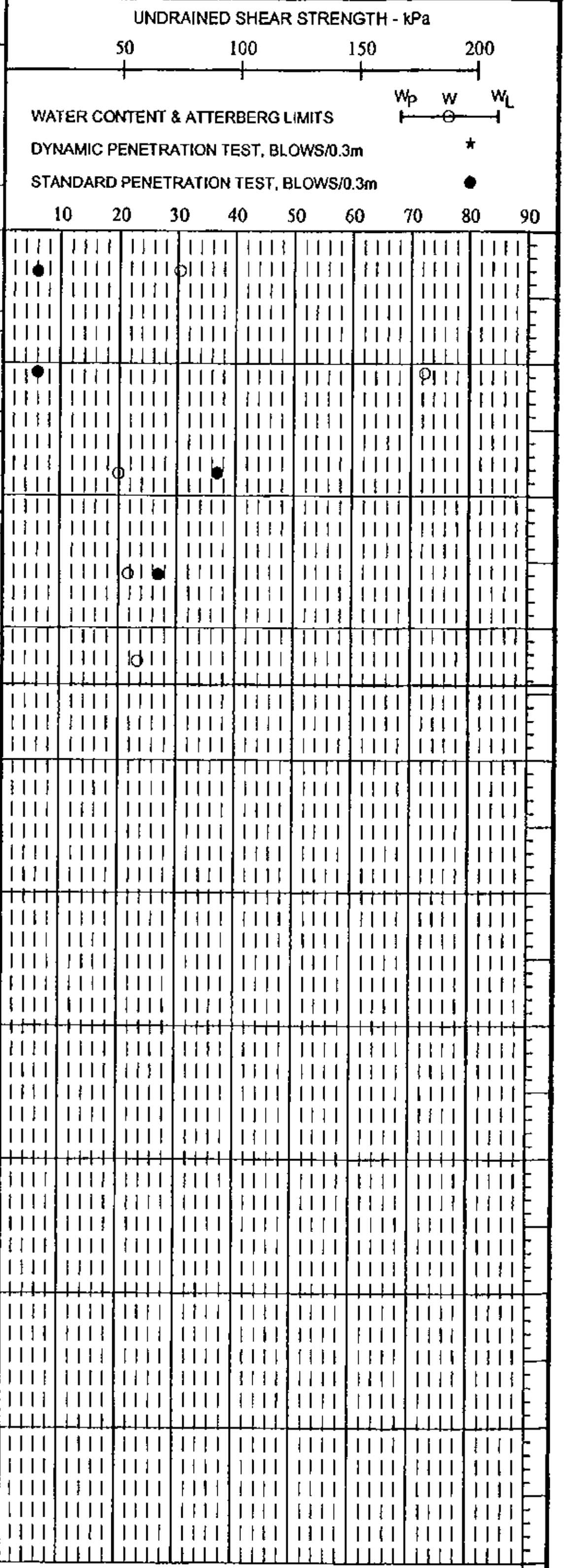
1 of 1

MONITORING WELL RECORD

MW07-37

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. MW07-37
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.62							
	99.5	75 mm TOPSOIL						
		Firm, greyish brown lean CLAY (CL)			SS	1	150	6
1	98.4			▼	SS	2	300	6
2		Compact to dense, greyish brown SANDY SILT (ML)		▽	SS	3	350	37
					SS	4	320	27
3	96.4				SS	5	280	50/ 75 mm
	96.2	Compact to dense, grey SANDY SILT (ML)						
4		End of Borehole Auger Refusal on Inferred Bedrock Monitoring Well Installed						
5								
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

JWL-OLD 1026929.GPJ SMART.GDT 07/06/22

Jacques Whitford

TEST PIT RECORD

TP07-38

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

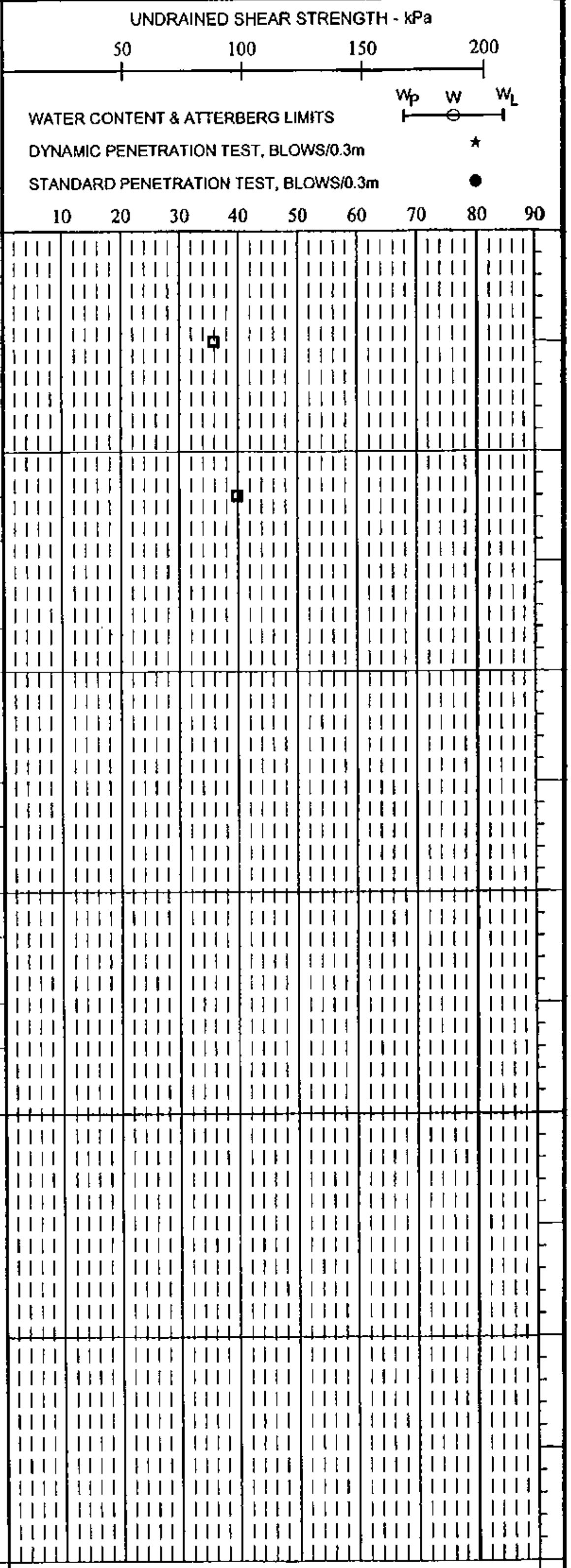
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-38

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.75	200 mm TOPSOIL			BS	1		
	99.6	Stiff, brown lean CLAY (CL)			BS	2		
1					BS	3		
	98.2	Dense, brown SANDY SILT (ML)			BS	4		
2					BS	5		
3					BS	6		
	96.4	Dense, brown sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	7		
4	95.8	End of Borehole						
5								
6								



∇ Inferred Groundwater Level

$\blacktriangledown$ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

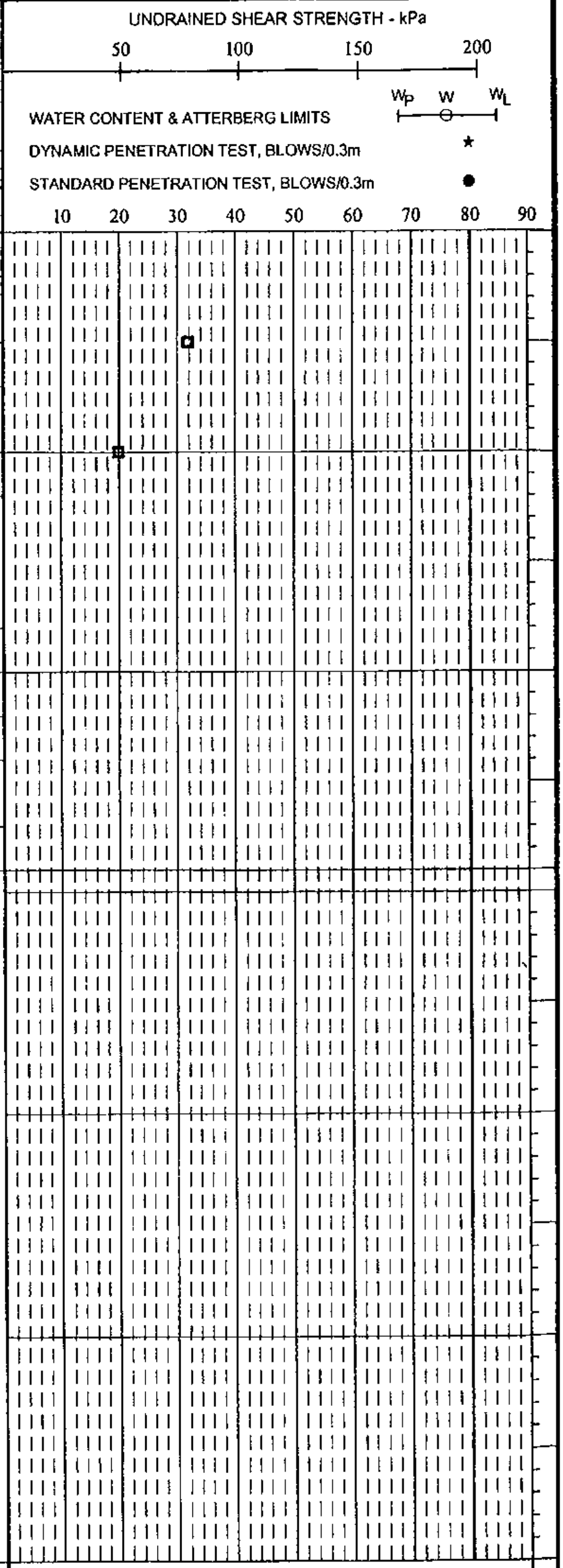
Jacques Whitford

TEST PIT RECORD

TP07-39

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-39
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	99.83	220 mm TOPSOIL			BS	1		
	99.6	Stiff, brown lean CLAY (CL)			BS	2		
1				▽	BS	3		
2	98.1	Dense, brown SANDY SILT (ML)			BS	4		
					BS	5		
3	96.9	End of Borehole Refusal on Inferred Bedrock			BS	6		
4								
5								
6								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

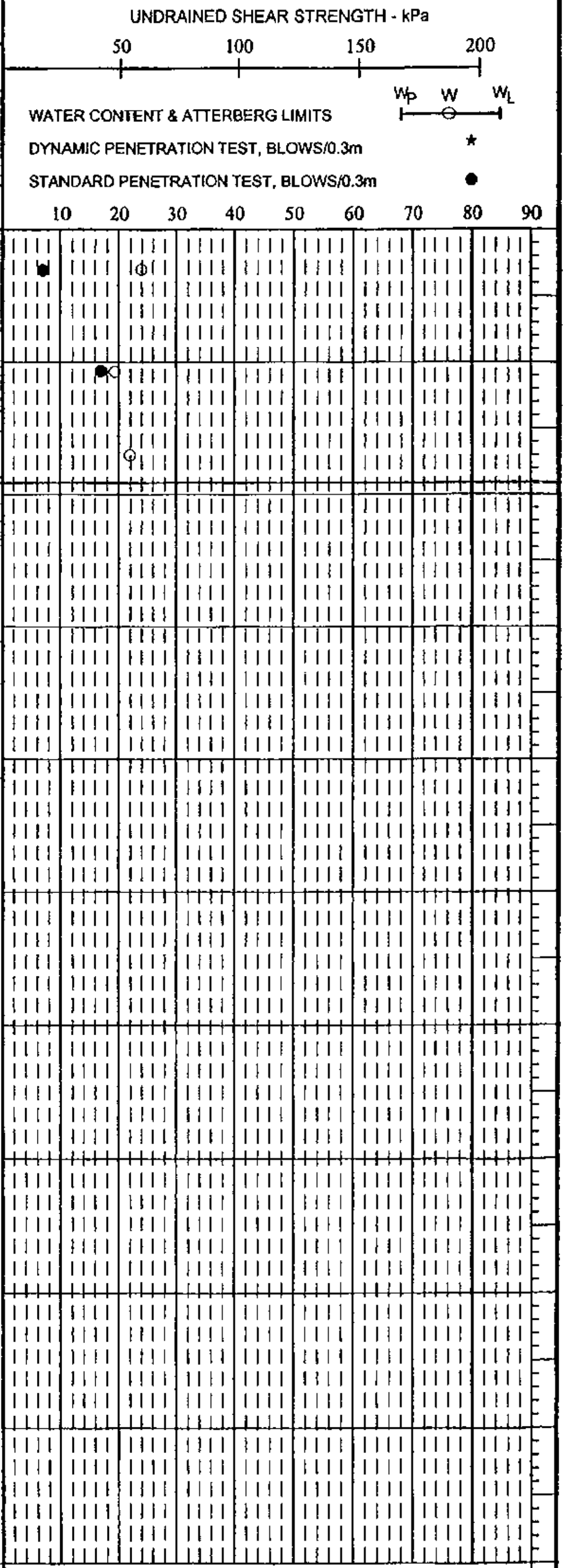
Jacques Whitford

MONITORING WELL RECORD

MW07-40

CLIENT Mattamy Homes
BOREHOLE No. MW07-40
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 15, 2007 WATER LEVEL
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.26 ~~100.1~~	150 mm TOPSOIL Compact to dense, brownish grey SANDY SILT (ML)			SS	1	320	7
1				∇	SS	2	420	17
	98.3				SS	3	300	50/ 50 mm
2		End of Borehole Auger Refusal on Inferred Bedrock Monitoring Well Installed						
3								
4								
5								
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-41

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-41

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**99.77**							
	99.6	200 mm TOPSOIL						
		Compact to dense, grey and brown SANDY SILT (ML)			BS	1		
				∇				
1					BS	2		
	98.0				BS	3		
2		Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	4		
	97.6	End of Borehole Refusal on Inferred Bedrock						
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-42

CLIENT Mattamy Homes
BOREHOLE No. TP07-42
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 14, 2007 WATER LEVEL
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**100.05**							
	99.8	200 mm TOPSOIL						
		Compact to dense, brown to grey SANDY SILT (ML)			BS	1		
				⊻				
1					BS	2		
2	98.0	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	3		
					BS	4		
3					BS	5		
	96.7	End of Borehole Refusal on Inferred Bedrock						
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

⊻ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

1 of 1

TEST PIT RECORD

TP07-43

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-43

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**100.16** 100.0	200 mm TOPSOIL		BS	1		
		Compact to dense, brown to grey SANDY SILT (ML)	▽	BS	2		
1				BS	3		
2				BS	4		
	97.5	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)		BS	5		
3	97.0			BS	6		
		End of Borehole					
4							
5							
6							

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-44

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-44

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**100.41**							
	100.2	200 mm TOPSOIL						
		Compact to dense, brown to grey SANDY SILT (ML)		⊻	BS	1		
1					BS	2		
					BS	3		
	98.8	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	4		
2	98.1				BS	5		
		End of Borehole Refusal on Inferred Bedrock						
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

⊻ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

Jacques Whitford

1 of 1

TEST PIT RECORD

TP07-45

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 15, 2007 WATER LEVEL June 20, 2007

BOREHOLE No. TP07-45

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	100.19							
	100.0	200 mm TOPSOIL						
		Compact to dense, brown to grey SANDY SILT (ML)		∇	BS	1		
1					BS	2		
				▼	BS	3		
2	98.3	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)						
					BS	4		
3	97.3	End of Borehole Refusal on Inferred Bedrock Standpipe Installed						
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-46

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

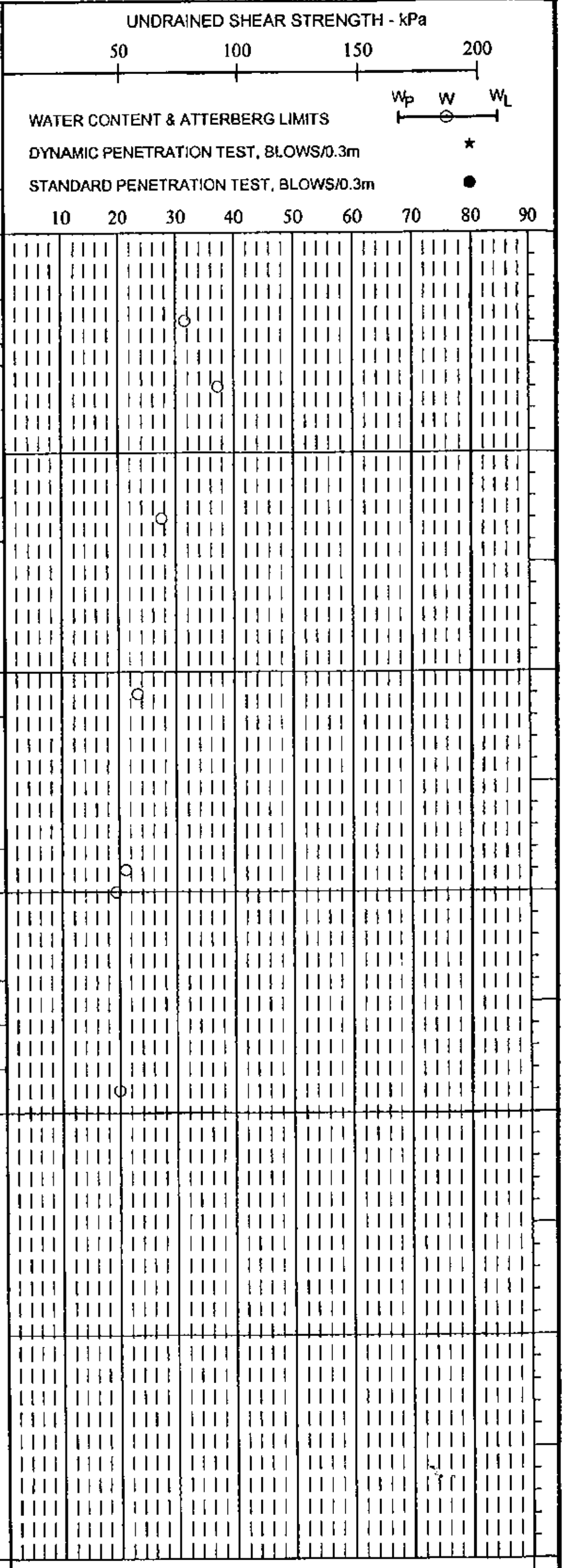
DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-46

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.16	300 mm TOPSOIL						
	99.9	Compact to dense, brown to grey SANDY SILT (ML)			BS	1		
				∇	BS	2		
1					BS	3		
2					BS	4		
3					BS	5		
	96.8	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
					BS	7		
4	96.2	End of Borehole Refusal on Inferred Bedrock						
5								
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-47

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007

BOREHOLE No. TP07-47
PROJECT No. 1026929
DATUM Local

UNDRAINED SHEAR STRENGTH - kPa: 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**100.24**						
	100.0	200 mm TOPSOIL		BS	1		
	99.6	Stiff, brown lean CLAY (CL)	▼	BS	2		
1		Compact to dense, brown and grey SANDY SILT (ML)		BS	3		
				BS	4		
2				BS	5		
3	97.0			BS	6		
		Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)		BS	7		
4				BS	8		
	95.7			BS	9		
		End of Borehole Standpipe Installed					
5							
6							

∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

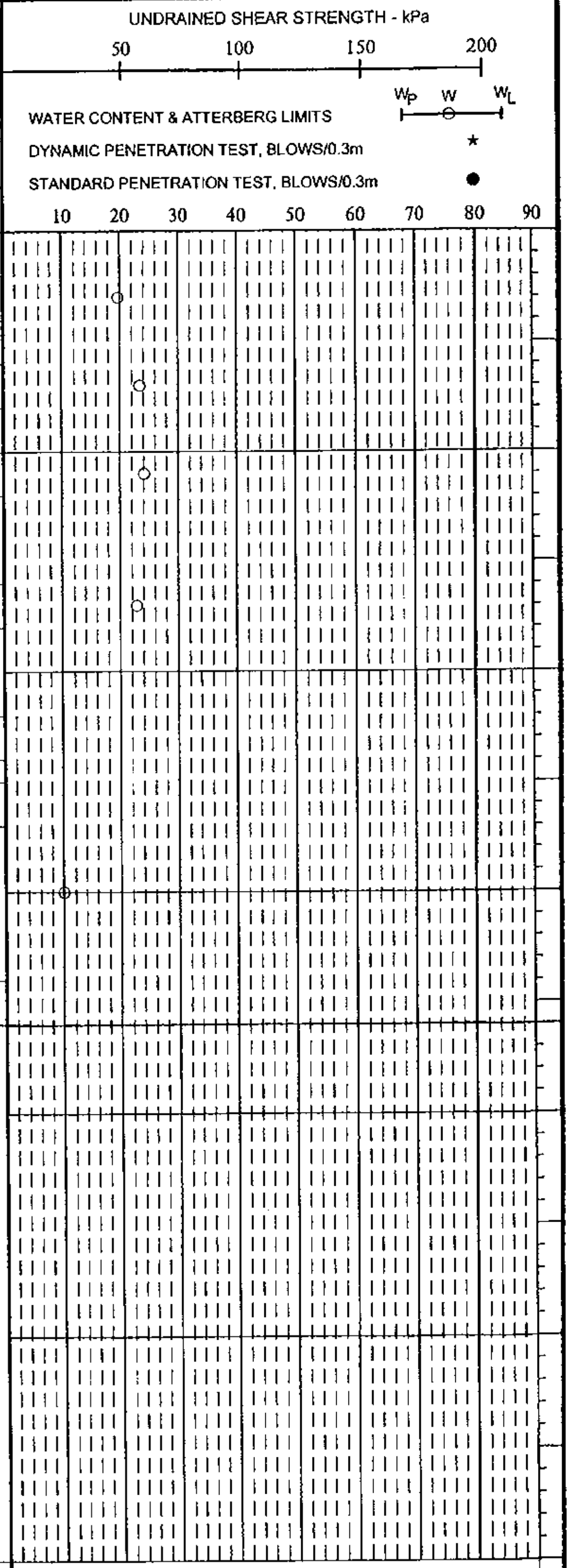
Jacques Whitford

TEST PIT RECORD

TP07-48

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-48
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.51							
	100.4	150 mm TOPSOIL						
		Compact to dense, brown and grey SANDY SILT (ML)		▽	BS	1		
					BS	2		
1					BS	3		
					BS	4		
2					BS	5		
	98.0	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
3					BS	7		
	96.9	End of Borehole Refusal on Inferred Bedrock						
4								
5								
6								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-49

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL
BOREHOLE No. TP07-49
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	100.67							
0	100.4	230 mm TOPSOIL						
		Compact to dense, brown and grey SANDY SILT (ML)			BS	1		
1					BS	2		
					BS	3		
2	98.7	End of Borehole Refusal on Inferred Bedrock						
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_p W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m *
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

1 of 1

TEST PIT RECORD

TP07-50

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-50

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.42	300 mm TOPSOIL			BS	1		
	100.1	Compact to dense, brown and grey SANDY SILT (ML)		∇	BS	2		
					BS	3		
1					BS	4		
2					BS	5		
					BS	6		
3					BS	7		
	96.9	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	8		
	96.6	End of Borehole Refusal on Inferred Bedrock						
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS Wp W WL

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

1 of 1

TEST PIT RECORD

TP07-51

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-51

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	100.34	300 mm TOPSOIL						
	100.0	Compact to dense, brown and grey SANDY SILT (ML)		⊻	BS	1		
					BS	2		
1					BS	3		
2					BS	4		
	97.5	Dense, grey SANDY SILT			BS	5		
3					BS	6		
					BS	7		
4	96.3	End of Borehole						
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

⊻ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-52

1 of 1

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-52

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.59	280 mm TOPSOIL			BS	1		
	100.3	Compact to dense, brown and grey SANDY SILT (ML)		∇	BS	2		
1					BS	3		
2					BS	4		
	98.1	Dense, grey SANDY SILT			BS	5		
3	97.4	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
4					BS	7		
					BS	8		
	96.1	End of Borehole						
5								
6								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

Jacques Whitford

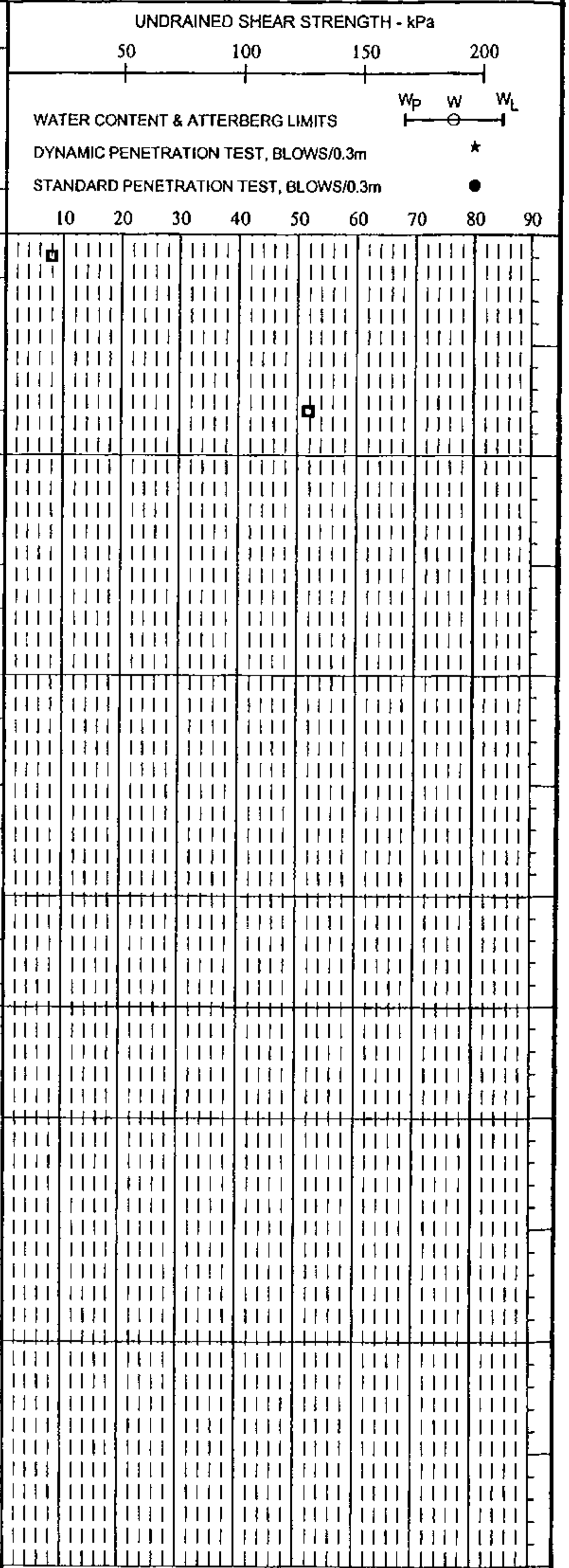
TEST PIT RECORD

TP07-53

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-53
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	100.94							
	100.7	200 mm TOPSOIL			BS	1		
		Firm, brown lean CLAY (CL)						
1					BS	2		
	99.4			∇				
		Compact to dense, brown and grey SANDY SILT (ML)			BS	3		
2					BS	4		
	98.5							
		Dense, grey sandy silt, trace gravel, occasional cobbles and boulders: TILL (ML)			BS	5		
3					BS	6		
	97.4							
		End of Borehole Refusal on Inferred Boulders						
4								
5								
6								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-54

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

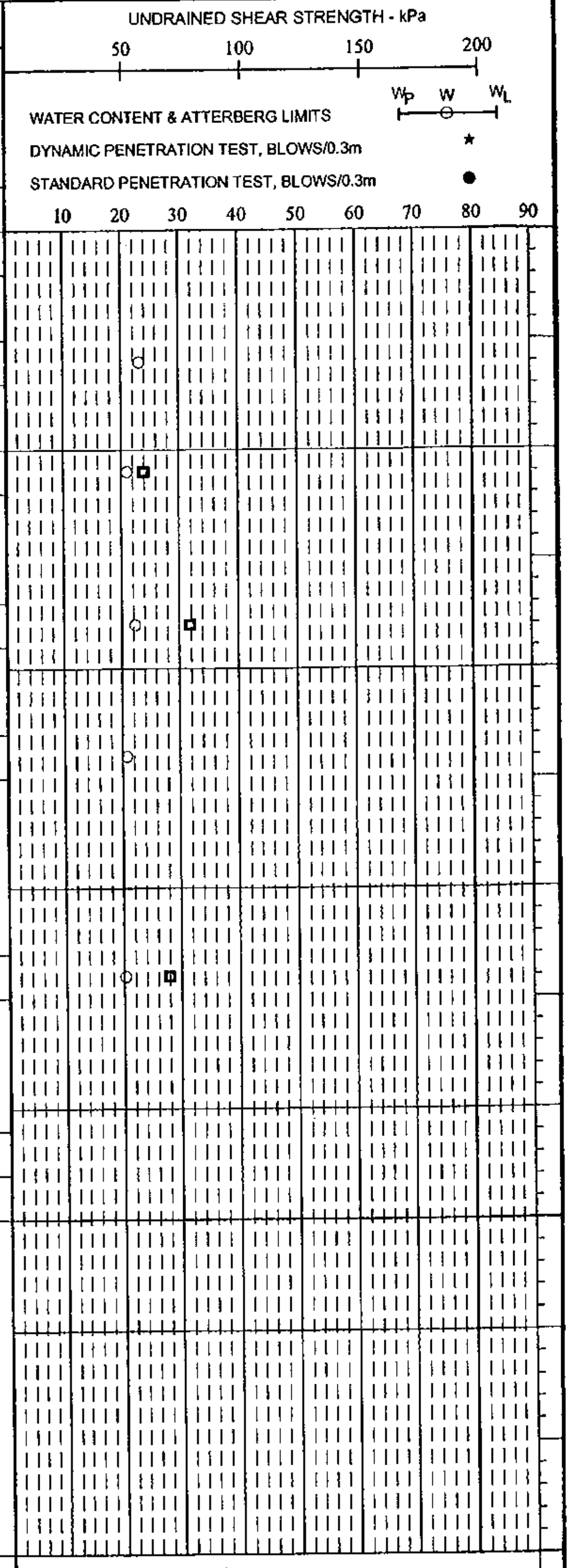
DATES: BORING June 15, 2007 WATER LEVEL

BOREHOLE No. TP07-54

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**101.37**							
	101.2	200 mm TOPSOIL			BS	1		
		Compact to dense, brown and grey SANDY SILT (ML)			BS	2		
1					BS	3		
				∇	BS	4		
2					BS	5		
3	98.4	Dense, grey SANDY SILT (ML)			BS	6		
4					BS	7		
	96.9	End of Borehole						
5								
6								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/22

Jacques Whitford

1 of 1

TEST PIT RECORD

TP07-55

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-55

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	101.07							
0	100.9	180 mm TOPSOIL			BS	1		
		Compact to dense, brown and grey SANDY SILT (ML)			BS	2		
					BS	3		
1				▽	BS	4		
2	99.1	Dense, grey SANDY SILT (ML)			BS	5		
	98.3	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
3					BS	7		
4	97.0	End of Borehole Refusal on Inferred Bedrock Standpipe Installed						
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-56

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL
BOREHOLE No. TP07-56
PROJECT No. 1026929
DATUM Local

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200
WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m *
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**101.21**	200 mm TOPSOIL			BS	1		
	101.0	Compact to dense, brown and grey SANDY SILT (ML)		⊻	BS	2		
1					BS	3		
2	99.2	Dense, grey SANDY SILT (ML)						
					BS	4		
3					BS	5		
	98.0	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	6		
	97.6	End of Borehole						
4								
5								
6								

⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

1 of 1

TP07-57

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-57
PROJECT No. 1026929
DATUM Local

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	102.95							
0	102.7	200 mm TOPSOIL			BS	1		
		End of Borehole Refusal on Inferred Bedrock						
1								
2								
3								
4								
5								
6								

$\underline{\nabla}$ Inferred Groundwater Level
$\blacktriangledown$ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-58

1 of 1

CLIENT Mattamy Homes
BOREHOLE No. TP07-58
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 15, 2007 WATER LEVEL
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	101.39							
	101.2	200 mm TOPSOIL			BS	1		
	100.9	Compact to dense, brown and grey SILTY SAND (SM)			BS	2		
	100.5	Dense, grey sandy silt, trace gravel: TILL (ML)			BS	3		
1		End of Borehole Refusal on Inferred Bedrock						
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

$\underline{\nabla}$ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date

1 of 1

Jacques Whitford

TEST PIT RECORD

TP07-59

CLIENT Mattamy Homes
BOREHOLE No. TP07-59
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 14, 2007 WATER LEVEL
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**101.13**							
	100.9	200 mm TOPSOIL			BS	1		
	100.6	Compact to dense, brown SANDY SILT (ML)			BS	2		
		End of Borehole Refusal on Inferred Bedrock						
1								
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_p W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m *
STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-60

CLIENT Mattamy Homes
BOREHOLE No. TP07-60
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 14, 2007 WATER LEVEL
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**101.36**							
	101.2	200 mm TOPSOIL			BS	1		
	100.5	Compact to dense, brown and grey SANDY SILT (ML)			BS	2		
1		Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	3		
					BS	4		
	99.5				BS	5		
2		End of Borehole Refusal on Inferred Bedrock						
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m *
STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-61

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-61
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	102.15	230 mm TOPSOIL			BS	1		
	101.9							
	101.8	Compact to dense, brown SILTY SAND (SM)			BS	2		
	101.5	Fractured Bedrock						
		End of Borehole						
1		Refusal on Inferred Bedrock						
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m *
STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-62

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-62
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	101.68							
	101.4	200 mm TOPSOIL			BS	1		
	101.2	Compact to dense, brown and grey SANDY SILT (ML)			BS	2		
	101.0	Dense, brown sandy silt, trace gravel, occasional cobbles: TILL (ML)			BS	3		
1		End of Borehole Refusal on Inferred Bedrock						
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200
WATER CONTENT & ATTERBERG LIMITS Wp W WL
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-63

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL
BOREHOLE No. TP07-63
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	101.44							
	101.2	200 mm TOPSOIL			BS	1		
	101.0	Compact to dense, brown SANDY SILT (ML)			BS	2		
		End of Borehole Refusal on Inferred Bedrock						
1								
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level
$\blacktriangledown$ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa App'd
△ Pocket Penetrometer Test, kPa Date

Jacques Whitford

TEST PIT RECORD

TP07-64

CLIENT Mattamy Homes

LOCATION Proposed Subdivision, Richmond, ON

DATES: BORING June 14, 2007 WATER LEVEL

BOREHOLE No. TP07-64

PROJECT No. 1026929

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	101.32	230 mm TOPSOIL			BS	1		
	101.1	Compact to dense, brown SILTY SAND (SM)			BS	2		
1	100.2	Dense, grey SILTY SAND (SM)		⊻	BS	3		
2					BS	4		
	98.9	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (SM)			BS	5		
3					BS	6		
	97.8	End of Borehole Refusal on Inferred Bedrock						
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

⊻ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-65

1 of 1

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 15, 2007 WATER LEVEL
BOREHOLE No. TP07-65
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	100.15							
0		250 mm TOPSOIL			BS	1		
	99.9	Compact to dense, brown SILTY SAND (SM)			BS	2		
1	99.3	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (SM)		∇	BS	3		
					BS	4		
2					BS	5		
3	97.2	End of Borehole Refusal on Inferred Bedrock or Boulders						
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ______
Date ______

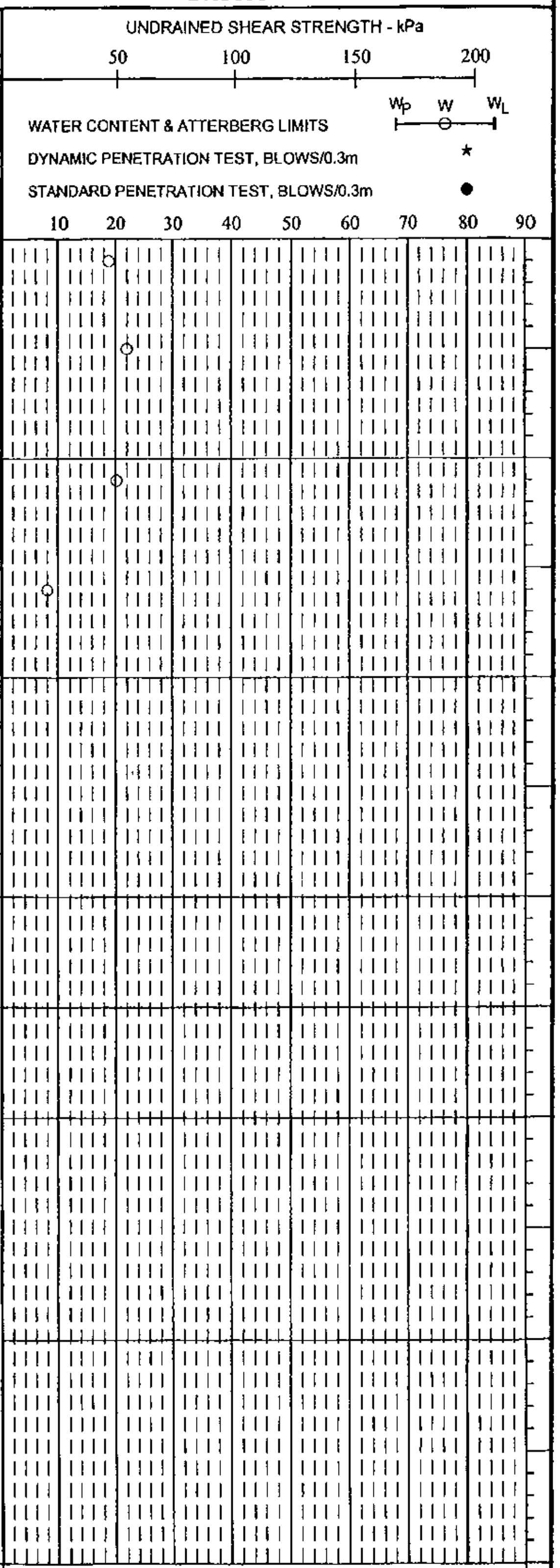
Jacques Whitford

TEST PIT RECORD

TP07-66

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. TP07-66
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	100.48					
	100.4	100 mm TOPSOIL	BS	1		
		Compact to dense, brown to grey SILTY SAND (SM)	BS	2		
1			BS	3		
	98.9	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (SM)	BS	4		
2			BS	5		
3			BS	6		
	97.0	End of Borehole Refusal on Inferred Boulders Standpipe Installed				
4						
5						
6						



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd ________
Date ________

Jacques Whitford

TEST PIT RECORD

1 of 1

TP07-67

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL
BOREHOLE No. TP07-67
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**101.50**							
	101.3	200 mm TOPSOIL			BS	1		
		Compact, brown SILTY SAND (SM)		∇	BS	2		
1					BS	3		
	100.2	Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (SM)						
	99.9				BS	4		
		End of Borehole Refusal on Inferred Bedrock						
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa: 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m *

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-68

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 20, 2007 WATER LEVEL
BOREHOLE No. TP07-68
PROJECT No. 1026929
DATUM Local

UNDRAINED SHEAR STRENGTH - kPa: 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0		130 mm TOPSOIL			BS	1		
		Compact, brown SILTY SAND (SM)			BS	2		
		Compact to dense, brown and grey SANDY SILT (ML)			BS	3		
					BS	4		
1		End of Hand Auger Hole Hand Auger Refusal						
2								
3								
4								
5								
6								

⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-69

1 of 1

CLIENT Mattamy Homes
BOREHOLE No. TP07-69
LOCATION Proposed Subdivision, Richmond, ON
PROJECT No. 1026929
DATES: BORING June 20, 2007 WATER LEVEL
DATUM Local

UNDRAINED SHEAR STRENGTH - kPa: 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	SAMPLES: NUMBER	SAMPLES: RECOVERY (mm)	SAMPLES: N-VALUE OR RQD
0		150 mm TOPSOIL			BS	1		
		Compact, brown silty sand, trace gravel, occasional cobbles: TILL			BS	2		
		End of Hand Auger Hole Hand Auger Refusal						
1								
2								
3								
4								
5								
6								

⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

Jacques Whitford

TEST PIT RECORD

TP07-70

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 20, 2007 WATER LEVEL
BOREHOLE No. TP07-70
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0		30 mm TOPSOIL Compact, brown SILTY SAND (SM)						
		End of Hand Auger Hole Hand Auger Refusal						
1								
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

JWL-OLD 1026929.GPJ SMART.GDT 07/06/21

Jacques Whitford

TEST PIT RECORD

TP07-71

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL
BOREHOLE No. TP07-71
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	102.71	230 mm TOPSOIL			BS	1		
	102.5	Fractured Bedrock						
	102.1	End of Borehole Refusal on Inferred Bedrock						
1								
2								
3								
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

$\underline{\nabla}$ Inferred Groundwater Level
$\blacktriangledown$ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

TEST PIT RECORD

TP07-72

CLIENT Mattamy Homes
LOCATION Proposed Subdivision, Richmond, ON
DATES: BORING June 14, 2007 WATER LEVEL June 20, 2007
BOREHOLE No. TP07-72
PROJECT No. 1026929
DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**102.44**							
	102.2	200 mm TOPSOIL			BS	1		
		Dense, grey sandy silt, trace gravel, occasional cobbles: TILL (SM)			BS	2		
1				▽				
				▼	BS	3		
2					BS	4		
					BS	5		
3	98.9							
		End of Borehole Refusal on Inferred Bedrock Standpipe Installed						
4								
5								
6								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date

APPENDIX C

Soil Grain Size Distribution Test Results

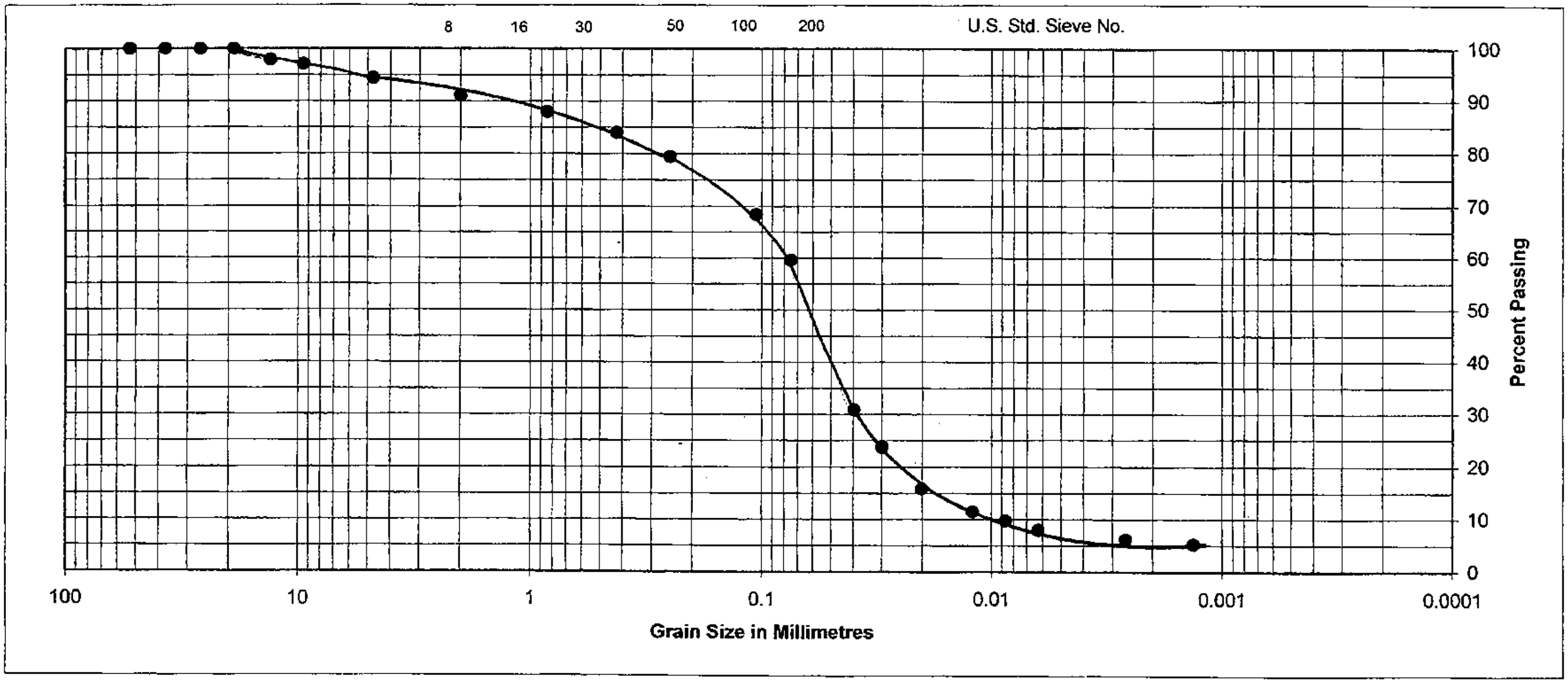
Jacques
Whitford

2781 Lancaster Rd. Tel: 613 738-0708
Ottawa, Ontario K1B 1A7 Fax: 613 738-0721

Hydrometer Analysis

Client: **Mattamy Homes**
Project: **Mattamy Homes, Rickmond**
Material Type:
Source: **BH07-18, SS-6**
Sampled From: **17.5' to 19.5'**
Sampled By: **Jeff Forrester**

Project No.: **1026929**
Test Method: **LS702**
Sample No.: **N/A**
Date Sampled: **14-Jun-07**
Tested By: **Blaine Miller**
Date Tested: **21-Jun-07**



Remarks:

Reviewed By: Brian Prevost
Date: June 22/2007

2781 Lancaster Rd.
Ottawa ON, K1B 1A7

Wash Sieve Analysis

Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	**LS 602 (ASTM C136)**
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**BH07-25**		
Sampled From :	**SS-5, 12.5' to 14.5'**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**14-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**20-Jun-07**

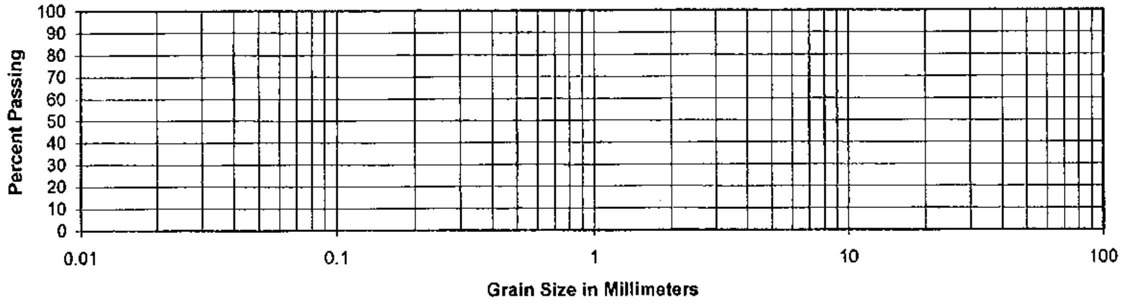
Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	266.5	
Sample Weight After Sieve :		Sample Weight After Wash :	37	**Corrected**
% Loss In Sieve :		% Passing No.200	86.1	N/A

Sieve Analysis

Sieve No.	Size of Opening		Wt. Retained	Cum Wt. Retained	% Passing	Specifications	
	Inches	mm	grams	grams		Min	Max
	3	76.2					
	2	53.0					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2					
	3/8	9.5					
+4	0.187	4.75		3.2	98.8		
		- 4.75		37.0			
8	0.0937	2.36					
16	0.0469	1.18					
30	0.234	0.600					
50	0.0117	0.300					
100	0.0059	0.150					
200	0.0029	0.075					
		Pan					

Classification of sample :	% Gravel :	1.2	% Sand :	12.7	% Silt and Clay :	86.1



Remarks :

Laboratory Supervisor :

Date: June 21/2007

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2781 Lancaster Rd.
Ottawa ON, K1B 1A7

Wash Sieve Analysis

Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	**LS 602 (ASTM C136)**
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**TP-75**		
Sampled From :	**SS-4**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**14-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**20-Jun-07**

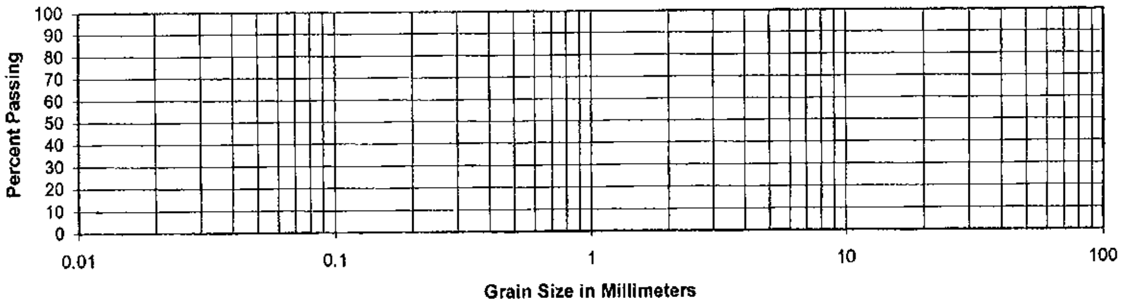
Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	**2454.1**	
Sample Weight After Sieve :		Sample Weight After Wash :	**1688.2**	**Corrected**
% Loss In Sieve :		% Passing No.200	**31.2**	**N/A**

Sieve Analysis

Sieve No.	Size of Opening Inches	Size of Opening mm	Wt. Retained grams	Cum Wt. Retained grams	% Passing	Specifications Min	Specifications Max
	3	**76.2**					
	2	**53.0**					
	1	**26.5**					
	3/4	**19.0**					
	5/8	**16.0**					
	1/2	**13.2**					
	3/8	**9.5**					
+4	**0.187**	**4.75**		**669.2**	**72.7**		
		- 4.75		**1688.1**			
8	**0.0937**	**2.36**					
16	**0.0469**	**1.18**					
30	**0.234**	**0.600**					
50	**0.0117**	**0.300**					
100	**0.0059**	**0.150**					
200	**0.0029**	**0.075**					
		Pan					

Classification of sample : % Gravel : 27.3 % Sand : 41.5 % Silt and Clay : 31.2



Remarks :

Laboratory Supervisor : Brian

Date: June 21/2007

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2781 Lancaster Rd.
Ottawa ON, K1B 1A7

Wash Sieve Analysis

Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	**LS 602 (ASTM C136)**
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**BH07-28**		
Sampled From :	**SS-3, 5' to 7'**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**15-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**20-Jun-07**

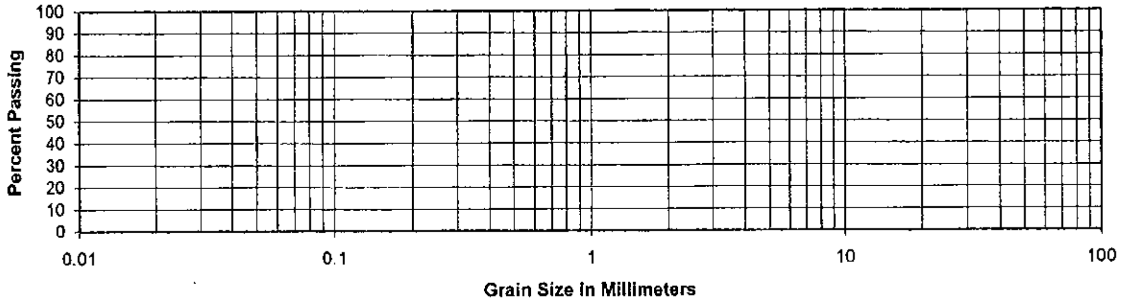
Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	325.3	
Sample Weight Aftter Sieve :		Sample Weight After Wash :	180.6	**Corrected**
% Loss In Sieve :		% Passing No.200	44.5	N/A

Sieve Analysis

Sieve No.	Size of Opening		Wt. Retained	Cum Wt. Retained	% Passing	Specifications	
	Inches	mm	grams	grams		Min	Max
	3	76.2					
	2	53.0					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2					
	3/8	9.5					
+4	0.187	4.75		1.0	99.7		
		- 4.75		180.5			
8	0.0937	2.36					
16	0.0469	1.18					
30	0.234	0.600					
50	0.0117	0.300					
100	0.0059	0.150					
200	0.0029	0.075					
		Pan					

Classification of sample :	% Gravel :	0.3	% Sand :	55.2	% Silt and Clay :	44.5



Remarks :

Laboratory Supervisor : Brian

Date: June 21/2007

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Ottawa ON, K1B 1A7

Wash Sieve Analysis

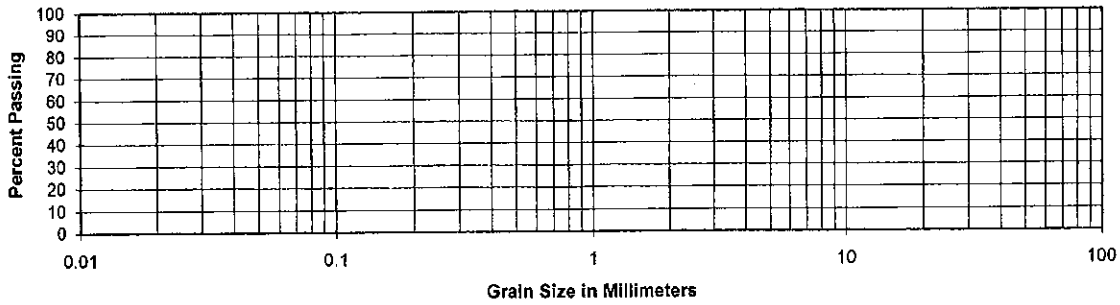
Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	**LS 602 (ASTM C136)**
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**TP-48**		
Sampled From :	**BS-3**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**14-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**21-Jun-07**

Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	254.2	
Sample Weight After Sieve :		Sample Weight After Wash :	8.3	**Corrected**
% Loss In Sieve :		% Passing No.200	96.7	N/A

Sieve Analysis

Sieve No.	Size of Opening		Wt. Retained	Cum Wt. Retained	% Passing	Specifications	
	Inches	mm	grams	grams		Min	Max
	3	76.2					
	2	53.0					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2					
	3/8	9.5					
+4	0.187	4.75		1.6	99.4		
		- 4.75		6.7			
8	0.0937	2.36					
16	0.0469	1.18					
30	0.234	0.600					
50	0.0117	0.300					
100	0.0059	0.150					
200	0.0029	0.075					
		Pan					
Classification of sample :			% Gravel : 0.6	% Sand :	2.6	% Silt and Clay :	96.7



Remarks :

Laboratory Supervisor : Brian

Date: June 22/2007

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Ottawa ON, K1B 1A7

Wash Sieve Analysis

Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	LS 602 (ASTM C136)
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**TP-48**		
Sampled From :	**BS-7**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**14-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**20-Jun-07**

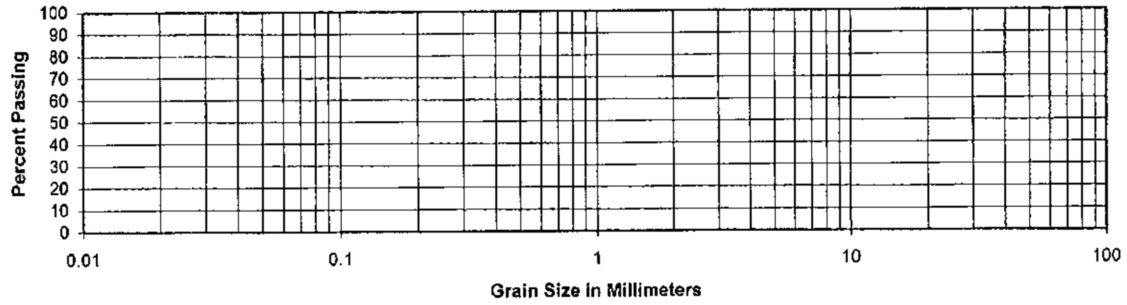
Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	385	
Sample Weight After Sieve :		Sample Weight After Wash :	232	**Corrected**
% Loss In Sieve :		% Passing No.200	39.7	N/A

Sieve Analysis

Sieve No.	Size of Opening		Wt. Retained	Cum Wt. Retained	% Passing	Specifications	
	Inches	mm	grams	grams		Min	Max
	3	76.2					
	2	53.0					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2					
	3/8	9.5					
+4	0.187	4.75		44.7	88.4		
		- 4.75		231.9			
8	0.0937	2.36					
16	0.0469	1.18					
30	0.234	0.600					
50	0.0117	0.300					
100	0.0059	0.150					
200	0.0029	0.075					
		Pan					

Classification of sample : % Gravel : 11.6 % Sand : 48.6 % Silt and Clay : 39.7



Remarks :

Laboratory Supervisor : Brian Prevost

Date: June 21/2007

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Ottawa ON, K1B 1A7

Wash Sieve Analysis

Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	**LS 602 (ASTM C136)**
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**TP-54**		
Sampled From :	**BS-3**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**14-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**21-Jun-07**

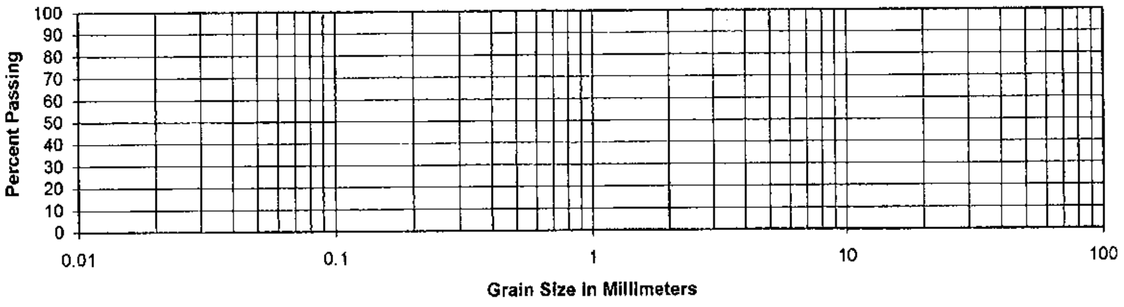
Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	**259.3**	
Sample Weight After Sieve :		Sample Weight After Wash :	**9.5**	**Corrected**
% Loss In Sieve :		% Passing No.200	**96.3**	**N/A**

Sieve Analysis

Sieve No.	Size of Opening		Wt. Retained	Cum Wt. Retained	% Passing	Specifications	
	Inches	mm	grams	grams		Min	Max
	3	76.2					
	2	53.0					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2					
	3/8	9.5					
+4	0.187	4.75		0.0	100.0		
		- 4.75					
8	0.0937	2.36					
16	0.0469	1.18					
30	0.234	0.600					
50	0.0117	0.300					
100	0.0059	0.150					
200	0.0029	0.075					
		Pan					

Classification of sample : % Gravel : 0.0 % Sand : 3.7 % Silt and Clay : 96.3



Remarks :

Laboratory Supervisor :

Date: June 22/2007

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2781 Lancaster Rd.
Ottawa ON, K1B 1A7

Wash Sieve Analysis

Client :	**Mattamy Homes**	Project No. :	**1026929**
Project :	**Mattamy Homes, Richmond**	Test Method :	**LS 602 (ASTM C136)**
Material Type :	**Soils / Aggregate:**		
Proposed Use :	**Fill / Granulars:**	Sample No. :	**N/A**
Supplier :	**N/A**		
Source :	**TP-66**		
Sampled From :	**SS-4**		
Sampled By :	**Jeff Forrester**	Date Sampled :	**14-Jun-07**
Tested By :	**Eric Naylor**	Date Tested :	**20-Jun-07**

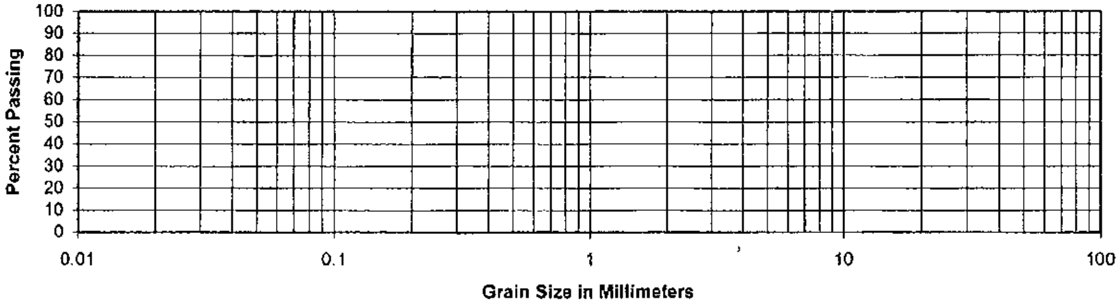
Wash Test Data

Sample Weight Before Sieve :		Sample Weight Before Wash :	**2163.8**	
Sample Weight Aftter Sieve :		Sample Weight After Wash :	**1424.5**	**Corrected**
% Loss In Sieve :		% Passing No.200	**34.2**	N/A

Sieve Analysis

Sieve No.	Size of Opening		Wt. Retained	Cum Wt. Retained	% Passing	Specifications	
	Inches	mm	grams	grams		Min	Max
	3	**76.2**					
	2	**53.0**					
	1	**26.5**					
	3/4	**19.0**					
	5/8	**16.0**					
	1/2	**13.2**					
	3/8	**9.5**					
+4	**0.187**	**4.75**		392.4	81.9		
		- 4.75		1424.3			
8	**0.0937**	**2.36**					
16	**0.0469**	**1.18**					
30	**0.234**	**0.600**					
50	**0.0117**	**0.300**					
100	**0.0059**	**0.150**					
200	**0.0029**	**0.075**					
		Pan					

Classification of sample : % Gravel : 18.1 % Sand : 47.7 % Silt and Clay : 34.2



Remarks :

Laboratory Supervisor :

Date: June 21/2007

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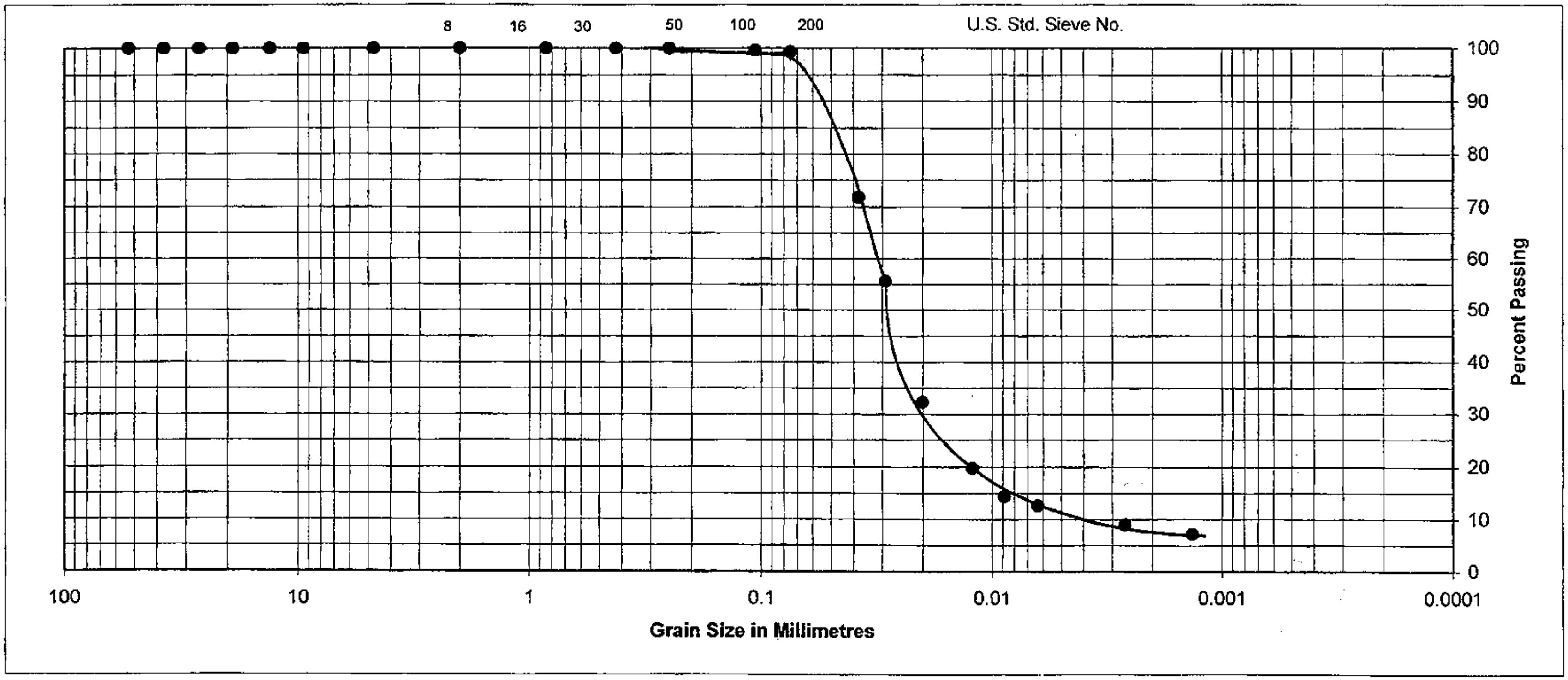
2781 Lancaster Rd.
Ottawa, Ontario K1B 1A7

Tel: 613 738-0708
Fax: 613 738-0721

Hydrometer Analysis

Client: **Mattamy Homes**
Project: **Mattamy Homes, Rickmond**
Material Type:
Source: **BH07-37, SS-3**
Sampled From: **5' to 7'**
Sampled By: **Jeff Forrester**

Project No.: **1026929**
Test Method: **LS702**
Sample No.: **N/A**
Date Sampled: **14-Jun-07**
Tested By: **Blaine Miller**
Date Tested: **21-Jun-07**



Remarks:

Reviewed By:
Date: June 22/2007

APPENDIX D

Symbols and Terms Used on the SCPTu Records
SCPTu Probe Records

Terminology Used on SCPTu Records

Key Terminology and Principles

SCPTu:

- Seismic Piezocone (SCPTu);
- A piezocone (CPTu) is an enhanced cone penetration test (CPT) probe that is able to measure porewater pressure (u);
- A seismic piezocone (SCPTu) is further enhanced to measure surface generated compression and shear waves at depth; used to define the shear wave velocity of soils.

Equipment Type and Governing Standard:

- 10 cm^2 seismic piezocone;
- 150 cm^2 friction sleeve;
- manufactured by Applied Research Associates, Inc.;
- ASTM Specification D3441.

PCPT Investigation Objectives:

- evaluate soil type and soil stratigraphy;
- estimate the relative density of granular soils and in situ undrained shear strength of cohesive soils.

Soil Behaviour Type (SBT):

- The SBT is selected based on a soil's response to cone penetration, which is different from an explicit soil type defined by specified laboratory testing procedures, but is normally what the geotechnical engineer requires for design purposes.
- The SBT can be classified on the basis of the soil friction ratio, f_s; ratio between the side shear on the friction sleeve and cone tip resistance.
- The SBT can also be classified on the basis of the normalized pore pressure, Bq; a function of the pore water response and the cone tip resistance.
- The "CPT Soil Behaviour Type Legend" used for this project is attached.

Canadian Foundation Engineering Manual (3rd Edition) Statement on the CPT

- "The most significant advantage that the electric cone penetrometers offer is their repeatability and accuracy."
- "One of the most important applications of the cone penetration test is to accurately determine the soil profile."

Key References:

T. Lunne, P.K. Robertson, and J.J.M. Powell (1997). "Cone Penetration Testing in Geotechnical Practice"; Spon Press.

P.W. Mayne (1986). "CPT indexing of in situ OCR in Clays"; Proceedings of the ASCE Specialty Conference In Situ '86: Use of In Situ Tests in Geotechnical Engineering, Blacksburg, 780-93, ASCE.

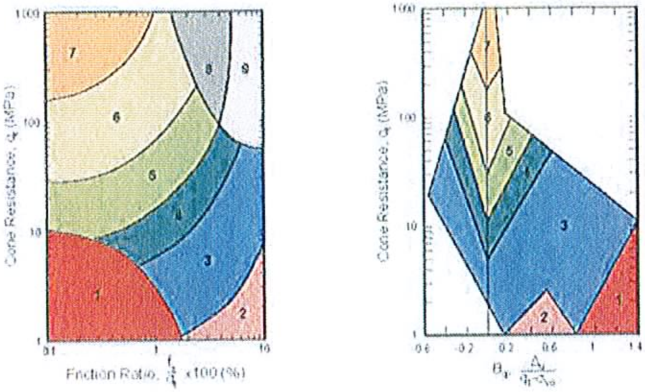
P.K. Robertson and R.G. Campanella (1988). "Guidelines for geotechnical design using CPT and CPTU"; University of British Columbia, Vancouver, Department of Civil Engineering, Soil Mechanics Series 120.

Jacques
Whitford

Terminology and Key Engineering Relationships

Parameter	Description	Symbol/Equation	Reference
Depth	Depth of the centroid of the sensor		
Elevation	Elevation of centroid of the sensor	Ground Surface – Depth	
Sleeve Stress	Sleeve Stress – interpolated to the depth of the tip	f_s	
Tip Stress, Uncorrected	Measured Tip Stress	q_c	
Tip Stress COR	Tip Stress, corrected for probe geometry	$q_t = q_c + u_2 x(1-a)$	
Ratio COR	Friction Ratio	$R_f = \frac{f_s}{q_t} x100\%$	
Pore Pressure	Measured Pore Pressure	u_2	
Soil Behaviour Type	Soil Behaviour Type	SBT	Lunne, Roberson and Powell, 1997
Overburden Stress		$\sigma_{vo} = \sum_{i=1}^{n} \gamma_i x h_i$	
Effective Overburden Stress		$\sigma'_{vo} = \sigma_{vo} - u_o$	
Normalized Tip Stress		$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$	Lunne, Robertson and Powell, 1997
Normalized Friction Ratio		$F_r = \frac{f_s}{q_t - \sigma_{vo}}$	Lunne, Robertson and Powell, 1997
Normalized Pore Pressure		$B_q = \frac{\Delta u}{q_t - \sigma_{vo}}$ $where\ \Delta u = u_2 - u_0$	Lunne, Robertson and Powell, 1997

CPT Soil Behavior Type Legend
(Robertson et al. 1990)



Zone		Soil Behavior Type
1	🟥	Sensitive, Fine Grained
2	🟥	Organic Soils-Peats
3	🟦	Clays; Clay to Silty Clay
4	🟦	Silt Mixtures; Clayey Silt to Silty Clay
5	🟩	Sand Mixtures; Silty Sand to Sandy Silt
6	🟨	Sands; Clean Sands to Silty Sands
7	🟧	Gravelly Sand to Sand
8	⬜	Very Stiff Sand to Clayey Sand*
9	⬜	Very Stiff Fine Grained*

*Overconsolidated or Cemented

Jacques Whitford

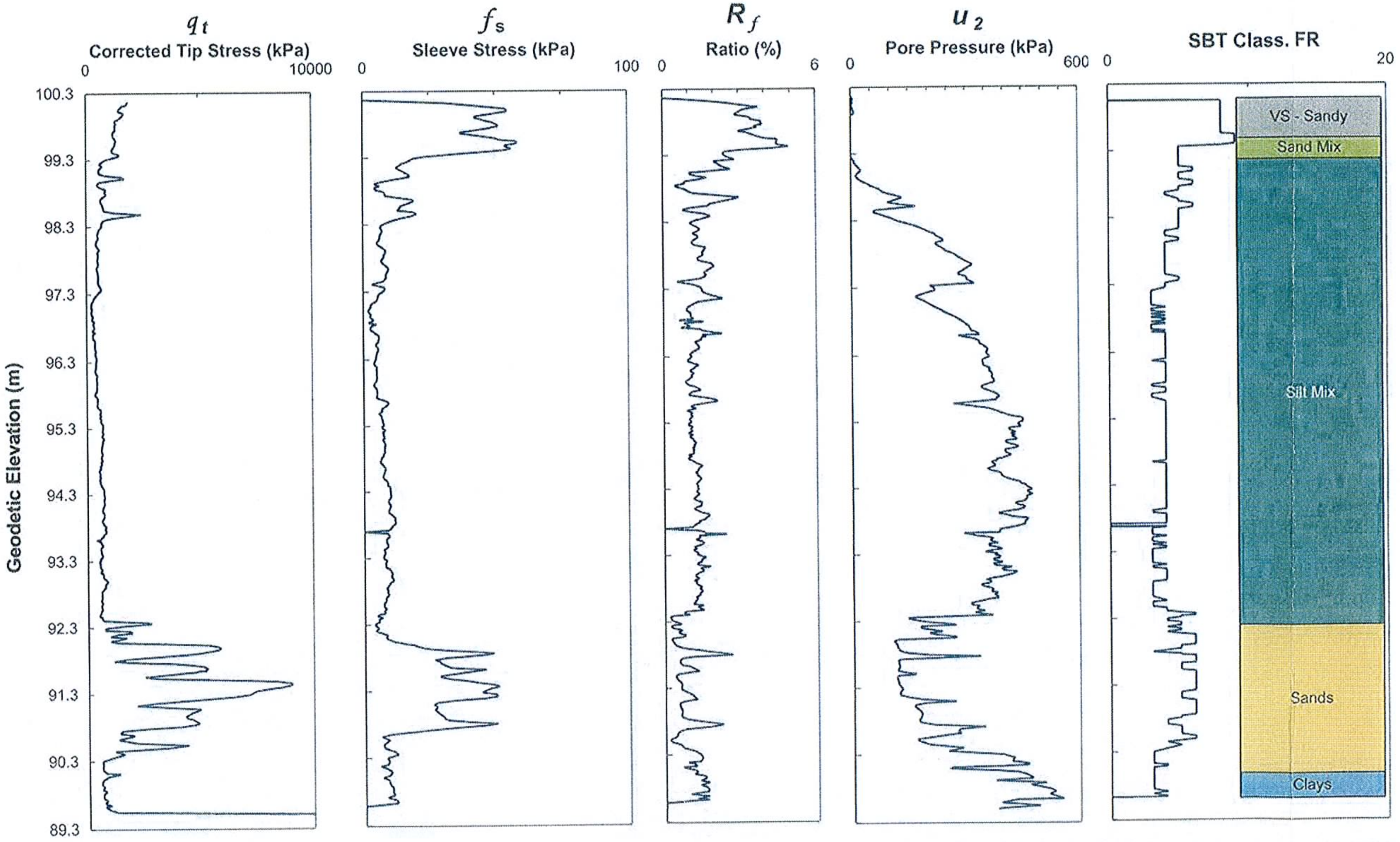
Jacques Whitford

Elevation: 100.23 m
SCPTu Start Elevation: 100.23 m
Groundwater Elevation: 98.98 m

Test Date: June 15, 2007
Project No. 1026929

CPT 07-2

Client: Mattamy Homes
Project: Proposed Residential Subdivision, Richmond, ON



Class FR: Friction Ratio Classification (Robertson, 1990)

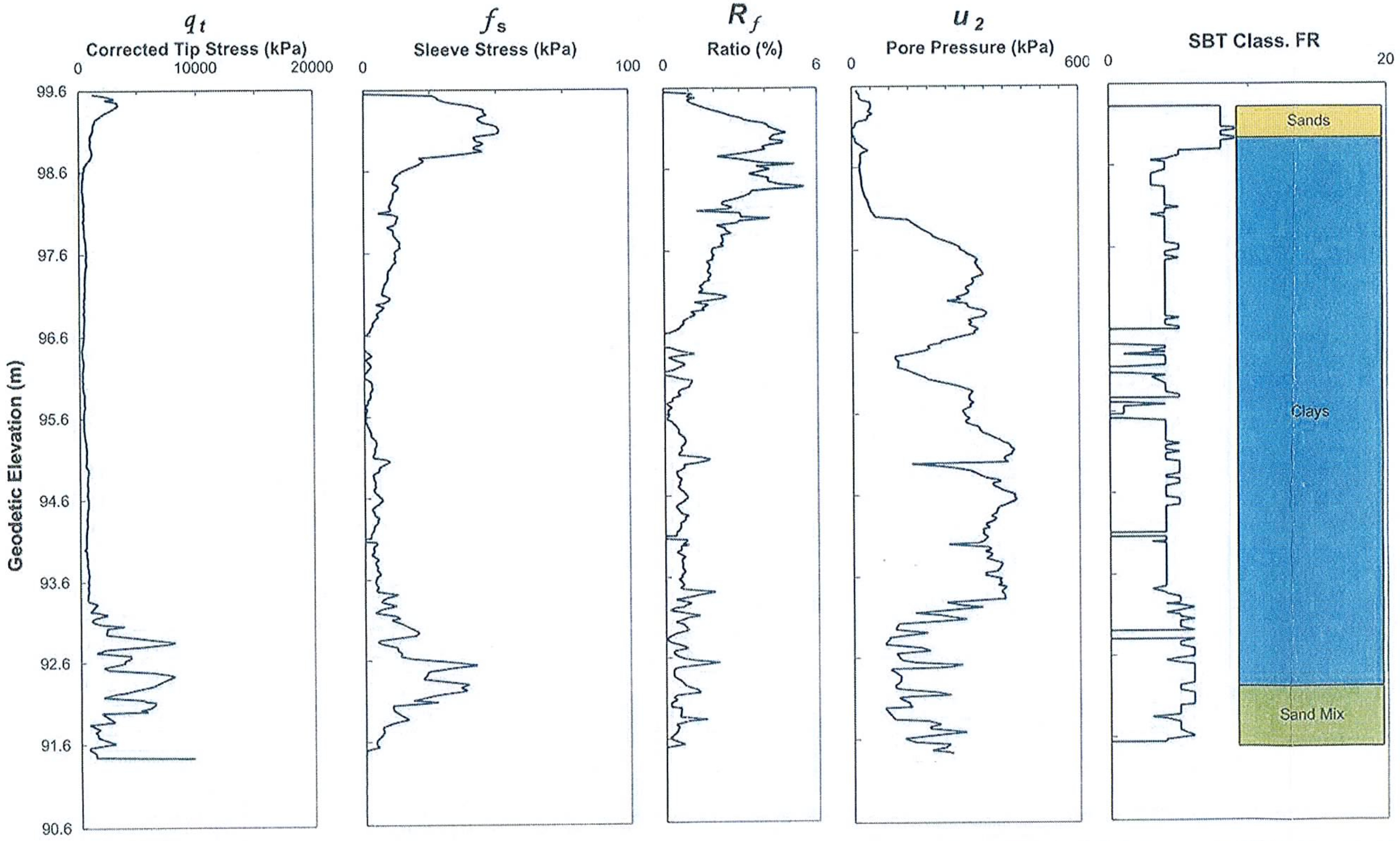
Jacques Whitford

Elevation: 99.60 m
SCPTu Start Elevation: 99.60 m
Groundwater Elevation: 98.51 m

Test Date: June 18, 2007
Project No. 1026929

CPT 07-9

Client: Mattamy Homes
Project: Proposed Residential Subdivision, Richmond, ON



Class FR: Friction Ratio Classification (Robertson, 1990)

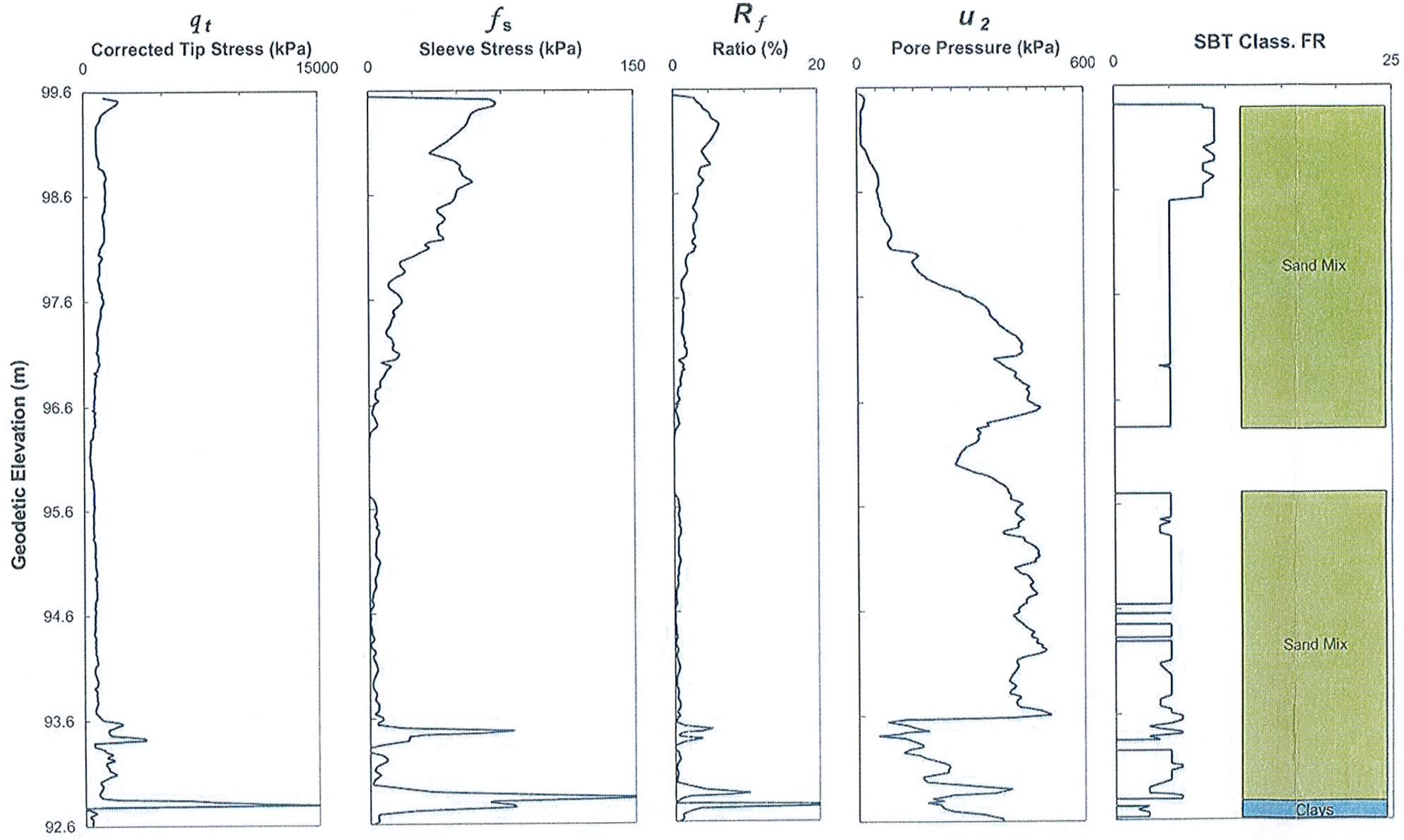
Jacques Whitford

Elevation: 99.59 m
SCPTu Start Elevation: 99.59 m
Groundwater Elevation: 98.38 m

Test Date: June 18, 2007
Project No. 1026929

CPT 07-15

Client: Mattamy Homes
Project: Proposed Residential Subdivision, Richmond, ON



Class FR: Friction Ratio Classification (Robertson, 1990)

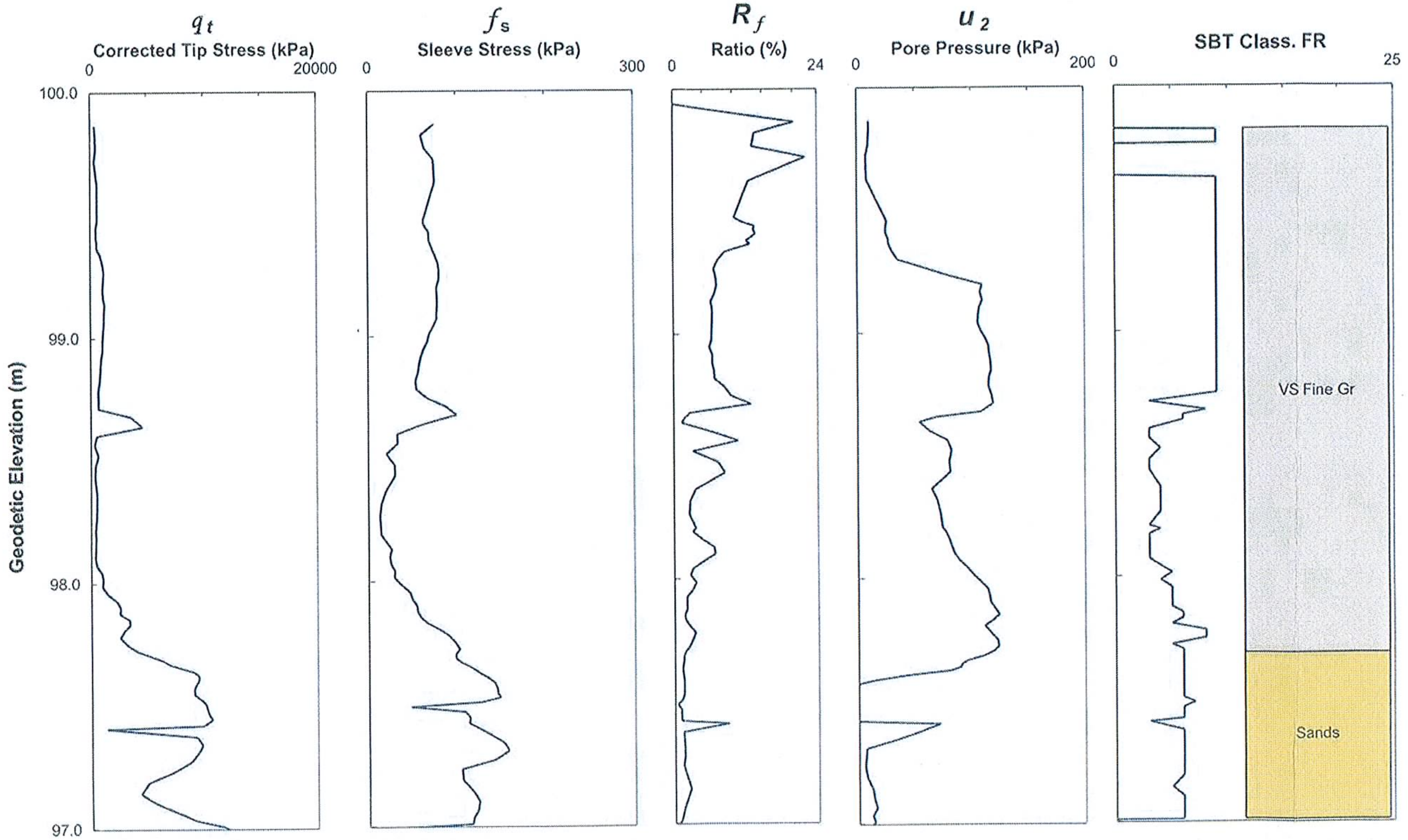
Jacques Whitford

Elevation: 99.94 m
SCPTu Start Elevation: 99.94 m
Groundwater Elevation: 98.62 m

Test Date: June 15, 2007
Project No. 1026929

CPT 07-20

Client: Mattamy Homes
Project: Proposed Residential Subdivision, Richmond, ON



Class FR: Friction Ratio Classification (Robertson, 1990)

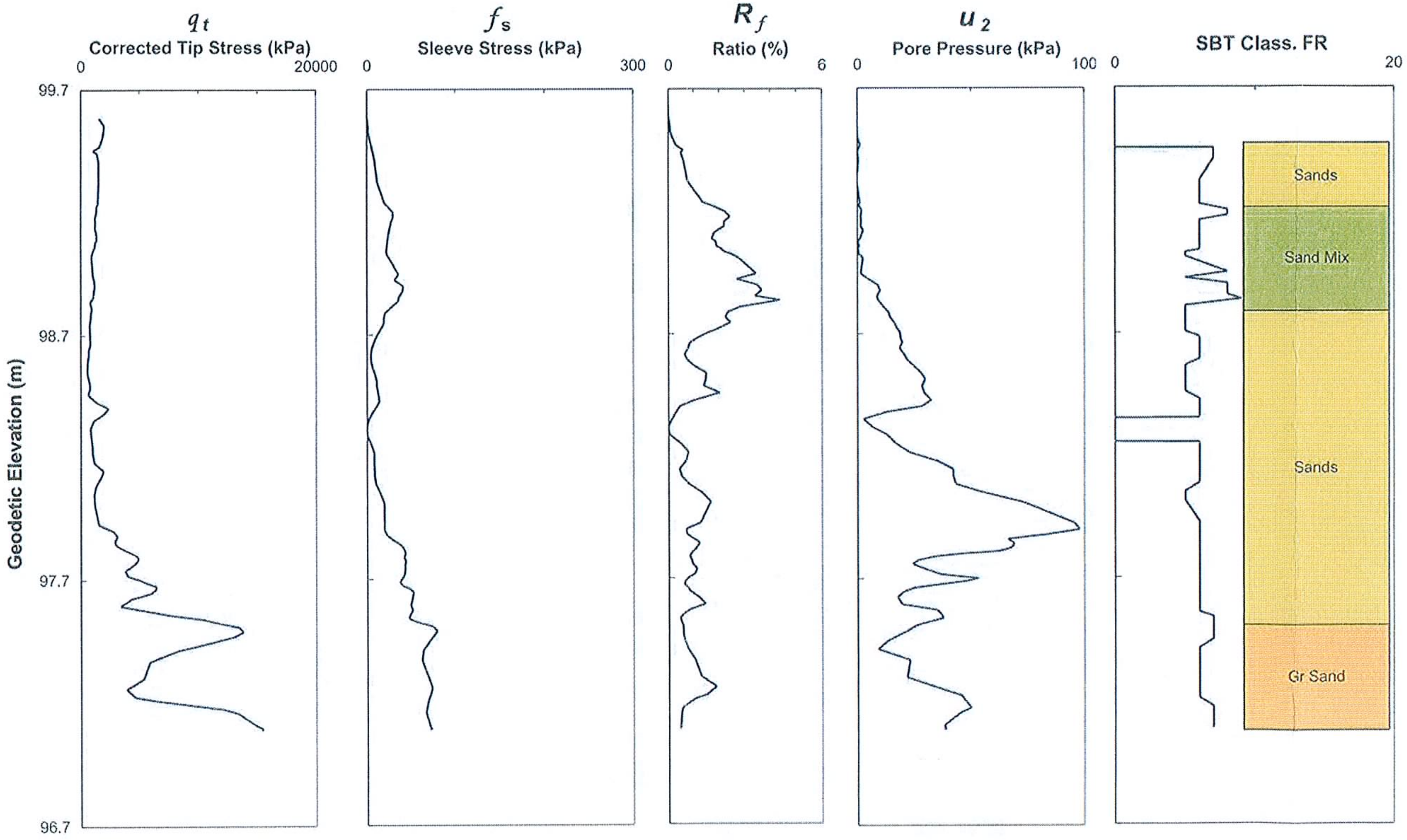
Jacques Whitford

Elevation: 99.64 m
SCPTu Start Elevation: 99.64 m
Groundwater Elevation: 98.19 m

Test Date: June 15, 2007
Project No. 1026929

CPT 07-35

Client: Mattamy Homes
Project: Proposed Residential Subdivision, Richmond, ON



Class FR: Friction Ratio Classification (Robertson, 1990)

APPENDIX C

Regulatory Floodplain Supplementary Information

RIDEAU VALLEY
CONSERVATION AUTHORITY

A member of Conservation Ontario

3889 Rideau Valley Drive, P.O. Box 599, Manotick, ON K4M 1A5
tel 613-692-3571 | 1-800-267-3504 | fax 613-692-0831 | www.rvca.ca

RVCA LETTER OF PERMISSION – ONT. REG. 174/06, S. 28 CONSERVATION AUTHORITIES ACT 1990, AS AMENDED.

Revised Date: March 3, 2009
Orig Date: February 17, 2009
File: RV5 0309

Mattamy Ltd. (Jock River)
123 Huntmar Drive
Ottawa, ON
K2S 1B9

Attention: Sean MacFarlane,

Subject: **Application for Development being the establishment of an earth berm on a property at part of Unit 19, Index Plan 4D-23 and part of Lot 2, Concession 2, former Township of Goulbourn, now in the City of Ottawa, bounded by Ottawa Street to the North and the Jock River to the south within the Village of Richmond.**

Dear Mr. MacFarlane,

The Rideau Valley Conservation Authority has reviewed the application submitted on behalf of Mattamy Ltd. and understands the proposal to be for:

1. The removal of fill associated with construction of a berm already on the property.
2. The establishment of a new berm closer to the municipal road and farther from the river.
3. The reinstatement of the land area at the existing/"*original*" berm site.
4. The placement of fill between the new berm and Ottawa Street to match existing surrounding grades.

This proposal was reviewed under Ontario Regulation 174/06, the "*Development, Interference with Wetlands, and Alteration to Watercourse and Shorelines*" regulation, which the Conservation Authority administers as well as the approved "Policies Regarding the Construction of Buildings and Structures, Placing of Fill and Alterations to Waterways" (Adopted by the Executive Committee, October 21, 1993 and last revised and approved by the Board of Directors, April 2006).

History

This proposal relates to two previous regulation application filings on the property, RV5-16/05 submitted and approved by the RVCA on December 9, 2005 under Ont. Regulation 166/90 and RV5-01/08 submitted by Mattamy relating to work undertaken in December 2007.

In 2005, a letter of permission was issued by the RVCA for the construction of a berm "to maintain flood risk mapping land levels as per (the 1980 Acres Floodplain) Mapping Study (96.0 metres)". The application indicated that the berm would be 15 cm to 30 cm in height and on site material would be used to construct the berm at a height of land determined in the field as the natural or high point of land, approximately the 96.0 metre contour. The berm as it exists today is 1-2 metres in height, 10-20 metres in width at the base, 2-100 metres from the Jock River and established more or less at the 95.5 metres contour.

On May 6, 2006, Ontario Regulation 166/90 (Fill, Construction and Alteration to Waterways Regulation) was rescinded and Ontario Regulation 174/06 was enacted. Both regulations were made pursuant to the provisions of Section 28 of the Conservation Authorities Act, and both gave responsibility for administration to the Rideau Valley Conservation Authority. The change in the regulatory legislation in May 2006 resulted in, amongst other things, incorporation of revised and extended hazard delineation along the Jock River (Jock River Floodplain Mapping Study completed in July 2005 by PSR). The new Mapping Study extended the hazard mapping upstream of the previous limit of study and provided for additional lands being subject to RVCA's regulatory jurisdiction; the 1:100 year flood level was considered to be 96.35 metres. A provincially mandated public process was followed in order for this new mapping to take effect.

The purpose of the 2005 application (previous property owner) was to establish a berm at the flood level, (96.0 metres – defined by the pre 2006 Flood Plain Mapping Study) thereby, preventing the remainder of the property from being subject to the floodplain regulations after incorporation of the New Mapping Study into the regulation in May 2006.

Project Scope

This application proposes removing the existing berm [established in Winter/Spring 2006, with further filling and reinforcement in December 2007] and locating it at the 2005 approved location. It is also proposed to remove the flapgate and culvert from the drainage easement until the issues relating to drainage for the property and greater surrounding area can be addressed. The berm will also be extended parallel along both sides of the drainage easement north up to Ottawa Street. The overall function of the drainage ditch/easement will be reviewed as part of a Storm Water Management and Drainage Plan to be completed by Mattamy respecting any future development applications.

The placing of fill for the berm and the associated filling behind the berm up to Ottawa Street is not expected to have a negative impact on the control of flooding. The filling operation and location of the "berm" should not impact the roadside conveyance of water and should not impact the conveyance of water through the Drainage Easement running through the centre of the subject property providing the work in undertaken as proposed.

This permission results in the re-location of the 2005 berm in the originally approved location. The filling between the relocated berm (maximum height 96.4 metres geodetic) and Ottawa Street will be to a maximum level of 96.5 and only to a level to taper to match existing grades.

Permission and Conditions

By this letter the Rideau Valley Conservation Authority hereby grants you permission to undertake the works on this property as outlined in the application, covering letter dated January 23, 2009 from David Schaeffer Engineering Ltd. and Drawings completed by same, Project 303, Figures 1-4, dated January 21, 2009 prepared for Mattamy Ltd. and all subject to the following conditions:

1. The existing berm will be removed from its current site. The material may be used for the construction of the new berm **if** deemed appropriate by a qualified Professional Engineer. Any excess material not required for the construction of the new berm may be used as fill in areas to the north of the relocated berm to a maximum elevation of 96.5 metres, noting the maximum berm height will be 96.4 metres. Any excess material will be disposed of in a suitable location outside all 1:100 year floodplains and regulated areas and not within a wetland.

2. The proposed berm shall be established as noted on the attached plan, (DSEL, Project 303, Figure 1-4 dated January 21, 2009) with the southern most boundary of the berm located no further south than the line denoted on the plan and established at an original grade no lower than 96. 1 metres

as depicted on the plan with the top of the berm at a maximum elevation of 96.4 metres or not greater than 0.3 metres in height. Grades along Ottawa Street will also be no higher than 96.4 metres.

3. The berm is to be located no closer than 3 metres to the drainage watercourse on both sides and/or **not** within the City of Ottawa's Drainage Easement, whichever is greater.

4. The culvert located in the drainage easement will be removed as the earthworks are completed and no fill or other obstruction will be placed in the drainage easement without approval from the City of Ottawa. The flap gate will be removed immediately from the culvert to ensure free flow of water.

5. The area of the original berm will be rehabilitated to original grade with plant material re-established as discussed in DSEL letter dated January 23, 2009.

6. A finished grading plan will be submitted as soon as the work is complete to confirm the height and location of the new berm and the grade height and location of the existing berm reinstated to original grade. A refundable deposit of $3000.00 is required to be submitted prior to commencement of the work.

7. No earthworks shall commence before May 1, 2009 and only if flows in the Jock River are considered at or lower than summer levels. Sediment control will be established as defined on Figure 3 prepared by David Schaeffer Engineering Ltd. (DSEL), dated January 21, 2009 and additional measures will be installed as deemed necessary.

8. Sediment and Erosion control Inspections will take place daily by DSEL or other appropriate and designated engineering staff while work is ongoing, and thereafter prior to and immediately following any forecasted rainfall event until such time as the site is revegetated, to ensure sediment control is operational and functional. This is ultimately the responsibility of the property owner and all persons associated with the construction/earthworks/grading on the property. Failure to comply may result in this permission being revoked and charges under other applicable law.

9. Any material excavated from the property and not required must be removed immediately and disposed of at a suitable disposal site outside the 100 year floodplain and regulated area.

10. The RVCA is to receive 48 hours notice of the proposed commencement of the proposed works to ensure compliance with all conditions.

11. There is to be **no** work at the bank or in the Jock River as part of this approval. The contractor will take all necessary measures to ensure no sediment migration into the river occurs and that no machinery, no grubbing and no further vegetation clearing takes place on the property as part of the proposed rehabilitation work. Failure to comply may result in this permission being revoked and charges under other applicable law.

12. It is acknowledged by all parties that the area of the fill operation between the new berm and Ottawa Street, when completed and confirmed, will be considered outside the 1:100 year flood plain of the Jock River. The Regulated Area of Ontario Regulation 174/06 nevertheless extends 15 metres from the 1:100 flood elevation which will be confirmed as the outer limit of the relocated berm. The portion of the property within the Regulated Area of Ontario Regulation 174/06 remains subject to the RVCA's regulatory jurisdiction regarding the *Development, Interference with Wetlands and Alteration to Watercourses"* regulation and associated implementation policies, as amended from time to time by the RVCA Board of Directors.

The regulatory mapping for Ontario Regulation 174/06 is reviewed once a year and amendments introduced as required. Once the works are completed, the Rideau Valley Conservation Authority agrees to pursue an amendment to the regulatory mapping to reflect the changes to the 1:100 floodplain contemplated in this approval.

Failure to comply with any or all conditions will result in the letter of permission being revoked with associated legal action to remedy the situation to the Conservation Authority's satisfaction.

By this letter, the Rideau Valley Conservation Authority assumes no responsibility or liability for any flood or erosion damage which may occur either to your property or the structures on it, or if any activity undertaken by you adversely affects the property or interests of adjacent landowners. This letter does not relieve you of the necessity or responsibility for obtaining any other federal or provincial permits including, but not limited to, approvals under the Public Lands Act, Ontario Water Resources Act, Building Code Act, Planning Act, Endangered Species Act etc.

Nothing in this letter of permission is intended to imply or confer any right of occupation or use of public land. This permission may not be transferred to any other party.

Should you have any questions, please contact Shelley Macpherson at our office.

Yours truly,

Donald A. Maciver, MCIP, RPP
Director of Planning

SM/DAM/sm

cc: Susan Murphy, Mattamy Ltd.
City of Ottawa, Guy Bourgon
City of Ottawa, Mike Wildman
City of Ottawa, Don Morse MCIP RPP
Helmut Brodmann LL.B. Bell, Baker
DSEL

NOTE: This letter of permission does not come into full force and effect until the attached copy of this letter is signed by the responsible corporate official, dated, and returned to the Authority offices in Manotick.

This letter's **return shall be taken as an acknowledgment and an acceptance of the conditions contained herein, as per the Authority's approval based on the application submitted.**

Pursuant to the provisions of S. 28 (12) of the Conservation Authorities Act (R.S.O. 1990, as amended) any or all of the conditions set out above may be appealed to the Executive Committee of the Conservation Authority in the event that they may be unsatisfactory or can not be met.

Name (printed) ______________________ **Date:** ______________

Corporate position / title: ______________________________

Signed: ______________________________

Supplementary STAFF REPORT

To: **RVCA Board of Directors**

Prepared by: **Bruce A. Reid, P.Eng., Director,**
Watershed Science and Engineering Services

Subject: **Van Gaal Drain Flood Plain Mapping – Final Report**

Date: **January 28, 2010**

At it's December 18, 2009 meeting, the Board postponed its decision with respect to the acceptance of the JF Sabourin and Associates Final Report on the Van Gaal Drain Flood Plain Mapping (item 6 on the December 18th agenda), in order to give "staff and proponent engineers a further chance to review and come to agreement on the methodology".

Staff met with representatives the City of Ottawa, and Mattamy Homes and their engineering consultants on January 14th. Minutes of that meeting are attached hereto.

There continues to be difference of opinion between Mattamy's consultants and RVCA's consultant with respect to the selection of the 1:100 year design flow (summer event) on the Van Gaal Drain. RVCA staff continue to be confident that using the JFSA design flows will be defensible before appeal tribunals (Mining and Lands Commissioner, OMB or the Courts) if and when necessary.

At the January 14th meeting we did achieve consensus on a step-by-step process to bring the matter to a successful conclusion, summarized as follows

1. RVCA will accept the JFSA flood plain mapping for original conditions of the land, and begin administering and enforcing O.Reg. 174/06 on the flood hazard areas defined in it.

2. Additional channel modifications will be done to increase the channel's conveyance capacity, compensating for the loss of conveyance capacity in overbank areas due to recent berm construction and returning the 1:100 year water surface profile to its pre-berm position. The work will be done by Mattamy, and will be subject to formal approval by the City's Drainage Superintendent as well as RVCA (under O.Reg. 174/06).

3. Grades will be raised in the areas behind (upland) of the existing berms, such that the finished grades will no longer be lower than the water surface profile

DSEL
david schaeffer engineering ltd
SMART SUBDIVISIONS™

120 Iber Road, Unit 203
Stittsville, Ontario, K2S 1E9
Tel. (613) 836-0856
Fax (613) 836-7183
www.DSEL.ca

September 14, 2012

Jocelyn Chandler M.Pl. MCIP, RPP
Rideau Valley Conservation Authority
3889 Rideau Valley Drive
Ottawa, ON K2C 3H1

Re: RVCA File No. 11-GOU-SUB
Richmond Village (South) Ltd.
Proposed Van Gaal Channel Re-Alignment

DSEL in collaboration with JFSA, nak design strategies, JTBES, and Kilgour and Associates have prepared a preliminary design in support of a Richmond Village (South) Ltd., proposal to realign and widen the Van Gaal Drain north of Perth Street in order to redefine the floodplain North of Perth Street. The following and attached supporting information was prepared to provide the Rideau Valley Conservation an opportunity provide preliminary feedback on the proposed re-alignment.

The supporting information is presented as a first step toward addressing the existing floodplain north of Perth Street. The following meetings and reports concerning the Van Gaal channel and associated floodplain north of Perth Street have taken place to date:

- January 28, 2010 – Supplemental Staff Report to the RVCA Board of directors, supporting a 2008 proposal to raise the grade of the subject property behind the existing constructed berms. The proposal to fill the area behind the berms was relying on fill generated from the development itself. To import fill specifically to raise the grade would have negative environmental impact (ie numerous trucks to import fill).
- May 19, 2011 – Representatives from Richmond Village (South) Ltd., met with RVCA staff to discuss a proposal to widen and realign and widen the Van Gaal drain north of Perth Street as a vehicle to remove the interim floodplain north of Perth Street.
- July 7, 2011 – DSEL submitted a preliminary design and supporting hydraulic analysis to realign and remove the interim floodplain north of Perth Street for RVCA review and input.
- August 5, 2011 – RVCA provided review comments by e-mail to be addressed and considered in the subsequent submission.

A new channel alignment has been contemplated to address the interim floodplain North of Perth Street and also addresses a number of additional issues. The current proposal is a deviation from the previously submitted channel re-alignment and as such, the consulting team felt it prudent to involve the RVCA prior to completing detailed design.

The following supporting information is attached:

- Site photographs of the existing Van Gaal Drain taken 2012-08-30.
    - At the time of the site visit, the existing drain was dry. Note that a total of 81.9mm of rain fell in the previous 30days leading to August 30, with 8.5mm of rain on August 25 (source: The Weather Network Online). The site photographs show that the existing channel has very few trees providing shade.
- Site photographs of the existing Todd Pond outlet taken 2012-08-30.
    - Nak design strategies was part of the team who created the Todd Pond outlet. This channel is fed by an upstream pond, and therefore has a consistent base flow. These photographs illustrate the desired end product for the proposed Van Gaal Drain outlet.
- nak design strategies Van Gaal Drain Re-Alignment and Channel Enhancement Plans
    - The attached plans illustrate a conceptual planting and meandering low flow channel. These plans were developed in coordination with JFSA, JTBES, and Kilgour and Assoc.
- J.F. Sabourin and Associates Inc – Technical Memo
    - Hydraulic Analysis of realigned drain. The channel hydrology considers the type and placement of the proposed landscaping treatment in predicting water levels along the realigned drain. The analysis shows that the 100-year event is contained within the channel. Some modest modifications to the inlet of the Perth Street culvert are required.
- JTBES – Technical memo
    - JTBES provided advice on the proposed re-aligned channel. Their office prepared additional investigations of the downstream water course and have noted areas of concern. The proposed re-aligned channel will reduce upstream velocities and will benefit the downstream system.
- Kilgour and Associates - Technical Memo
    - Kilgour and Associates were involved in discussions pertaining to the realigned drain and were asked for their feedback and input. Their technical memorandum comments on the proposed design and gives design advise.

The following summarizes additional benefits the re-aligned channel will have on the site as a whole.

1. Addresses and contains the interim floodplain north of Perth Street.

    a. Proposed channel realignment and widening meets the goals set forth to contain the 100-year flood elevations without increasing water levels.
    b. Eliminates the need to import soils to redefine floodplain per previous agreement. Importing additional soils would prevent continued agricultural practices from taking place. Would unnecessarily add numerous vehicles required to import soils, which would have a negative impact on the existing community. IE noise / traffic.
2. Fish Habitat
    a. Channel definition is greater in length and provides opportunity for natural channel design thus improving and adding to the total available habitat.
3. Potential for preservation of woodlot.
    a. Channel definition would potentially minimize the impact on the existing woodlot.
4. No road crossing
    a. Re-aligning channel to the eastern property line eliminates need for the proposed road crossing.
5. Potential for benefit to mitigate existing downstream erosion.
    a. Low flow channel definition may provide opportunities to reduce in stream velocities providing a benefit downstream (JTBES to confirm requirements, JFSA to review impact of greater friction in channel on predicted water levels.)
6. Removes Isolated Stormwater Management Facility
    a. The previous alignment required the introduction of a small stormwater management facility to service a 3.4ha area. The previously conceived facility was not optimal in that it serviced an area less than MOE recommendations (greater than 5ha). Eliminating this facility reduces infrastructure requirements for development. In turn potential reduction in capital expenditures as well as operation and maintenance.

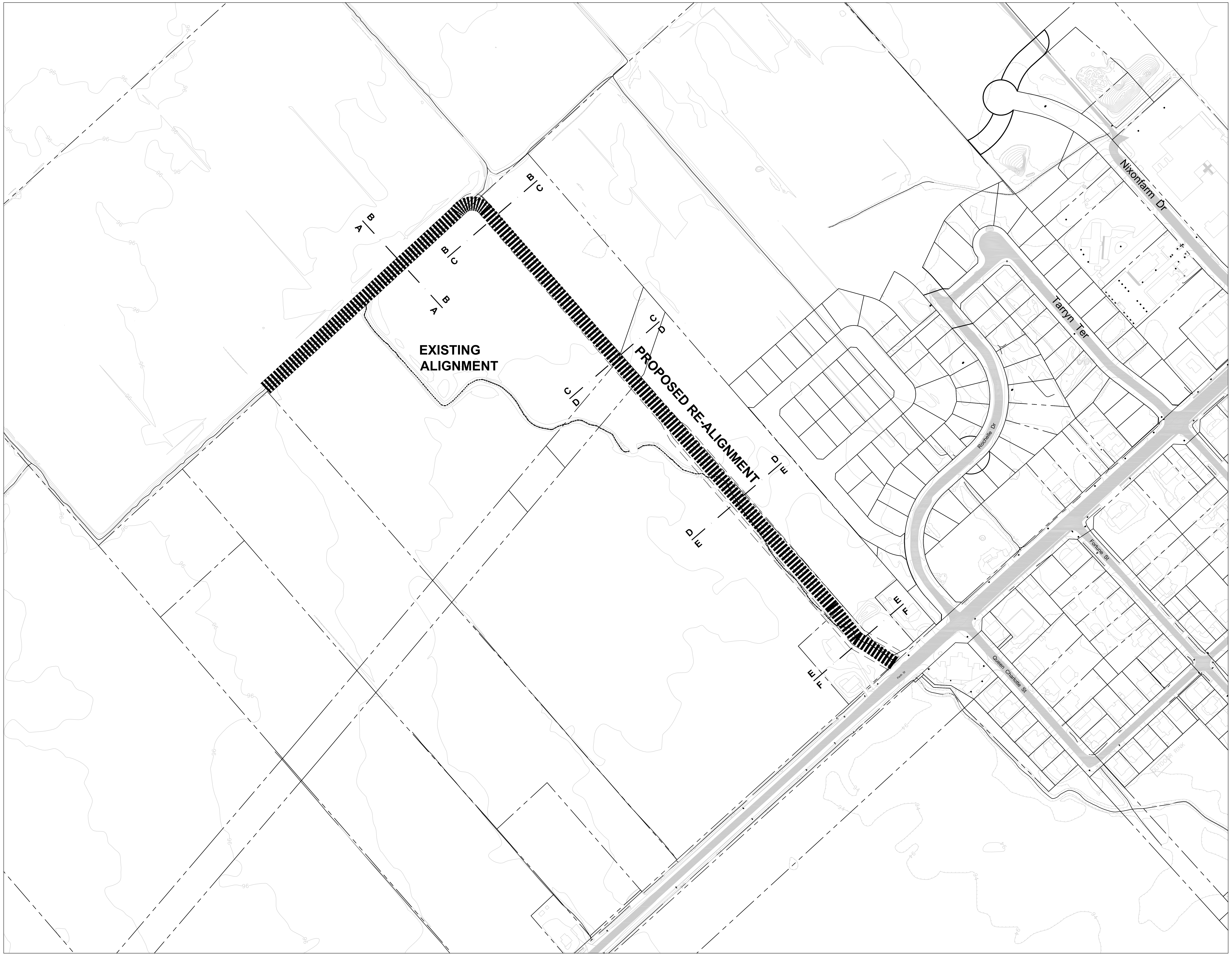
We look forward to discussing the attached proposal and moving forward with detailed engineering plans.

Yours truly,
David Schaeffer Engineering Ltd.

Per: Adam D. Fobert, P.Eng.

Yours truly,
David Schaeffer Engineering Ltd.

Per: Stephen J. Pichette, P.Eng.

VAN GAAL DRAIN RE-ALIGNMENT & CHANNEL ENHANCEMENTS

Richmond Village Development Corporation

CONTEXT

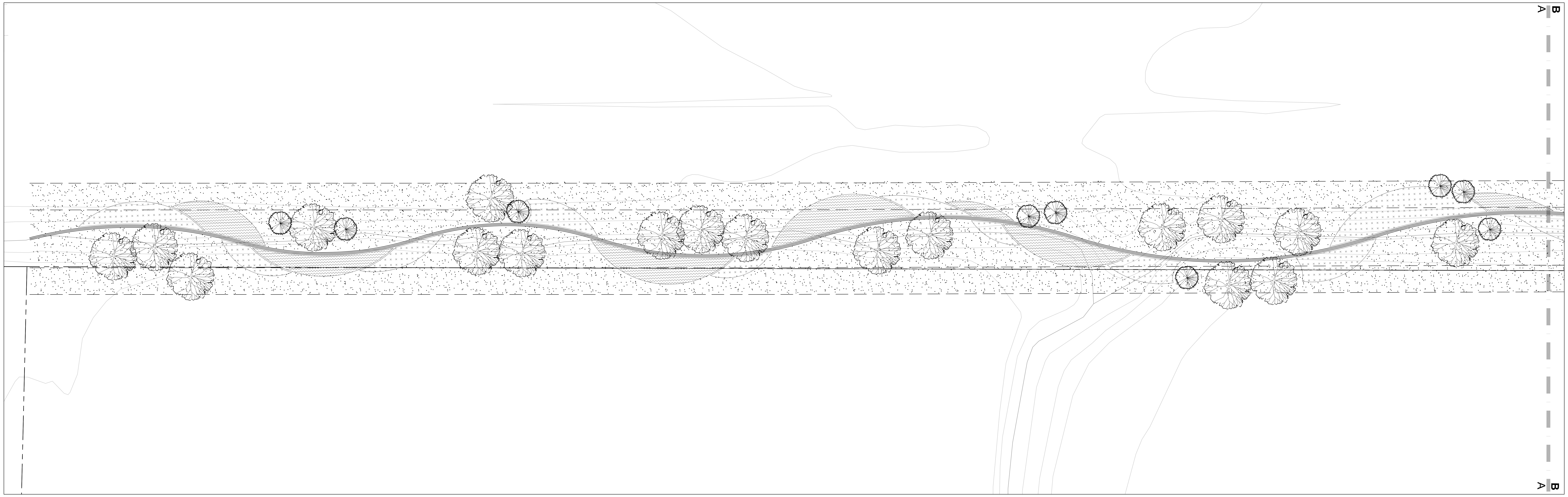
N

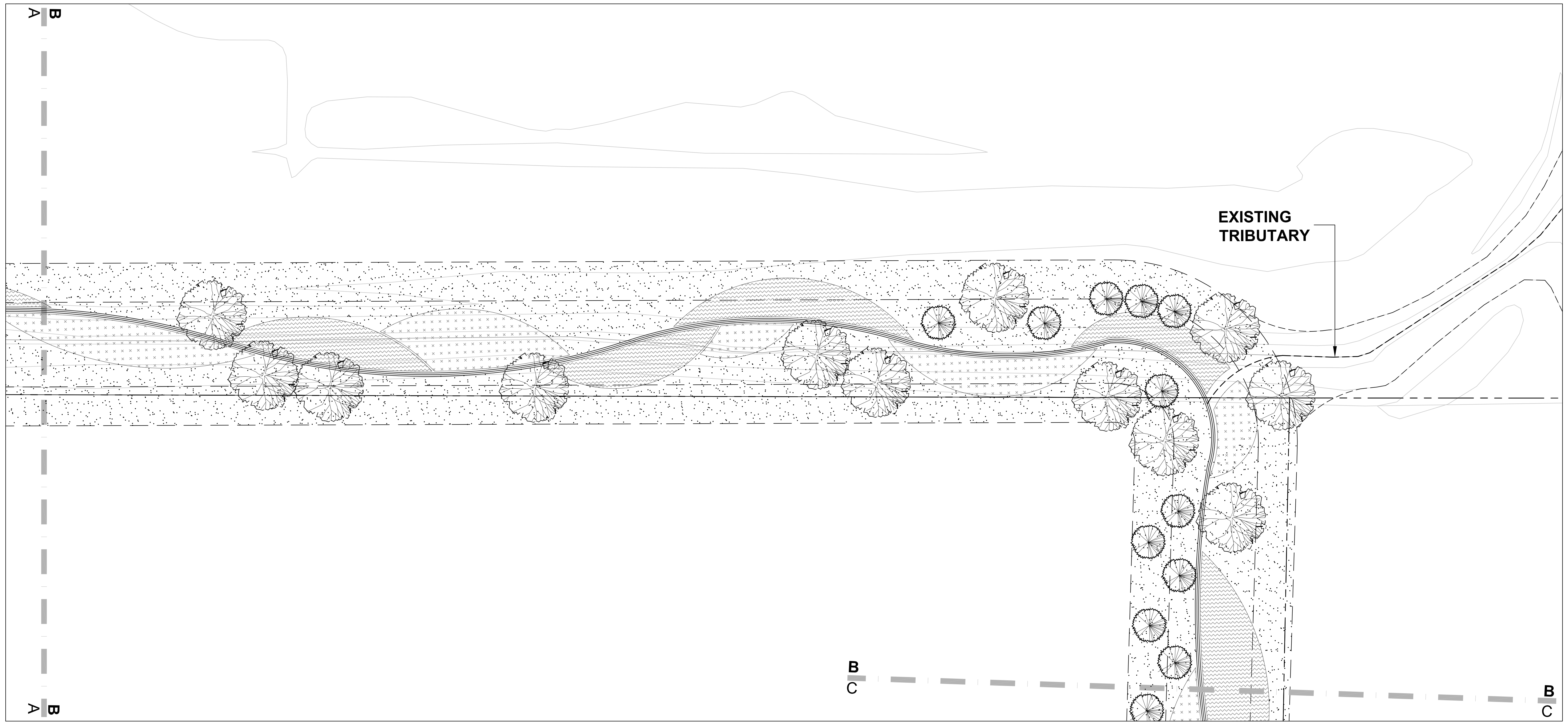
12.08.30

1:2000

Job No. 1-12___

nak design strategies
250 Besserer Street | Studio 100 | Ottawa, Ontario | K1N 6B3
tel: 613.237.2345 | fax: 613.237.6423 | nak@nak-design.com





PROPOSED PLANT LIST

DECIDUOUS TREES
- SUGAR MAPLE
- RED MAPLE
- SILVER MAPLE
- BUR OAK
- RED OAK
- TREMBLING ASPEN
- BLACK WILLOW

CONIFEROUS TREES
- TAMARACK
- WHITE SPRUCE
- WHITE PINE
- WHITE CEDAR

SHRUBS
- PEACH-LEAVED WILLOW
- BEBB'S WILLOW
- PUSSY WILLOW
- STAGHORN SUMAC
- COMMON ELDERBERRY
- SERVICEBERRY
- NANNYBERRY
- HIGHBUSH CRANBERRY
- RED OSIER DOGWOOD
- GRAY DOGWOOD
- CHOKECHERRY

WET MEADOW SEED MIX
- 25% CANADA BLUE JOINT GRASS
- 25% ROUGH-STALKED MEADOW GRASS
- 20% HIGHLAND COLONIAL BENTGRASS
- 15% CREEPING RED FESCUE
- 5% TALL WHITE ASTER
- 10% NEW ENGLAND ASTER

DRY-MESIC MEADOW SEED MIX
- 25% CANADA BLUE GRASS
- 25% CREEPING RED FESCUE
- 25% PERENNIAL RYEGRASS
- 10% RED CLOVER
- 10% BLACK-EYED SUSAN
- 5% NEW ENGLAND ASTER

LEGEND

✿	DECIDUOUS TREE
✺	CONIFEROUS TREE
— —	TOP / BOTTOM OF SLOPE
- - - -	EXISTING TRIBUTARY
▒	SEEDING
░	SHRUB MIX 'A'
▨	SHRUB MIX 'B'
▧	STABILIZATION PLANTING

PLANTING KEY

⬡ XX / #	TREE SPECIES QUANTITY
○ x / #	SHRUB SPECIES QUANTITY

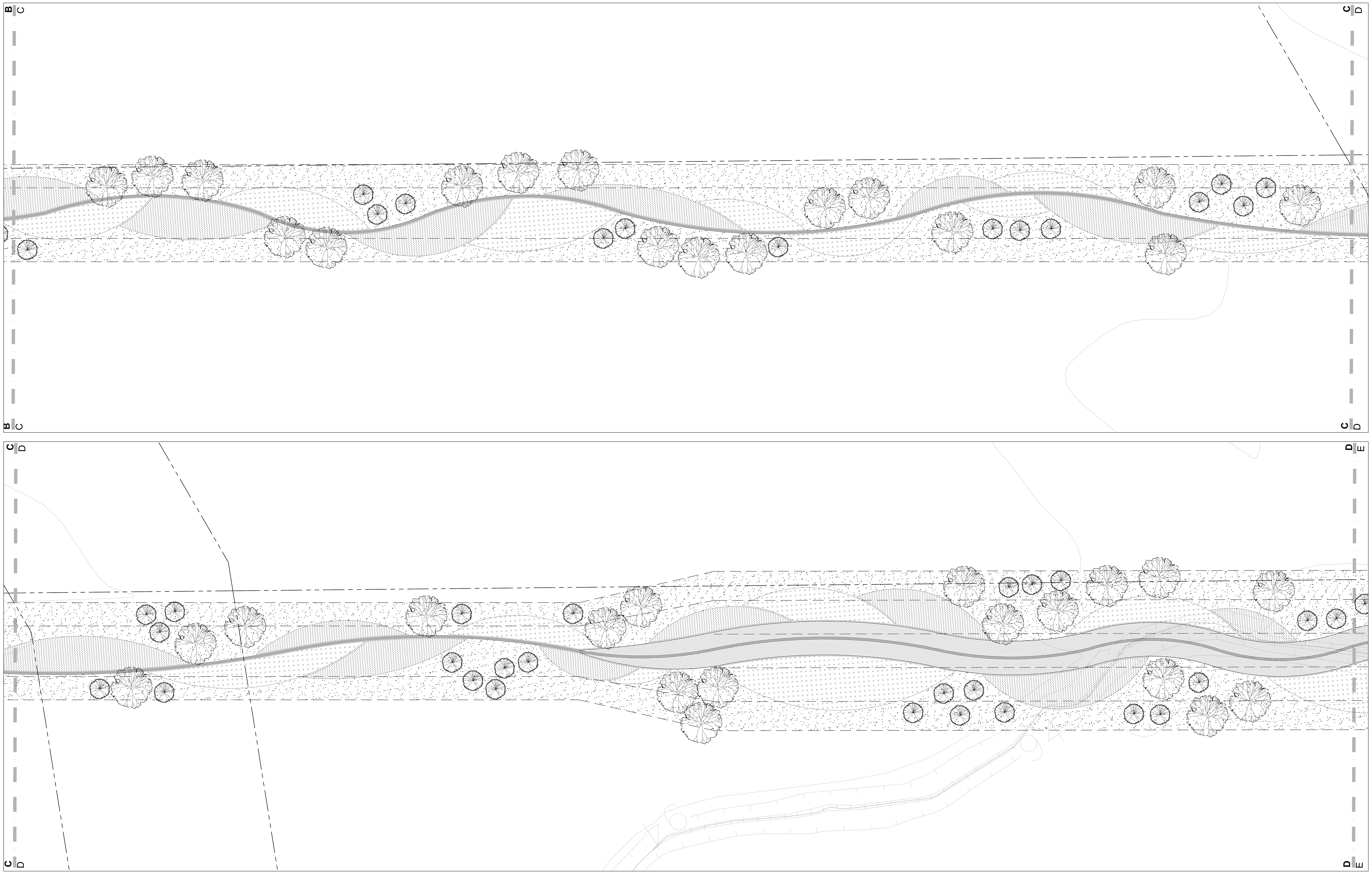
VAN GAAL DRAIN RE-ALIGNMENT & CHANNEL ENHANCEMENTS

Richmond Village Development Corporation

LAYOUT PLANS

12.09.31
1:200
Job No. 1-12___

nak design strategies
250 Besserer Street | Studio 100 | Ottawa, Ontario | K1N 6B3
tel: 613.237.2345 | fax: 613.237.6423 | nak@nak-design.com



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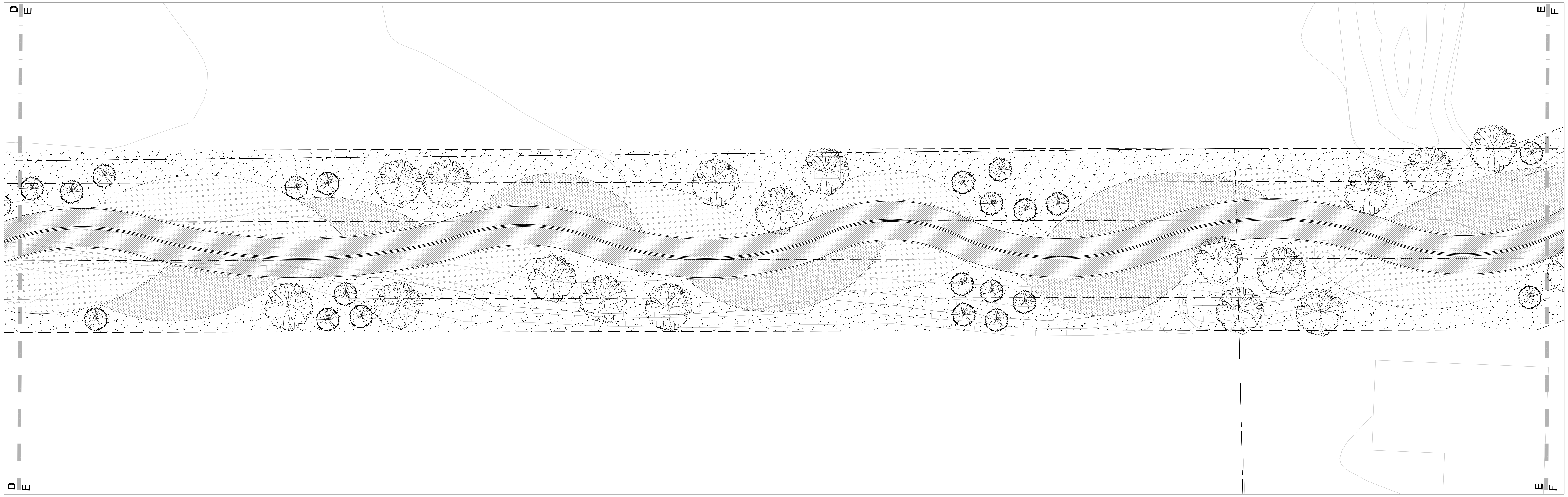

PROPOSED PLANT LIST

DECIDUOUS TREES
- SUGAR MAPLE
- RED MAPLE
- SILVER MAPLE
- BUR OAK
- RED OAK
- TREMBLING ASPEN
- BLACK WILLOW

CONIFEROUS TREES
- TAMARACK
- WHITE SPRUCE
- WHITE PINE
- WHITE CEDAR

SHRUBS
- PEACH-LEAVED WILLOW
- BEBB'S WILLOW
- PUSSY WILLOW
- STAGHORN SUMAC
- COMMON ELDERBERRY
- SERVICEBERRY
- NANNYBERRY
- HIGHBUSH CRANBERRY
- RED OSIER DOGWOOD
- GRAY DOGWOOD
- CHOKECHERRY

WET MEADOW SEED MIX
- 25% CANADA BLUE JOINT GRASS
- 25% ROUGH-STALKED MEADOW GRASS
- 20% HIGHLAND COLONIAL BENTGRASS
- 15% CREEPING RED FESCUE
- 5% TALL WHITE ASTER
- 10% NEW ENGLAND ASTER

DRY-MESIC MEADOW SEED MIX
- 25% CANADA BLUE GRASS
- 25% CREEPING RED FESCUE
- 25% PERENNIAL RYEGRASS
- 10% RED CLOVER
- 10% BLACK-EYED SUSAN
- 5% NEW ENGLAND ASTER

LEGEND

✺	DECIDUOUS TREE
⊛	CONIFEROUS TREE
- - -	TOP / BOTTOM OF SLOPE
▒	SEEDING
░	SHRUB MIX 'A'
▦	SHRUB MIX 'B'
▨	STABILIZATION PLANTING

PLANTING KEY

⬡ XX / #	TREE SPECIES QUANTITY
◯ x / #	SHRUB SPECIES QUANTITY

VAN GAAL DRAIN RE-ALIGNMENT & CHANNEL ENHANCEMENTS
Richmond Village Development Corporation

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J.F. Sabourin and Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL
CONSULTANTS

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FAX: (613) 836-0332
WEB: www.jfsa.com

September 6, 2012

David Schaeffer Engineering Limited
120 Iber Road, Unit 203
Ottawa, Ontario K2S 1E9

Attention: **Mr. Adam Forbert, P.Eng.**

Subject: **Richmond Village Development / Proposed Realignment of Van Gaal Drain** *our file:922-11*

As requested by your office, we have evaluated, based on the available information as described below, the preliminary channel dimensions required to contain the 100-year design water levels within the proposed realignment of the Van Gaal Drain. It is understood that approximately 900 m of the existing Van Gaal Drain upstream of Perth Street will be realigned to follow the boundary of the Richmond Village Development Corporation site in the Village of Richmond.

In undertaking this work, the following information was considered:

1) HEC-RAS models of Van Gaal Drain under existing conditions (spring and summer) were obtained from the *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* (JFSA, November 2009). The November 2009 *Floodplain Mapping Report* defined the maximum flood levels in the Van Gaal Drain based on three scenarios: (1) the Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River; (2) The Van Gaal 100-year spring snowmelt plus rainfall peak flow reaches the Jock River; and (4) The Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain.

2) The HEC-RAS models of existing conditions were modified to reflect the proposed channel realignment based on information provided by DSEL. As noted above, it is proposed that approximately 900 m of the existing Van Gaal Drain upstream of Perth Street be realigned to follow the boundary of the Richmond Village Development Corporation site. Note that in order to best define the upstream limit of the channel realignment, existing conditions cross-sections were interpolated in HEC-RAS every 50 m between existing conditions cross-sections 2478 and 2157. The downstream limit of the channel realignment is defined by the Perth Street crossing. Refer to Figure 1 for the proposed channel realignment and cross-section locations.

3) Note that the proposed channel realignment will not impact flows in the Van Gaal Drain. As such, flows provided in the existing conditions HEC-RAS models were also used for the proposed conditions models.

4) The proposed channel dimensions were set to contain the 100-year flood levels within the channel for all three spring and summer scenarios, and to set the 100-year proposed conditions water levels at comparable cross-sections (2554, 2478, and 1615 - 1340) equal to or less than the maximum existing conditions flood levels defined in the November 2009 *Floodplain Mapping Report*. Note that 100-year water levels within the majority of the realigned channel are not comparable to existing conditions, given the different locations of the existing and proposed cross-sections.

5) The required low flow channel dimensions for the realigned channel are currently unknown. As such, a 0.3 m deep low flow channel with 0.3 m bottom width, 1H:1V side slopes and 0.9 m top width was assumed for the purposes of this analysis. The top of the low flow channel (beginning of the floodplain) was set to match existing minimum channel elevations at the upstream and downstream limits of the proposed channel

realignment. It should therefore be noted that the bottom of the proposed low flow channel is 0.3 m below the invert of the Perth Street culvert.

6) Two typical cross-sections were sized for the proposed channel realignment; Section A, from the upstream limit of the realignment to cross-section 1682; and Section B, from cross-section 1682 to the downstream limit of the realignment at Perth Street. Refer to Figure 2 for the typical proposed cross-section dimensions. Section A has a floodplain width of 7.5 m and a total depth of 1.7 m based on the existing grade of surrounding lands and the depth required to contain the 100-year design water levels in the channel. Section B has a floodplain width of 15.0 m and a variable depth; the top of the channel is set to match the existing grade of the surrounding land in order to ensure that the adjacent existing Cedarstone Subdivision is unaffected by the proposed channel realignment.

7) The existing conditions HEC-RAS models specify Manning's roughness coefficients of 0.035 for the low flow channel, 0.05 for the banks under spring conditions and 0.08 for the banks under summer conditions. These Manning's roughness coefficients are generally to be maintained in the proposed realigned channel. However, plantings are to be selected in Section B for a 5.0 m wide area of the proposed floodplain (centred around the low flow channel) to set a low Manning's roughness coefficient of 0.04 under both spring and summer conditions.

8) The entrance loss coefficient of Perth Street culvert was changed from 0.5 under existing conditions to 0.2 under proposed conditions, to represent proposed changes to the culvert entrance consisting of a headwall parallel to the embankment (no wingwalls) with three edges rounded to radius of 1/12 barrel dimension. This proposed conditions entrance loss coefficient is in accordance with the *HEC-RAS River Analysis System Hydraulic Reference Manual Version 4.1* (US Army Corps of Engineers, January 2010).

Based on the above information, 100-year design water levels for the proposed realignment of the Van Gaal Drain under the three spring and summer scenarios were determined using HEC-RAS and are presented in Table 1. The proposed conditions 100-year water levels are contained within the proposed channel for all scenarios. Furthermore, as may be seen in Table 1, the proposed conditions water levels at comparable cross-sections are equal to or less than the maximum existing conditions flood levels defined in the November 2009 *Floodplain Mapping Report*.

Yours truly,
J.F. Sabourin and Associates Inc.

Laura Pipkins, P.Eng.

cc: J.F. Sabourin, M.Eng, P.Eng.
Director of Water Resources Projects

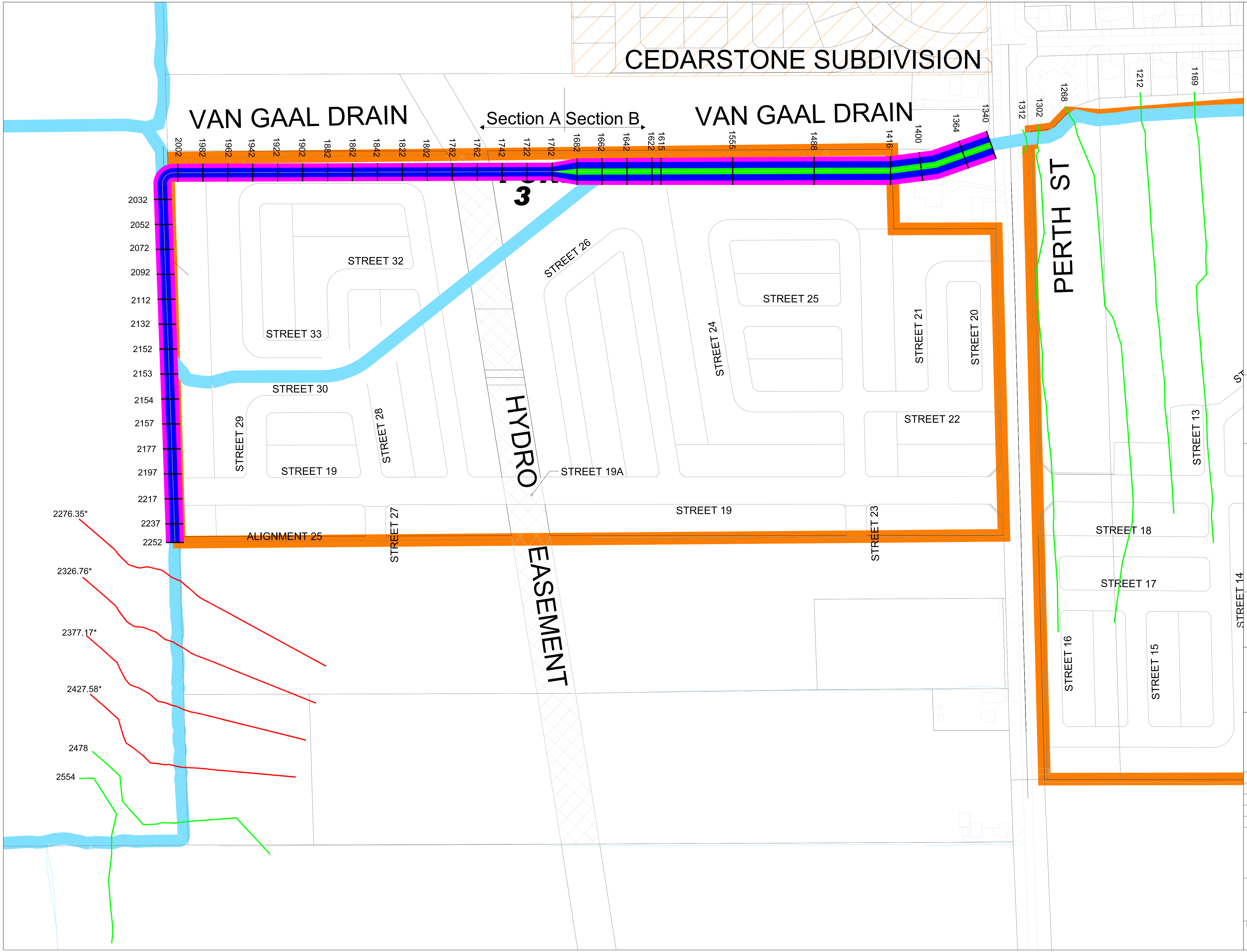
Table 1: Water Levels on Van Gaal Drain Reach 2 to Perth Street Under Proposed Conditions (1)

River Station	Scenario			
	1	*2*	*4*	*Max. Allowable* (2)
2554	96.26	96.28	95.86	96.28
2478	96.13	96.14	95.77	96.16
2427.58*	96.04	96.05	95.71	N/A
2377.17*	95.95	95.95	95.63	N/A
2326.76*	95.86	95.88	95.53	N/A
2276.35*	95.80	95.64	95.14	N/A
2252	95.77	95.59	95.04	N/A
2237	95.75	95.57	95.02	N/A
2217	95.72	95.55	94.99	N/A
2197	95.69	95.52	94.95	N/A
2177	95.67	95.49	94.92	N/A
2157	95.64	95.47	94.89	N/A
2154	95.62	95.44	94.85	N/A
2153	95.57	95.39	94.81	N/A
2152	95.52	95.35	94.77	N/A
2132	95.48	95.31	94.73	N/A
2112	95.44	95.26	94.69	N/A
2092	95.39	95.22	94.65	N/A
2072	95.35	95.18	94.61	N/A
2052	95.31	95.14	94.57	N/A
2032	95.26	95.10	94.53	N/A
2002	95.20	95.03	94.48	N/A
1982	95.15	94.99	94.44	N/A
1962	95.11	94.95	94.41	N/A
1942	95.06	94.90	94.38	N/A
1922	95.02	94.86	94.35	N/A
1902	94.97	94.82	94.32	N/A
1882	94.92	94.77	94.29	N/A
1862	94.87	94.73	94.27	N/A
1842	94.83	94.69	94.25	N/A
1822	94.78	94.64	94.23	N/A
1802	94.73	94.60	94.21	N/A
1782	94.67	94.56	94.19	N/A
1762	94.62	94.51	94.18	N/A
1742	94.56	94.47	94.17	N/A
1722	94.50	94.42	94.16	N/A
1702	94.44	94.37	94.15	N/A
1682	94.44	94.38	94.15	N/A
1662	94.42	94.37	94.15	N/A
1642	94.41	94.36	94.15	N/A
1622	94.40	94.35	94.14	N/A
1615	94.39	94.35	94.14	94.61
1555	94.37	94.33	94.14	94.55
1488	94.34	94.31	94.14	94.45
1416	94.32	94.30	94.14	94.41
1400	94.31	94.29	94.14	94.36
1364	94.30	94.29	94.13	94.31
1340	94.17	94.16	94.12	94.21

(1) Scenario Descriptions:

1. The Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River.
2. The Van Gaal Drain 100-year spring snowmelt plus rainfall peak flow reaches the Jock River.
4. The Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain.

(2) Maximum water level at existing cross-sections as per "*Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond*" (JFSA, November 2009)



LEGEND :

🟧 LIMITS OF SUBDIVISION

— PROPOSED CROSS–SECTION

— EXISTING CROSS–SECTION

— EXISTING (INTERPOLATED) CROSS–SECTION

— EXISTING CHANNEL ALIGNMENT

PROPOSED CHANNEL ALIGNMENT (SECTION A) :

- 2.5H:1V SECTION; 14.416 M TOTAL WIDTH
- 0.3 M WIDE LOW FLOW CHANNEL
- 7.5 M WIDE FLOODPLAIN SECTION

PROPOSED CHANNEL ALIGNMENT (SECTION B) :

- 2.5H:1V SECTION; VARIABLE TOTAL WIDTH
- 0.3 M WIDE LOW FLOW CHANNEL
- 5.0 M WIDE MINIMAL VEGETATION SECTION
- 15.0 M WIDE FLOODPLAIN SECTION

SCALE :

0 25 50 75 100 125 150m

J.F. Sabourin & Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS
OTTAWA (613) 836–3884
GATINEAU (819) 243–6858

CLIENT :

DSEL
david schaeffer engineering ltd
120 IBER ROAD, UNIT 203
OTTAWA, ONTARIO, K2S 1E9
(613) 836–0856

PROJECT :

PROPOSED REALIGNMENT OF
VAN GAAL DRAIN

	BY	DATE	DESCRIPTION	BY

LOCATION OF PROPOSED
CROSS–SECTIONS

FIGURE 1

DESIGNED:
DRAWN: LP
VERIFIED: JFS
APPROVED: JFS

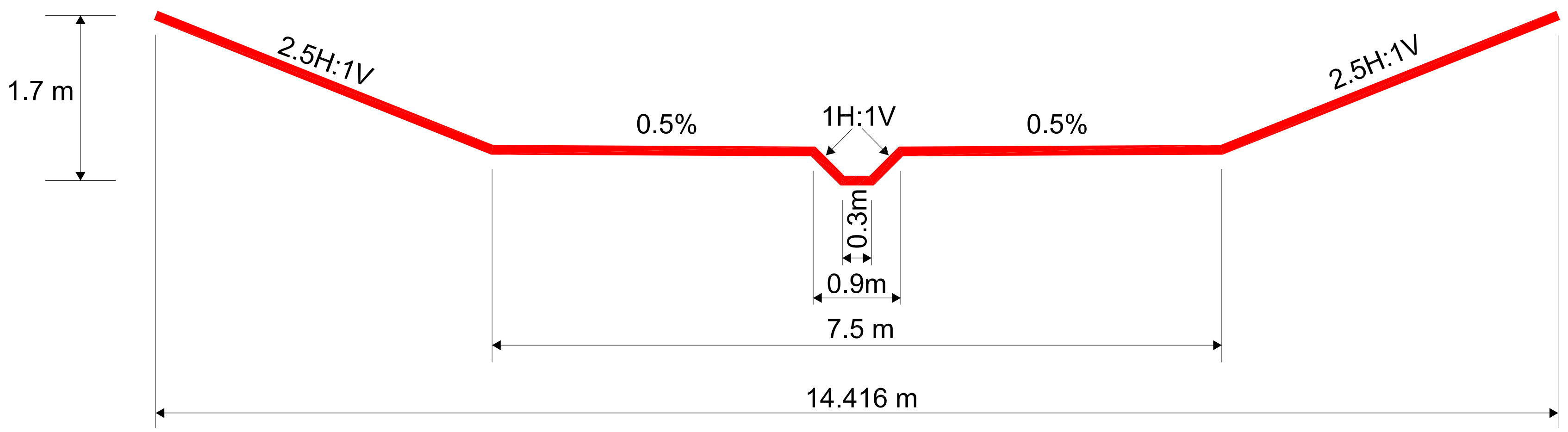
DRAWING REF.
922–11\201209 Channel Realignment\Design\CAD\JFSA Figures.dwg

DATE	PROJECT No.
Sep/12	922–11

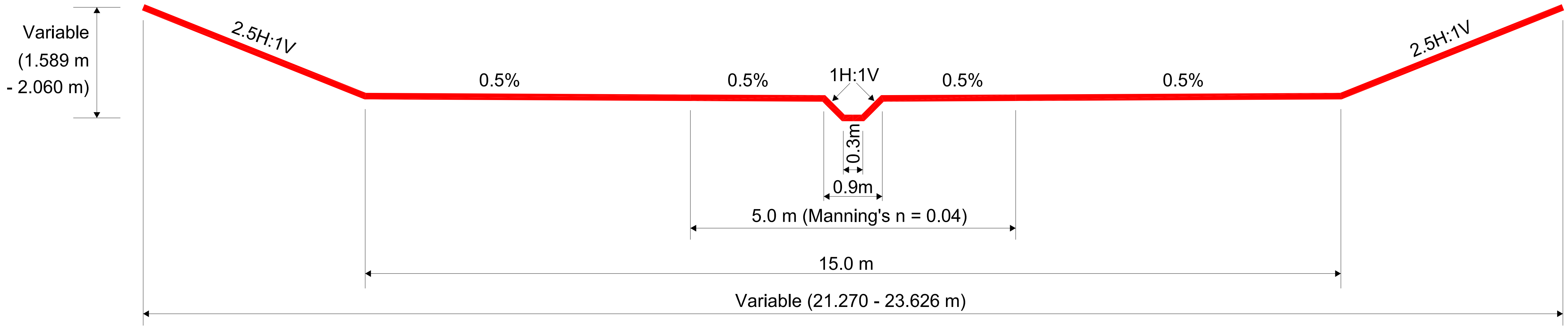
LEGEND :

🟥 PROPOSED CROSS–SECTION

SECTION A :



SECTION B :



SCALE :

0 50 100 150 200 250 300 350 400cm

J.F. Sabourin & Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS
OTTAWA (613) 836–3884
GATINEAU (819) 243–6858

CLIENT :

DSEL
david schaeffer engineering ltd
120 IBER ROAD, UNIT 203
OTTAWA, ONTARIO, K2S 1E9
(613) 836–0856

PROJECT :

PROPOSED REALIGNMENT OF
VAN GAAL DRAIN

	BY	DATE	DESCRIPTION	BY

DIMENSIONS OF PROPOSED
CROSS–SECTIONS

FIGURE 2

DESIGNED:
DRAWN: LP
VERIFIED: JFS
APPROVED: JFS

DRAWING REF.
922–11\201209 Channel Realignment\Design\CAD\JFSA Figures.dwg

DATE: Sep/12

PROJECT No.: 922–11

JTBES
JTB Environmental Systems Inc.
Fluvial Geomorphology Natural Channel Design Coastal Processes Erosion Control

September 6, 2012

David Schaeffer Engineering Limited
120 Iber Road, Unit 203,
OTTAWA, Ontario K2S 1E9

Attention: Mr. Adam Fobert, P. Eng

Subject: Richmond Village Development: Van Gaal Drain Geomorphology Assessment

---

JTBES was contacted by DSEL to complete an erosion threshold analysis for the Van Gaal drain. A previous erosion analysis did not appear to factor in sensitivity to erosion for the reach between Perth Street and Fortune Avenue, and it did not include any information between Fortune Avenue and the Jock River outlet.

Further, an assessment of the potential for realigning the existing drain upstream of Perth Street to accommodate site development has also been requested. The overall benefit of the realignment relative to maintaining the existing condition is discussed.

REALIGNMENT OF THE VAN GAAL DRAIN UPSTREAM OF PERTH STREET

A realignment concept has been provided which shows the drain shifting from its current alignment across the property to a position along the boundary of the property. This realignment will result in an overall straightening of the drain corridor; however the low flow function of the drain will be designed to show a sinuous path.

The result of this shift in position will be an increase in the overall stream length of the drain. Normally there would be concerns with sedimentation as energy budgets are affected by such realignments, particularly in low-gradient sections. However, based on our site visits in 2012 it is possible to create a series of steeper and gentler sections along the realigned drain to maintain sediment transport relationships and overall drain function.

A low flow channel will be incorporated into the design. The purpose of the low flow is to provide for fish habitat; the purpose of the overbank areas will be to provide velocity control which will aid in erosion protection. The overall design will be a positive outcome considering the existing condition of the drain. This is discussed further below.

CAUSES OF EXISTING EROSION ON THE VAN GAAL DRAIN

Erosion on the Drain is caused by a number of factors, some of which are natural and others which are induced as a result of changes in land use/hydrologic behaviour.

Natural erosion delivers sediment to the Drain through a number of possible mechanisms, including sheetwash, frost heave and desiccation fracturing, gravity failures due to oversteepening of the banks, and natural weathering of the clays (caused by repeated wet/dry cycles) which weakens the structural bonds in the clay matrix. Once operated upon by these mechanisms flowing water is easily able to erode and transport this weakened material. Large clumps of bank, once in the active channel, get quickly broken down into constituent clay particles which are cohesionless and very susceptible to erosion by flowing water.

Further, as banks become partly separated by slumps, flow from upstream can get behind the failing portion of the bank and hydraulic pressure forces the bank to fail more quickly. All of these processes (with the exception of frost heaving due to the time of year of the assessment) were visible along the drain.

Induced changes are the result of human activity upstream in the watershed. In this case, a recent small residential development along Rochelle Drive/Mira Court upstream of Perth Street may have changed the hydrologic properties of overland flow in the area. The increased impervious surface, and apparent lack of stormwater detention, delivers runoff quickly to the Drain during storms, creating a first pulse of fast flow through the Drain during storm events. When additional storm inlets are added to the mix (for instance the drain at Queen Charlotte Street) the cumulative effect of this fast rise in stage and velocity could result in erosion.

Finally, there is the construction of the Drain itself to consider. It is not known whether the Drain cross section was designed using flow analysis at the time of design (though the fact that the 2-year flow is well outside the cross-sectional area of the Drain under current conditions may be telling); if the necessary hydrologic calculations were not used to size the Drain then there is a chance it is undersized. This is currently evident as the Drain appears to be widening in response to flows, though it may be a combination of factors that is causing this to occur and, given the upstream development, may have sent the Drain past a stability threshold and initiated erosion at the scale which is evident today.

Results from the geomorphic analysis indicate the Van Gaal Drain system is a sediment rich as well as energy rich (at times of flowing water) system, which erodes, transports and deposits bed material under rising and falling hydrographs under existing conditions.

Mobilization of bed materials which have been deposited from upstream will occur under almost all flows and should be encouraged as the system needs to flush out these large deposits of silts and clays. Establishing a threshold discharge based on these surficial deposits is not appropriate as their transport requires very low velocities. Since the ‘bedrock’ layer is comprised of tight, cohesive clays and is highly resistant to erosion by flowing water, that material is also unsuitable for establishing thresholds. Therefore, given these two points and the fact that the Drain is eroding its banks in multiple locations, the most appropriate erosion threshold for use in this analysis is the bank erosion threshold.

Erosion along banks can be caused by flows that exceed the theoretical critical velocity for entrainment of the cohesive bank material. Assessment of the conditions of the creek show that the banks are comprised of consolidated clay materials, ranging from coarse to fine clay. When these materials are exposed to flowing water, velocities of between 0.225 metres per second (coarse clay) and 0.400 metres per second (fine clay) are required to entrain (erode) these materials (ref. Hjulstrom, 1935).

IMPLICATIONS

Existing Erosion

The analysis clearly shows that there are significant erosion sites on the Van Gaal Drain that have their cause in a number of areas. Addition of stormwater flows from the proposed site, even if controlled to the threshold rate, will not prevent existing erosion from continuing. It must be well understood that existing erosion will continue to occur for the simple reason that once erosion scars develop in banks they become weak points and as such are susceptible to continual erosion unless an intervention is undertaken. For those sites downstream of the stormwater connection point erosion will continue to occur, the rates of erosion may be lower as there will be some control on velocities which does not occur at present. For those sites upstream of the connection point, the degree and rates of erosion that are currently occurring will continue and that material which is eroded into the Drain will transport to the downstream reach. This continual influx of high sediment loads from upstream will complicate the erodibility of the downstream section.

OVERALL BENEFIT OF THE REALIGNMENT UPSTREAM OF PERTH STREET

Given erosion conditions in the drain downstream of Perth Street, application of the erosion thresholds and re-design of the drain upstream will provide the following benefit:

1. The existing erosive conditions in the drain upstream of Perth Street will be remediated in the design;and
2. Downstream erosion will be mitigated somewhat through the design upstream through the use of erosion thresholds and design principles tasked with controlling erosional velocities

Existing erosion along the Drain will continue once the site has been developed and while the degree of erosion may be somewhat addressed using the thresholds outlined in this study, the fact of the matter is that all erosion will not cease.

This condition assumes the Drain remains in its current configuration. With that condition in place, application of the erosion thresholds will not exacerbate erosion. There is an opportunity to mitigate some of the existing erosion in Reach 3 in a manner which will reduce some stress on reaches 2 and 1, which would be an overall benefit to the Drain system. Considering use of the Drain by aquatic species there is an added benefit to modification of the form and function of the Drain from the homogeneous nature it currently displays to a more diverse state. This can be done using natural channel design principles.

Natural Channel Design Principles

The purpose of natural channel design is to create/modify a system that is not properly functioning to a state where it is more in equilibrium with processes acting upon it. Doing so builds into the system a natural resilience to flow variability which is found in all stable, functioning watercourse systems. With respect to the Drain, and specifically Reach 3 from Perth Street to Fortune Street, it currently is classified as a straight channel with steep, vertical eroding banks that prevent connection to a floodplain during frequent flow events. Low flow events occupy the same channel width as high flow events which means that, as the Drain dries up, water depth is spread over a wide area resulting in shallow water which is not conducive to fish health. In addition, the concentration of flow during multiple events concentrates energy and erosion. This combination of factors acts to further entrench the Drain and create sidewall erosion as it tries to create a stable form over time.

JTBES has modelled a three-stage channel as described above at a coarse level and finds that through natural channel design it is possible to contain stormwater velocities below the threshold value through the site. Doing so lessens pressure on un-restored reaches downstream.

Details of the channel have also incorporated the channel dimensions used by JFSA in their floodplain analysis and findings support the notion that a stable, functioning system can be developed for the realigned sections of the Van Gaal Drain. Upon acceptance in principle by approval agencies JTBES will complete the full design for submission and will provide documentation with respect to function and lessening of downstream impacts.

If I can provide any further details please contact me directly.

Respectfully Submitted,

John T. Beebe, PhD
JTB Environmental Systems Inc.
CAMBRIDGE, ON

Kilgour & Associates Ltd.
Environmental Consultants
2285C St. Laurent Blvd., Unit 16, Ottawa, Ontario, K1G 4Z6, 613.260.5555

2012 September 7 **Our File: 156**

Mr. Adam Fobert
David Schaeffer Engineering Ltd.
120 Iber Road, Unit 203
Ottawa, Ontario, K2S 1E9

Dear Mr. Fobert,

Reference: **Richmond Village Development, Van Gaal Drain Realignment.**

Per your request, we have reviewed the proposed realignment of the Van Gaal Drain for the Richmond Village Development Corporation, as illustrated in Figures 1 and 2 provided by JF Sabourin and Associates, and in the illustrations of the landscaping plans prepared by NAK Design.

Under a prior assignment in support of development in the area, we demonstrated that the feature can contain up to 16 species of fish (Kilgour & Associates and Parish Geomorphic, 2010). Northern Pike (*Esox lucius*) are known to inhabit the feature, while a pike spawning was feature created near Perth Street. Pike are considered a key species for the area, and any works to the channel should consider the spawning requirements of the species.

We also understand that the Van Gaal Drain was dry this year associated with reduced precipitation in June and July. The feature was very dry in 2008 as well. The feature, therefore, is best described as providing intermittent fish habitat. The existing feature has a bankfull width of ~ 5 m, and has no riparian zone providing shade (Kilgour & Associates and Parish Geomorphic, 2010).

The proposed realignment will lengthen the channel, and increase the wetted area during periods of high flow. The realigned feature will also have a low-flow channel that will enhance the fish habitat value for a longer period of time through dry periods. The floor of the realigned channel will is to be planted with a mixture of plants including a mixture of grasses and sedges that are selected to grow under wet and dry conditions.

The realignment of the feature should be enhance ecological functions of the feature and broader area. Northern pike can be anticipated to use the flat shelves, planted with grasses and sedges, for spawning in the spring when the shelves are inundated with water. Riparian canopy will result in greater shade, and reduce heating of the channel. The improved riparian corridor associated with the realigned feature will increase the movements of larger wildlife like deer, from the Jock River corridor through and to the Richmond Wetland to the west of the study area (assuming that other connections are also made). The various plantings within the corridor will increase the diversity of plant life, thus providing a greater diversity of potential habitats for avifauna.

Realignment of the feature presents a minor risk to fishes in the system. Assuming that fish habitat features are provided for in the design, we anticipate that all of the fish species that were demonstrated to be using the Van Gaal Drain will continue to use the drain after the realignment.

Regards,

KILGOUR & ASSOCIATES LTD.

Bruce Kilgour, PhD

Literature Cited

Kilgour & Associates Ltd., and Parish Geomorphic. 2010. Mattamy Richmond Lands, Natural Environment and Impact Assessment Study.

RIDEAU VALLEY
CONSERVATION AUTHORITY

3889 Rideau Valley Dr, Box 599
Manotick, Ontario K4M 1A5
Ph: (613) 692-3571 fax: 692-0831

MEMO

Date: October 16, 2012

From: Jocelyn Chandler

To: Adam Fobert. P.Eng., Stephen J. Pichette, P.Eng. DSEL

RE: **Richmond Floodplain Lands, Proposed Van Gaal Channel Re-Alignment.**

RVCA staff have reviewed the submission dated September 14, 2012 which proposes a preliminary design to realign and widen the Van Gaal Drain north of Perth Street. The intent is to ultimately redefine the boundaries of the 1:100 year floodplain associated with the Van Gaal Drain.

Staff provide the following comments for your consideration prior to submitting an application for full technical and regulatory policy review under O.Reg 174/06.

- Provide details on corridor width and setbacks showing that applicable policies are being achieved.
- Provide cross sections showing the 3 tier channel as described in the text. It is the understanding of staff that a three tier channel should incorporate a low flow channel, bank full channel (more frequent events) and floodplain.
- Calculation showing loss of direct surface runoff within the subject reach of the Van Gaal Drain once the proposed municipal storm system is in place and overland run-off no longer contributes to instream flow. This comment arises out of the removal of the conceptual swm pond at the upstream location. A longer channel may not be beneficial if there are substantially reduced flows through urbanization of adjacent lands.
- Habitat features built into the new channel should consider all species of fish that have been recorded in the existing channel. For example if there are species that rely in riffle habitat to carry out their life processes then they should be included within the design. Once the detail design stage commences a review of the local fish species should be undertaken and measures to incorporate those habitat types should be incorporated into the design. If the channel is improved adequately it is conceivable that other species from the Jock River might access the new channel that have not in the past.
- Consider including re-use of existing stream bed materials to seed new bed etc.
- Provide status regarding municipal drain. The applicant must have support of the Drainage Superintendent to either abandon the drain or undertake these works.
- Provide information on how the groundwater recharge and base flow contributions will be maintained.

- A statement, supported by necessary analysis, from the consultant must be provided that the design will not adversely impact the control of upstream or downstream flooding.
- The design of the culvert modifications should be described in detail, with appropriate reference to standard manuals/texts.

Required documents for review under O.Reg 174/06:

- Application forms. One for each land owner. (One consultant may act as the agent for all owners provided a letter of authorization is submitted for each).
- Municipal Drain status paperwork if abandoned or comments/authorization from Municipal Drainage Dept.
- Location of works plan showing affected property parcels & ownership (for physical works and related setback influences).
- Documentation regarding the imposition of the setbacks, if applicable, on neighbouring properties must be provided and proof of such resolution (ie. signed documents acknowledging and accepting impact of the setbacks on adjacent property from property owners).
- This application will trigger a floodplain amendment therefore all technical calculations and modelling to support the amendment should be provided.
- Grading plan and cross sections of proposed design
- Design drawings for changes to culvert under Perth St.
- Any information on changes upstream or downstream of subject area.
- Natural Channel Design plans including:
  - Cross-section, profiles and plan view
  - Planting species, amounts, density, location
  - Pools, riffles, instream structures.
  - Description of materials (aggregates, stream bed materials, etc).
- Location of municipal pathway in the corridor and the width must be show on plans.
- Sediment and Erosion Control Plan
- Phasing and Implementation Plan – when will this work be undertaken? When will the new channel be established and the connection to the up and downstream connections be made.
- Fish rescue/relocation plan
- Post effectiveness monitoring plan (5 years)
- Fee Level 5 = $2140.00

This memo has been prepared in consultation with and on behalf of RVCA Watershed Science and Engineering, and Regulatory staff. Technical report review fees will be invoiced separately. Please contact me if you have any questions or require additional information.

Jocelyn Chandler, M.Pl., RPP, MCIP
Planner, RVCA

APPENDIX D

Geomorphology

J.F. Sabourin and Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL
CONSULTANTS

52 Springbrook Drive
Ottawa (Stittsville), ON K2S 1B9
TEL: (613) 836-3884
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October 31, 2013

David Schaeffer Engineering Limited
120 Iber Road, Unit 203
Ottawa, Ontario K2S 1E9

Attention: **Kevin Murphy, P.Eng.**

Subject: **Richmond Village (South) Limited Subdivision /**
Continuous Erosion Analysis *our file: 922-11*

As requested by your office, we have performed, based on the available information described below, a continuous erosion analysis for Van Gaal Drain in the City of Ottawa under existing and proposed conditions.

As per the October 2013 *Richmond Village (South) Limited Subdivision / Preliminary Stormwater Management Analysis*, the proposed Richmond Village (South) development consists of a 126.81 ha drainage area to be treated by two Stormwater Management (SWM) facilities; SWM Facility 1 (91.82 ha at 51% imperviousness) discharging to Van Gaal Drain, and SWM Facility 2 (34.99 ha at 51% imperviousness) discharging to the Jock River. It should be noted that excess major system flows from 30.79 ha and 21.75 ha of the SWM Facility 1 drainage area will discharge directly to the Van Gaal Drain and Moore Drain Tributary, respectively. Additionally, of the undeveloped lands south of the proposed subdivision, 97.50 ha will drain through SWM Facility 1, 71.8 ha will drain through SWM Facility 2, and 94.2 ha will be conveyed through the subdivision by a tributary of the Moore Drain.

Existing drainage characteristics of the subject site, and of the undeveloped lands south of the proposed subdivision, are as per the *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* (November 2009, JFSA). Refer to the October 2013 *Preliminary SWM Analysis* memo for existing and proposed drainage plans and further details.

Continuous SWMHYMO models of the Van Gaal Drain under existing and proposed conditions were created for the purposes of this erosion analysis based on the single-event SWMHYMO models submitted with the October 2013 *Preliminary SWM Analysis* memo. Continuous modelling parameters were set as follows for both existing and proposed conditions:

APII=[50], APIK=[0.90]/day;	used to compute the Antecedent Precipitation Index during the continuous simulation. Without model calibration these are the default values.
IaREC=[6](hrs);	the time that it takes for the Initial Abstraction over pervious areas to recover during a dry period in undeveloped areas.
SMIN=[-1], SMAX=[-1](mm);	the negative values indicate that the storage volume in the SCS procedure will vary between the "S" determined for AMC I and AMC III conditions of the entered CN value in undeveloped areas.
SK=[0.03]/(mm);	a calibration coefficient that can typically vary from 0.01 to 0.3 for undeveloped areas. The higher the value, the more runoff generated. To set the baseline for existing conditions, we decided to take a value in the low range.

InitGWResVol=[100](mm), GWResK=[0.9](mm/day/mm), VhydCond=[1](mm/hr);	parameters that are used to simulate both the groundwater storage and discharge to surface watercourses from undeveloped areas. Without adequate field measurements, these parameters were selected based on previous experience.
IaRECper=[3](hrs);	the time that it takes for the Initial Abstraction over pervious areas to recover during a dry period in urban areas.
IaRECimp=[2](hrs);	the time that it takes for the Initial Abstraction over impervious areas to recover during a dry period in urban areas.
InterEventTime=[12](hrs);	the continuous dry time required to reset the parameters in the SCS procedure to their initial values.

Under existing and proposed conditions, by means of 36 years of continuous hydrologic simulations using hourly rainfall data from the Ottawa International Airport from 1967 to 2003 (excluding missing 2001 rainfall data), flows at the Fortune Street erosion site were computed and compared. It should be noted that restoration works are proposed for this critical erosion site and several other points along the Van Gaal Drain. The erosion thresholds at Fortune Street were set at 60 L/s, 151 L/s and 385 L/s, as provided by Coldwater Consulting Limited to correspond to their critical shear stress thresholds of 0.5 Pa, 1.0 Pa and 2.0 Pa.

Based on the 60 L/s erosion threshold, erosion occurs for 479.45 hours and 596.88 hours in an average year under existing and proposed conditions, respectively; that is, for 9.87% and 12.29% of the total simulation duration. This corresponds to a 24.5% increase in erosion under proposed conditions. Similarly, based on the 151 L/s erosion threshold, erosion occurs for 328.44 hours and 362.94 hours in an average year under existing and proposed conditions, respectively; that is, for 6.76% and 7.47% of the total simulation duration. This corresponds to a 10.5% increase in erosion under proposed conditions. Finally, based on the 385 L/s erosion threshold, erosion occurs for 203.81 hours and 201.84 hours in an average year under existing and proposed conditions, respectively; that is, for 4.20% and 4.15% of the total simulation duration. This corresponds to a 1.0% decrease in erosion under proposed conditions.

A summary of the erosion analysis results may be found in Attachment A. Digital SWMHYMO modelling input and output files are also attached.

Yours truly,
J.F. Sabourin and Associates Inc.

L. Pipkins

Laura Pipkins, P.Eng.

cc: J.F. Sabourin, M.Eng, P.Eng.
Director of Water Resources Projects

Attachment A: Simulated Annual Erosion Hours in the Van Gaal Drain at Fortune Street

ATTACHMENT

A

Simulated Annual Erosion Hours
In the Van Gaal Drain at Fortune Street

JFSA Water Resources and Environmental Consultants

SIMULATED ANNUAL EROSION HOURS IN THE VAN GAAL DRAIN AT FORTUNE STREET

Table 1A: Existing Conditions (60 L/s Erosion Threshold)

Year [1]	Duration (h)	Peak Flow (m^3/s)	Average Flow (m^3/s)	Erosion Hours (h)	Total Exceedance (%)
1967	2544	6.576	0.129	373.00	14.66
1968	5160	5.096	0.068	488.80	9.47
1969	5160	5.825	0.049	382.50	7.41
1970	5160	6.150	0.058	452.30	8.77
1971	5160	5.267	0.051	419.80	8.14
1972	5160	10.071	0.111	668.30	12.95
1973	5160	7.253	0.088	585.50	11.35
1974	4392	3.542	0.038	324.00	7.38
1975	3696	5.493	0.080	444.50	12.03
1976	5160	3.728	0.045	421.80	8.17
1977	5160	5.297	0.062	512.00	9.92
1978	5160	5.782	0.050	486.00	9.42
1979	5160	8.422	0.107	608.80	11.80
1980	5160	3.583	0.054	544.80	10.56
1981	5160	21.687	0.142	701.80	13.60
1982	5160	5.226	0.043	442.30	8.57
1983	5160	6.867	0.054	465.50	9.02
1984	3696	4.399	0.069	394.00	10.66
1985	5160	3.375	0.045	446.80	8.66
1986	5160	9.724	0.120	731.00	14.17
1987	5160	6.648	0.068	494.80	9.59
1988	5160	7.423	0.066	462.00	8.95
1989	5160	4.226	0.045	413.80	8.02
1990	5160	7.061	0.079	542.80	10.52
1991	5160	4.841	0.051	442.30	8.57
1992	5160	9.789	0.072	490.50	9.51
1993	5160	1.914	0.048	532.50	10.32
1994	4416	4.772	0.077	471.80	10.68
1995	2952	12.479	0.098	248.30	8.41
1996	5136	4.963	0.052	427.50	8.32
1997	5160	1.542	0.029	396.50	7.68
1998	5088	2.992	0.044	428.00	8.41
1999	4440	4.772	0.052	450.50	10.15
2000	5160	7.496	0.064	502.30	9.73
2002	5088	12.146	0.095	535.80	10.53
2003	4440	5.617	0.09	527.30	11.88
Average	4858	6.446	0.069	479.45	9.87

(1) Based on a simulation period from April 1st to October 31st.

SIMULATED ANNUAL EROSION HOURS IN THE VAN GAAL DRAIN AT FORTUNE STREET

Table 1B: Proposed Conditions (60 L/s Erosion Theshold)

Year [1]	Duration (h)	Peak Flow (m^3/s)	Average Flow (m^3/s)	Erosion Hours (h)	Total Exceedance (%)
1967	2544	4.540	0.110	458.80	18.03
1968	5160	3.532	0.063	615.30	11.92
1969	5160	4.144	0.046	463.00	8.97
1970	5160	4.088	0.054	567.80	11.00
1971	5160	3.474	0.048	528.30	10.24
1972	5160	7.493	0.102	867.00	16.80
1973	5160	4.845	0.081	745.80	14.45
1974	4392	2.280	0.037	387.30	8.82
1975	3696	3.628	0.073	575.50	15.57
1976	5160	2.671	0.043	513.30	9.95
1977	5160	3.518	0.059	629.80	12.21
1978	5160	3.804	0.048	589.30	11.42
1979	5160	6.585	0.099	805.30	15.61
1980	5160	2.665	0.051	640.30	12.41
1981	5160	17.112	0.128	893.00	17.31
1982	5160	3.685	0.043	534.00	10.35
1983	5160	5.072	0.051	573.80	11.12
1984	3696	3.216	0.063	472.50	12.78
1985	5160	2.345	0.044	550.50	10.67
1986	5160	7.984	0.110	938.30	18.18
1987	5160	4.851	0.064	623.30	12.08
1988	5160	4.984	0.062	582.80	11.29
1989	5160	2.795	0.044	519.30	10.06
1990	5160	5.063	0.073	687.50	13.32
1991	5160	3.393	0.048	521.00	10.10
1992	5160	6.769	0.067	612.00	11.86
1993	5160	1.350	0.048	636.30	12.33
1994	4416	3.266	0.071	605.30	13.71
1995	2952	10.216	0.084	308.50	10.45
1996	5136	3.443	0.049	536.80	10.45
1997	5160	1.037	0.029	474.00	9.19
1998	5088	1.986	0.042	519.30	10.21
1999	4440	3.255	0.051	542.00	12.21
2000	5160	5.238	0.060	622.30	12.06
2002	5088	9.318	0.09	686.30	13.49
2003	4440	4.038	0.082	662.00	14.91
Average	4858	4.658	0.064	596.88	12.29

(1) Based on a simulation period from April 1st to October 31st.

Avg. Change in Erosion Threshold Exceedance: 24.5%

SIMULATED ANNUAL EROSION HOURS IN THE VAN GAAL DRAIN AT FORTUNE STREET

Table 2A: Existing Conditions (151 L/s Erosion Threshold)

Year [(1)]	Duration (h)	Peak Flow (m^3/s)	Average Flow (m^3/s)	Erosion Hours (h)	Total Exceedance (%)
1967	2544	6.576	0.129	288.00	11.32
1968	5160	5.096	0.068	349.30	6.77
1969	5160	5.825	0.049	266.30	5.16
1970	5160	6.150	0.058	295.80	5.73
1971	5160	5.267	0.051	292.80	5.67
1972	5160	10.071	0.111	468.00	9.07
1973	5160	7.253	0.088	402.30	7.80
1974	4392	3.542	0.038	206.30	4.70
1975	3696	5.493	0.080	307.80	8.33
1976	5160	3.728	0.045	284.00	5.50
1977	5160	5.297	0.062	361.00	7.00
1978	5160	5.782	0.050	323.00	6.26
1979	5160	8.422	0.107	446.30	8.65
1980	5160	3.583	0.054	369.00	7.15
1981	5160	21.687	0.142	464.50	9.00
1982	5160	5.226	0.043	281.00	5.45
1983	5160	6.867	0.054	295.80	5.73
1984	3696	4.399	0.069	297.30	8.04
1985	5160	3.375	0.045	322.50	6.25
1986	5160	9.724	0.120	502.50	9.74
1987	5160	6.648	0.068	313.30	6.07
1988	5160	7.423	0.066	327.00	6.34
1989	5160	4.226	0.045	272.00	5.27
1990	5160	7.061	0.079	374.00	7.25
1991	5160	4.841	0.051	273.50	5.30
1992	5160	9.789	0.072	338.30	6.56
1993	5160	1.914	0.048	348.30	6.75
1994	4416	4.772	0.077	328.80	7.45
1995	2952	12.479	0.098	185.50	6.28
1996	5136	4.963	0.052	278.00	5.41
1997	5160	1.542	0.029	256.50	4.97
1998	5088	2.992	0.044	295.30	5.80
1999	4440	4.772	0.052	306.80	6.91
2000	5160	7.496	0.064	343.80	6.66
2002	5088	12.146	0.095	393.80	7.74
2003	4440	5.617	0.09	365.30	8.23
Average	4858	6.446	0.069	328.44	6.76

(1) Based on a simulation period from April 1st to October 31st.

SIMULATED ANNUAL EROSION HOURS IN THE VAN GAAL DRAIN AT FORTUNE STREET

Table 2B: Proposed Conditions (151 L/s Erosion Theshold)

Year (1)	Duration (h)	Peak Flow (m^3/s)	Average Flow (m^3/s)	Erosion Hours (h)	Total Exceedance (%)
1967	2544	4.540	0.110	315.50	12.40
1968	5160	3.532	0.063	388.00	7.52
1969	5160	4.144	0.046	282.00	5.47
1970	5160	4.088	0.054	329.30	6.38
1971	5160	3.474	0.048	306.50	5.94
1972	5160	7.493	0.102	559.50	10.84
1973	5160	4.845	0.081	468.30	9.08
1974	4392	2.280	0.037	199.50	4.54
1975	3696	3.628	0.073	344.30	9.32
1976	5160	2.671	0.043	294.00	5.70
1977	5160	3.518	0.059	391.00	7.58
1978	5160	3.804	0.048	350.50	6.79
1979	5160	6.585	0.099	527.50	10.22
1980	5160	2.665	0.051	408.30	7.91
1981	5160	17.112	0.128	542.50	10.51
1982	5160	3.685	0.043	295.50	5.73
1983	5160	5.072	0.051	325.80	6.31
1984	3696	3.216	0.063	325.30	8.80
1985	5160	2.345	0.044	360.30	6.98
1986	5160	7.984	0.110	567.50	11.00
1987	5160	4.851	0.064	349.00	6.76
1988	5160	4.984	0.062	365.80	7.09
1989	5160	2.795	0.044	291.50	5.65
1990	5160	5.063	0.073	418.30	8.11
1991	5160	3.393	0.048	300.50	5.82
1992	5160	6.769	0.067	377.30	7.31
1993	5160	1.350	0.048	367.30	7.12
1994	4416	3.266	0.071	364.00	8.24
1995	2952	10.216	0.084	201.80	6.84
1996	5136	3.443	0.049	297.50	5.79
1997	5160	1.037	0.029	259.00	5.02
1998	5088	1.986	0.042	321.00	6.31
1999	4440	3.255	0.051	341.30	7.69
2000	5160	5.238	0.060	378.00	7.33
2002	5088	9.318	0.09	447.00	8.79
2003	4440	4.038	0.082	405.30	9.13
Average	4858	4.658	0.064	362.94	7.47

(1) Based on a simulation period from April 1st to October 31st.

Avg. Change in Erosion Threshold Exceedance: 10.5%

SIMULATED ANNUAL EROSION HOURS IN THE VAN GAAL DRAIN AT FORTUNE STREET

Table 3A: Existing Conditions (385 L/s Erosion Threshold)

Year [(1)]	Duration (h)	Peak Flow (m^3/s)	Average Flow (m^3/s)	Erosion Hours (h)	Total Exceedance (%)
1967	2544	6.576	0.129	205.80	8.09
1968	5160	5.096	0.068	231.30	4.48
1969	5160	5.825	0.049	171.80	3.33
1970	5160	6.150	0.058	188.30	3.65
1971	5160	5.267	0.051	171.50	3.32
1972	5160	10.071	0.111	276.00	5.35
1973	5160	7.253	0.088	273.80	5.31
1974	4392	3.542	0.038	109.00	2.48
1975	3696	5.493	0.080	176.80	4.78
1976	5160	3.728	0.045	161.30	3.13
1977	5160	5.297	0.062	225.30	4.37
1978	5160	5.782	0.050	194.80	3.78
1979	5160	8.422	0.107	283.30	5.49
1980	5160	3.583	0.054	203.30	3.94
1981	5160	21.687	0.142	306.00	5.93
1982	5160	5.226	0.043	170.00	3.29
1983	5160	6.867	0.054	177.00	3.43
1984	3696	4.399	0.069	191.80	5.19
1985	5160	3.375	0.045	193.50	3.75
1986	5160	9.724	0.120	302.50	5.86
1987	5160	6.648	0.068	182.50	3.54
1988	5160	7.423	0.066	220.00	4.26
1989	5160	4.226	0.045	155.50	3.01
1990	5160	7.061	0.079	268.00	5.19
1991	5160	4.841	0.051	177.30	3.44
1992	5160	9.789	0.072	205.80	3.99
1993	5160	1.914	0.048	212.50	4.12
1994	4416	4.772	0.077	227.00	5.14
1995	2952	12.479	0.098	104.00	3.52
1996	5136	4.963	0.052	161.30	3.14
1997	5160	1.542	0.029	131.00	2.54
1998	5088	2.992	0.044	184.00	3.62
1999	4440	4.772	0.052	183.00	4.12
2000	5160	7.496	0.064	207.30	4.02
2002	5088	12.146	0.095	262.00	5.15
2003	4440	5.617	0.09	243.00	5.47
Average	4858	6.446	0.069	203.81	4.20

(1) Based on a simulation period from April 1st to October 31st.

SIMULATED ANNUAL EROSION HOURS IN THE VAN GAAL DRAIN AT FORTUNE STREET

Table 3B: Proposed Conditions (385 L/s Erosion Theshold)

Year (1)	Duration (h)	Peak Flow (m^3/s)	Average Flow (m^3/s)	Erosion Hours (h)	Total Exceedance (%)
1967	2544	4.540	0.110	211.30	8.31
1968	5160	3.532	0.063	228.30	4.42
1969	5160	4.144	0.046	169.80	3.29
1970	5160	4.088	0.054	175.00	3.39
1971	5160	3.474	0.048	171.30	3.32
1972	5160	7.493	0.102	295.50	5.73
1973	5160	4.845	0.081	278.00	5.39
1974	4392	2.280	0.037	105.00	2.39
1975	3696	3.628	0.073	174.80	4.73
1976	5160	2.671	0.043	153.00	2.97
1977	5160	3.518	0.059	222.30	4.31
1978	5160	3.804	0.048	183.00	3.55
1979	5160	6.585	0.099	287.00	5.56
1980	5160	2.665	0.051	185.00	3.59
1981	5160	17.112	0.128	319.80	6.20
1982	5160	3.685	0.043	158.00	3.06
1983	5160	5.072	0.051	170.30	3.30
1984	3696	3.216	0.063	191.50	5.18
1985	5160	2.345	0.044	187.00	3.62
1986	5160	7.984	0.110	316.50	6.13
1987	5160	4.851	0.064	182.50	3.54
1988	5160	4.984	0.062	221.50	4.29
1989	5160	2.795	0.044	152.00	2.95
1990	5160	5.063	0.073	277.00	5.37
1991	5160	3.393	0.048	172.30	3.34
1992	5160	6.769	0.067	207.80	4.03
1993	5160	1.350	0.048	197.80	3.83
1994	4416	3.266	0.071	233.80	5.29
1995	2952	10.216	0.084	95.80	3.25
1996	5136	3.443	0.049	154.00	3.00
1997	5160	1.037	0.029	112.80	2.19
1998	5088	1.986	0.042	179.30	3.52
1999	4440	3.255	0.051	182.50	4.11
2000	5160	5.238	0.060	208.30	4.04
2002	5088	9.318	0.09	273.30	5.37
2003	4440	4.038	0.082	233.30	5.25
Average	4858	4.658	0.064	201.84	4.15

(1) Based on a simulation period from April 1st to October 31st.

Avg. Change in Erosion Threshold Exceedance: -1.0%

Erosion Hazard Assessment:
Van Gaal Drain, Richmond, ON

Submitted to: David Schaeffer Engineering Limited

Prepared by: M.H. Davies, Ph.D., P.Eng.

Coldwater Consulting Ltd.
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(613) 747-2544

Version: 1.3

Date: 1 November 2013

Version History

Version	Date	Status	Comments	Reviewed by	Approved by
0.91	3 October 2013	For discussion with DSEL	Completed 1st draft	NJM	MHD
1.0	10 Oct 2013	For discussion with RVCA and City	Working Draft	MHD	MHD
1.1	15 Oct 2013	For discussion with RVCA and City	Working Draft	MHD	MHD
1.2	15 Oct 2013	For discussion with RVCA and City	Working Draft	MHD	MHD
1.3	1 Nov 2013	For Client Use	Final Report	MHD	MHD

For further information please contact:

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Provisos

This report was prepared by Coldwater Consulting Ltd. The recommendations and opinions contained in this report are based upon a limited dataset and a limited scope of work. The material contained herein reflects the judgement of Coldwater Consulting Ltd. in light of the information available to them at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Parties. Coldwater Consulting Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

Submitted 1 November 2013,

M.H. Davies, Ph.D., P.Eng.
Coldwater Consulting Ltd.

LICENSED PROFESSIONAL ENGINEER
01 Nov 2013
M. H. DAVIES
PROVINCE OF ONTARIO

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EXECUTIVE SUMMARY

An analysis has been undertaken of present erosion conditions in the van Gaal Drain in Richmond, ON. Using hydrologic analysis supplied by J.F. Sabourin & Associates, Coldwater Consulting Ltd. has assessed erosion thresholds for the length of the drain extending from Perth Street downstream to the Jock River. An existing HEC-RAS one-dimensional flow model was refined over the study area and used to compute bed shears and erosive power. Using a 36-year continuous hydrologic simulation (supplied by JFSA) and a critical bed shear stress of 0.5 Pa, it is shown that while the number of hours that erosion thresholds are exceeded will increase by 12% post-development, these increases mostly occur during low flow conditions. The effects of flood attenuation by stormwater management facilities offset this effect, resulting in a net decrease in erosional power (as quantified by the Erosion Index) by 21%.

Although conditions post-development will be an improvement over existing conditions, the drain will continue to experience erosion. Conceptual designs are presented herein for protection and restoration measures that will protect critical areas – notably in the immediate vicinity of Fortune St. and downstream of Fowler St.

1 Introduction

Coldwater Consulting Ltd., (Coldwater) has been engaged to provide the following analysis and design services to David Schaeffer Engineering Ltd (DSEL):

- Review previous studies of the Van Gaal Drain, its present condition and hydraulic conditions as provided by DSEL;
- Conduct site investigations of the drain, its bed composition, morphology and bank characteristics for the reach (VGR1) extending from Perth Street downstream to its confluence with the Jock River; and,
- Develop a preliminary erosion hazard model and present quantitative estimates for any required restoration works.

1.1 Background Information

The following documents were referenced when preparing this design brief:

- Report from JFSA "Floodplain mapping report for the van Gaal and Arbuckle municipal drains in the village of Richmond", provided by DSEL dated November 2009.
- Report by Robinson Consultants (Robinson) for the City of Ottawa on the Arbuckle Municipal Drain dated February 2010
- Report by Parish Geomorphic (Parish) and Kilgour and Associates (Kilgour) for Mattamy Homes on the Mattamy Lands natural environment and assessment on impacts from development dated February 2010
- Report from JTB Environmental Systems Inc. " Van Gaal Drain Erosion Assessment, Richmond, Ontario", provided by DSEL dated October 2012.
- Memorandum from JTB Environmental Systems Inc. "Richmond Village Development: Existing Erosion Remediation Costs", provided by DSEL dated November 2012.
- Report from JTB Environmental Systems Inc. " Van Gaal Drain Restoration Memo, Richmond, Ontario", provided by DSEL dated January 2013.
- HEC-RAS model developed and provided by JFSA, September 2013.
- Continuous time series of discharge for pre- and post-development at 4 junctions (provided by JFSA, September 2013),
- JFSA report on continuous erosion analysis and updated pre-and post development hydrology (JFSA file 922-11, dated 18 Oct, 2013),
- Stormwater Management Planning and Design Manual, (Ontario MoE, March 2003),
- Cross-sectional data (Arbuckle drain cross sections.dwg) surveyed by J.D.Barnes and provided by DSEL.

2 Background

2.1 Previous Work

Six reports dealing with Van Gaal Drain and surrounding area were reviewed in the preparation of the present report. These reports are the first six documents listed in Section 1.1.

Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond (JFSA, 2009)

The JFSA report details the flood risk mapping that was performed for the Arbuckle and Van Gaal Drains. Hydrologic analyses were performed using the SYMHYMO model and hydraulic analyses using the HEC-RAS model. Based on a review of earlier DSEL models, the drainage area was estimated at 1147 ha. The report notes that access to certain areas could be obtained and so it was necessary to assume cross-sectional profiles based on earlier data at these sections. Channel sections were modelled using a Manning's n value of 0.035 for the channels, and 0.08 for the summer floodplain and 0.05 for the spring floodplain.

Three 100-year spring and summer event discharges were studied and the maximum discharge from the Van Gaal Drain at the Jock River was found to be 16.419 m^3/s. The 2-year spring and summer event discharges varied ranged from 5.666 m^3/s to 7.883 m^3/s. Flood risk elevations were computed at various stations along the river from the maximum of three values (Van Gaal Drain spring flood, Van Gaal Drain summer flood, Jock River flood).

Engineer's Report, Arbuckle Municipal Drain Modifications and Improvements, Goulbourn Ward (Robinson, 2010)

The Robinson report notes that while the Arbuckle Award Drain has existed from the late 1800s, the Van Gaal Municipal Drain was only constructed in 1971. Based on contour mapping, the drainage area was estimated at 1095 ha. SYMHYMO modelling was also performed and 2-year return period flows at various points in the Van Gaal Drain were within the range determined be JFSA (2009); the 100-year event was not investigated. The report details recommendations for:

- works to improve drainage, including culvert replacement and re-leveling, and land clearing and excavation;
- erosion control, such as buffer strips, rock protection, rootwads and revetment, and;
- flow checks and sediment traps.

The report also contains a discussion of the apportioning of costs for construction and future maintenance. The discussion identifies six principles that should be used to determine assessment and then applies the principles and rules from the Drainage Act to determine project cost sharing.

Mattamy Richmond Lands Natural Environment and Impact Assessment Study, (Parish and Kilgour, 2010)

> This report covers a broad range of environmental topics, including an erosion threshold analysis. The erosion threshold analysis was conducted at four sites; however, only one site ("VG-R2") was located on the main branch of the Van Gaal Drain and this site was upstream of Perth St, the limit of the present analysis. The analysis was based on critical shear stress and permissible velocities, and found that the calculated erosion thresholds for the four reaches were discharges well below bankfull. At the VG-R2 site, the critical discharge was calculated to be 0.33 m^3/s.

Van Gaal Drain Erosion Assessment, (JTBES, 2012a)

> This report describes work that was undertaken to investigate erosion downstream of Perth Street, a reach not specifically investigated in the earlier Mattamy report (Parish and Kilgour, 2010). This study delineated the drain into three reaches: Reach 1 running between the Jock River and the Fowler Culvert; Reach 2 running between the Fowler Culvert and the Fortune Culvert, and; Reach 3 running between the Fortune Culvert and the Perth Culvert. Following on a site visit, 51 erosion assessment sites were investigated and categorized, and, based on this, four sites were chosen for erosion threshold assessment. The threshold was determined as the critical velocity for the bank material, which in this case is coarse clay. The critical velocity for coarse clay was given as 0.225 m/s, which led to critical discharges for the four sections ranging from 0.02 to 0.05 m^3/s.

Richmond Village Development: Existing Erosion Remediation Costs, (JTBES, 2012b)

> The purpose of this memo was to summarize the causes of erosion along the Van Gaal Drain and to estimate costs to remediate the erosion. It was concluded that, under existing conditions, Reach 3 had the most severe erosion and that, although not all sites required remediation, repairs at selected locations could simply shift the problem to a downstream site. It was deemed preferable to remediate all sites at once. It was also noted that even if the additional stormwater flows were limited to the threshold rate, erosion of the Drain would continue to occur. Consequently, a redesign for the section south of Perth Street was recommended. The cost for all 51 sites identified previously was estimated to be $1.41 million.

Van Gaal Drain Restoration Memo, (JTBES, 2013)

> The purpose of this memo was to detail remediation costs for the erosion problems on the Van Gaal Drain downstream of Perth St. The work expands on the information provided in the earlier memo (JTBES, 2012b) and identifies 32 sites for remediation (4 in Reach 1, 7 in Reach 2 and 21 in Reach 3). Descriptions of works at the sites are presented. No reassessment of the erosion threshold is attempted; however, it is noted that after the proposed remediation works were completed, the threshold discharges for the site could be increased an undefined amount.

2.2 Review

The report and memos cited in the previous section provide valuable information about the previous work performed on the Van Gaal Drain. Of paramount importance here are the four last documents, which address erosion threshold analyses and provide a basis for the design of the stormwater management system and the required treatment of the existing drain.

A problem with the approaches taken by both Parish and JTBES is that even when using reasonable critical shear stress values, calculations performed in narrow channels will predict erosive conditions for almost all flows. However, erosion is a natural process and the erosion threshold in the channel should be exceeded to maintain a healthy system. It is not the aim of the development works to eliminate erosion, but to ensure that post-development conditions do result in a substantial increase in erosion. Clearly, a more sophisticated approach that integrates the impact of all events is required. The present work will examine not just the frequency with which the erosion threshold is exceeded but also the total amount of erosion that occurs both pre- and post-project.

In common with both the Parish and JTBES approaches, the present work will require an erosion threshold for the Van Gaal Drain. The first erosion threshold analysis (Parish and Kilgour, 2010) utilized a permissible tractive force technique to establish a critical discharge. Although the resulting discharge may appear to be low, this is a valid geomorphic approach and an approach based on similar principles will be employed in the present work. Loose, clayey soils are competent below unit tractive forces, or shear stresses, ranging between 0.5 Pa and 2 Pa (Chow, 1959). This range of critical shear stresses will be used for the modelling work presented herein.

3 Field Investigation

A site visit to Van Gaal Drain was conducted on 26 August 2013 to review and characterize the site. The day was sunny with some cloudy periods with daytime high of 24°C. The last recorded precipitation was 22 August 2013. Photos and notes regarding the channel geometry were taken at numerous locations at this time. A rapid geomorphic assessment and a rapid stream assessment were also taken during the site visit.

The field investigation found several sites with significant erosion on the Van Gaal Drain. This agrees with previous studies. For most of the reach between Perth St. and Jock River, the channel is too narrow for the current hydrologic conditions. The existing drain will continue to erode and widen until it reaches equilibrium - even without the proposed storm management plan. Figure 1 through Figure 4 show examples of erosion in the Van Gaal Drain. Figure 5 shows the location of each photo.

Figure 1 Typical undercutting banks between Perth St. and Fortune St.

Figure 2 Eroding bank downstream of Fortune St.

Figure 3 Bank widening upstream of Fowler St.

Figure 4 Eroding banks downstream of Fowler St.

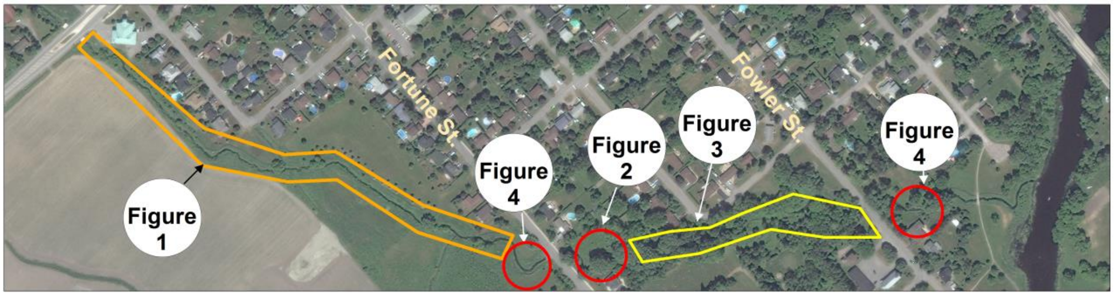


Figure 5 Locations of photographs

4 Erosion Model

Flow conditions in the Van Gaal drain were evaluated under existing conditions (pre-project) as well as under fully developed conditions (post-project) which includes the proposed storm water management pond. As noted in previous studies (JTB 2012, DSEL 2009), the erosion thresholds for this reach are very low and hence are exceeded frequently. In such situations, it is often beneficial to examine not just the frequency with which the erosion threshold is exceeded but also the total amount of erosion that occurs both pre- and post-project. By computing the amount by which the erosion threshold is exceeded, its duration, and the area of stream-bed affected, the erosional effort of 'effective work' can be computed (MOE, 2003).

This measure, in comparison to frequency of exceedance analysis, provides a more complete picture of the erosional consequences of a project.

4.1 Methodology

An erosion hazard model was developed to estimate the erosion potential for pre- and post-development on the Van Gaal Drain. A cumulative effective work approach was developed In addition to analysis of the frequency of exceedance of erosion thresholds. This model computes an erosion index (EI) based on the cumulative effective work, W_i The cumulative effective work is calculated as :

$$W_i = \sum (\tau - \tau_c) V \Delta t \qquad \text{(Eq. 1)}$$

where τ is the shear stress generated by the flow, τ_c is critical shear stress for either the bed or the bank, V is the mean channel velocity and Δt is the time step. For the present analysis, EI is multiplied by the wetted perimeter, P, to express the results as the total erosional energy across the channel width (Joules/m):

$$\mathrm{EI} = \sum (\tau - \tau_c) V P \Delta t \qquad \text{(Eq. 2)}$$

Calculating EI requires a continuous time series of discharge and a table relating discharge to shear stress, velocity and wetted perimeter. The continuous time series of discharge for pre- and post-development was provided by JFSA from their hydrologic model (JFSA, 2009). The dataset spans 36 years (April to October) between 1967 and 2003 with a time step of 15-minutes.

4.2 Critical Shear Stress

It is difficult to accurately determine the *in situ* critical shear stress for small streams. Typically, there is no single specific value that captures the range of erosion and transport processes that occur. Critical shear stress is dependent on a range of variables, include sediment size and type, weathering, vegetation, biological activity, etc. It can vary spatially for even small streams and can also be dependent upon weather conditions, freeze-thaw activities and exposure. The present modelling exercise was performed using several critical shear stress values that spanned the range of expected values. Based on the characteristics of the stream bed and banks, critical shear thresholds are estimated to be between 0.5 and 2.0 Pa.

Sensitivity tests were performed which showed that, while the magnitudes of the predicted erosion varied, the relative performance of the two scenarios tested (pre- and post-development) were unaffected by the value of critical shear stress selected.

4.3 HEC-RAS Model Refinement

The original HEC-RAS model (JFSA, 2009) covers a very large domain and was found not to have sufficient resolution in the reach downstream of Fortune St. New cross-sectional survey data was provided by DSEL and were incorporated into the HEC-RAS model by Coldwater.

Figure 6 shows the original model (with only one cross-section below Fowler St.) and Figure 7 shows the refined model (with 7 cross-sections below Fowler St.).

Figure 6 Original HEC-RAS model provided by JFSA with limited number of cross-sections d/s of Fortune St.

Figure 7 HEC-RAS model provided by JFSA with supplemental cross-sections added.

4.4 Threshold Flow Rates

Station 592, (located 592 m upstream of the Jock River) was identified as a critical erosion section. Using the refined HEC-RAS model, critical shear stresses of 0.5, 1 and 2 Pa were found to be associated with flow rates of 60, 158 and 385 L/s, respectively. These 'threshold flow rates' were transmitted to JFSA for their 'Continuous Erosion Analysis'.

4.5 Model Operation

The erosion model requires two types of input; the predicted discharge for the scenario being studied and station-specific values for shear stress, velocity and wetted perimeter as a function of discharge. These station-specific values were calculated using the refined HEC-RAS model. At each station, 22 discharges scenarios ranging between 0 m^3/s and 16.42 m^3/s were modelled to obtain these relationships (16.42m^3/s being the 1:100 year flow level). In total 726 (= 22 scenarios x 33 stations) sets of discharge, shear stress, velocity and wetted perimeter data were created to model the Van Gaal Drain between Perth St. and Jock River.

The erosion model was applied sequentially to each station. For each year, the erosion model stepped through the 15-minute time series of discharges from the JFSA hydrologic model. At each time step, the model interpolated the shear stress, velocity and wetted perimeter from the input discharge. These values were used to compute EI. The cumulative value of EI was also stored, as was the total time where the shear stress at the station was above critical.

Simulations were performed for both pre-development and post-development hydrographs. At noted in Section 4.2 above, the erosion model was run for a range of critical shear stresses. The results presented below are from the simulations with τ_c = 0.5 Pa.

5 Results

5.1 Single Event Simulation

This section shows comparisons of erosion potential for pre- and post-development on the Van Gaal Drain for a selected rainfall event, 9 April 1980 to 12 April 1980, at Station 592, using a critical shear threshold of 0.5 Pa.

Figure 8 shows the hydrograph for the event. The peak discharge decreases by 0.5 m^3/s from pre- to post-development. Pre-development discharge remains higher than post-development condition until it hits the 19.25 hour mark. The shear stress, shown in Figure 9, mimics the pattern. The shear stress is lower up until after the peak of the event. Shear stress is above critical until 31.75 hr for pre-development condition and until 40.5 hr for post-development case. Figure 10 shows the erosion index (EI) results; post-development EI remains lower than pre-development conditions until 19.25 hr. In total, there's 30.75 hours of erosion for the pre-development condition and 39.5 hours of erosion for the post-development condition. The cumulative erosion index for the event, a measure the impact of the event on the stream, is shown in Figure 11. Although the total duration of erosion for the event is longer in the post-development case, the cumulative EI is less; the cumulative EI for the pre-development case is 66.1 MJ and 50.8 MJ for the post-development case. This pattern was found to be repeated for the majority of the events at stations downstream of the retention pond (Stations 746 and lower). Stations upstream of this point showed lesser variation in the results of the two scenarios.

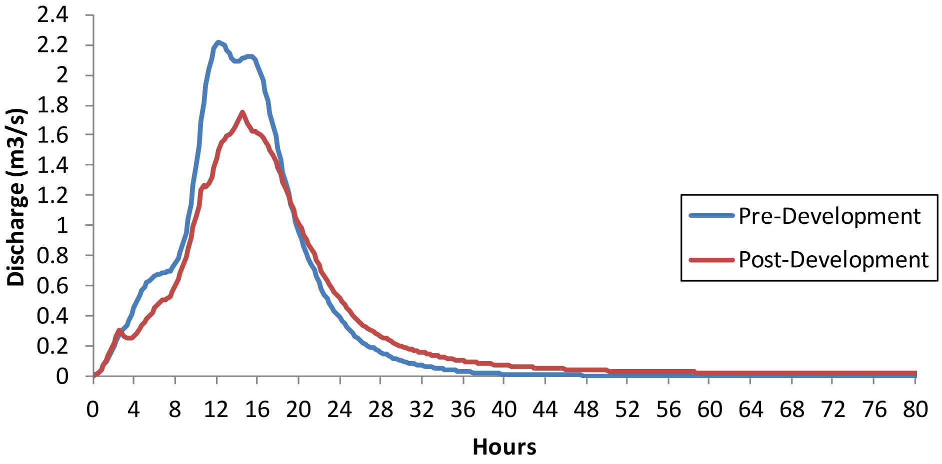


Figure 8 Continuous time series of discharge for pre- and post-development cases for the April 1980 event

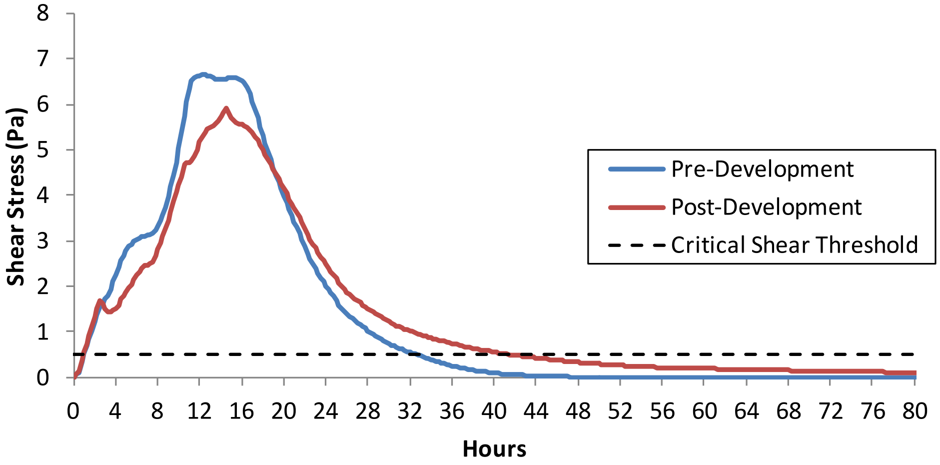


Figure 9 Continuous time series of shear stress for pre- and post-development cases for the April 1980 event

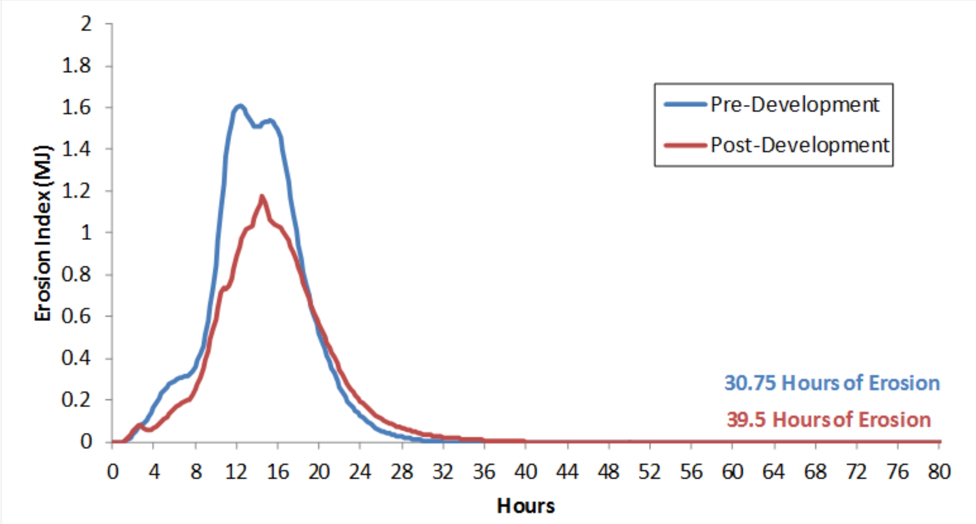


Figure 10 Continuous time series of erosion index for pre- and post-development cases for the April 1980 event

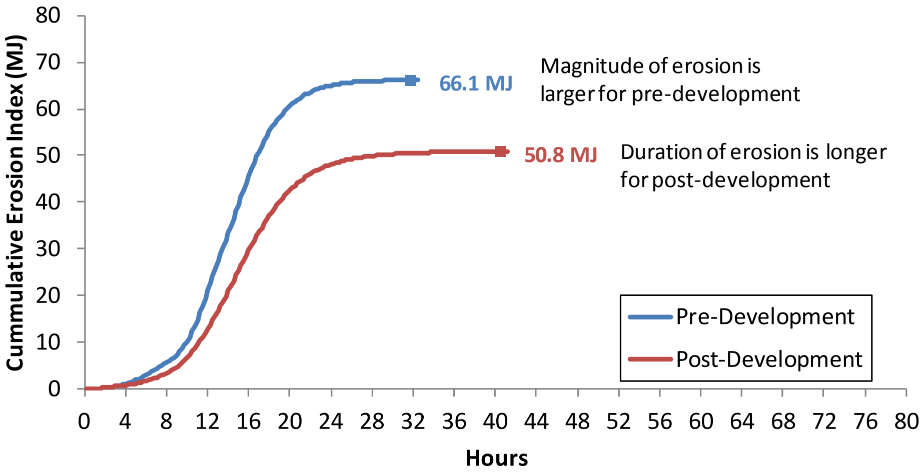


Figure 11 Cumulative erosion index for pre- and post-development cases for the April 1980 event

5.2 Long-term Simulations

The results in this section are for the model application at all stations for a 36-year simulation. Continuous time series of discharge for pre- and post-development (from SWMHYMO simulations) were provided by JFSA in September 2013 and an updated version in October 2013. The results from the extended HEC-RAS model were used to compute a 36-year analysis of flow and erosion conditions within the van Gaal drain (as per the methodology described in Section 4.4).

Figure 12 and Figure 13 show the average annual time above critical shear and the change in this measure computed as post-development minus pre-development. Upstream of Station 746 there is little change. Downstream of this point, the post-development case tends to spend more time in an erosional state. However, as was illustrated by the example in Section 5.1, this is not truly representative of the impact of the changes to the system on potential erosion of the stream. Figure 14 and Figure 15 show the annual average erosion index and change in this measure. These plots illustrate that the impact on the stream of the changes to the system lead to a reduction in erosion potential along the entire reach. This is illustrated in plan view in Figure 16.

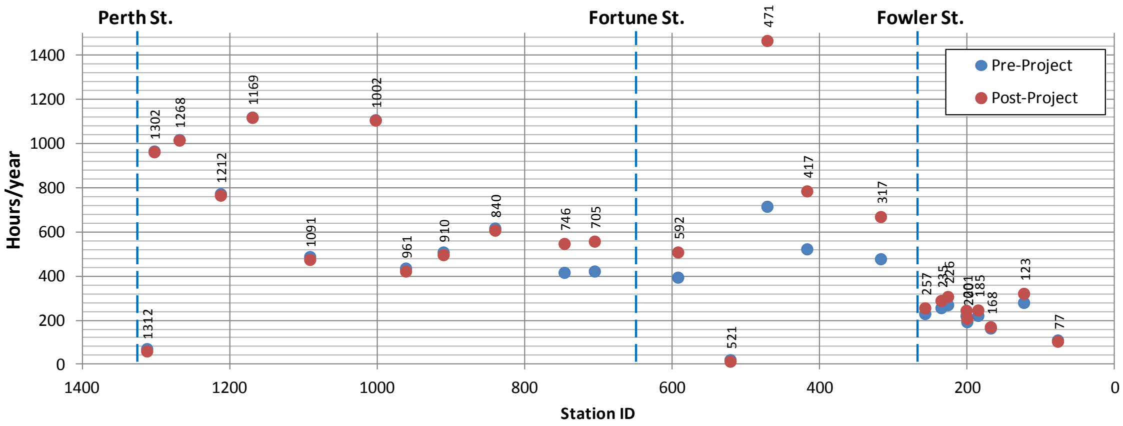


Figure 12 Average annual time above critical shear for a 36-year simulation

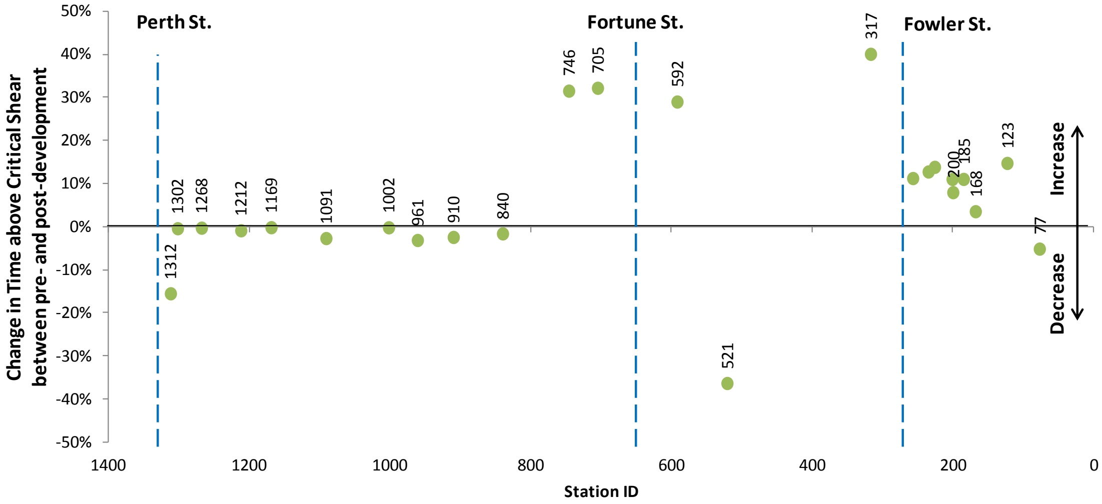


Figure 13 Change in annual time above critical shear for a 36-year simulation (post - pre)

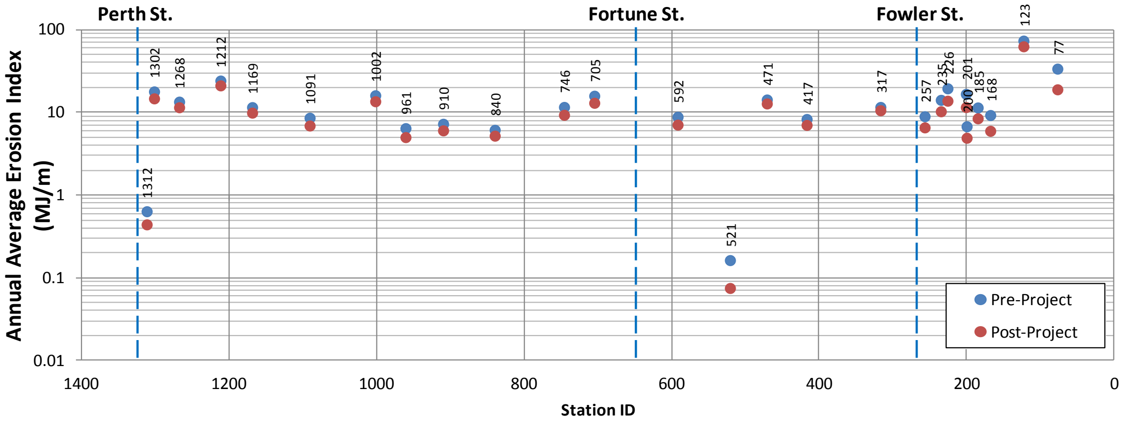


Figure 14 Annual average erosion index for a 36-year simulation

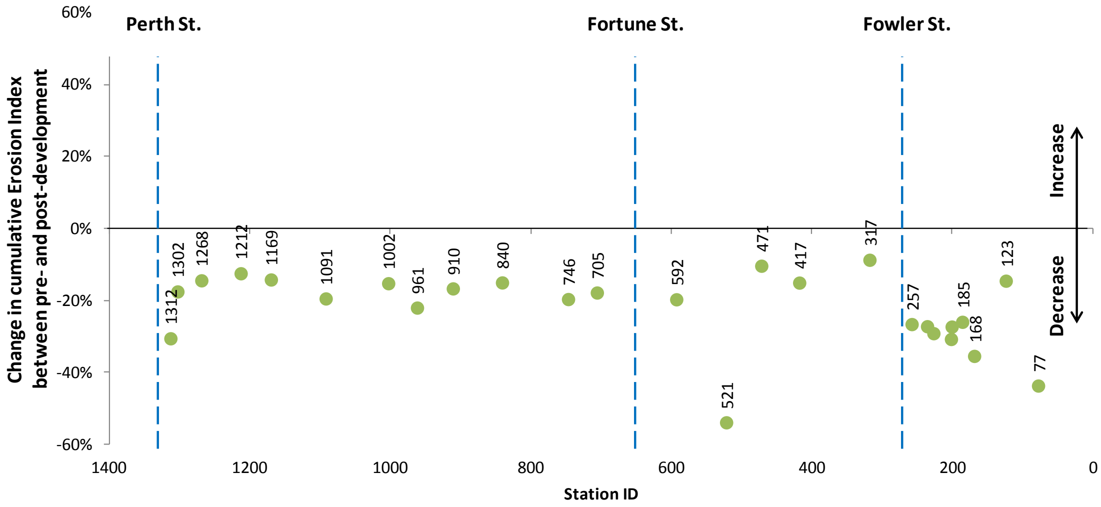


Figure 15 Change in erosion index for a 36-year simulation (post - pre)

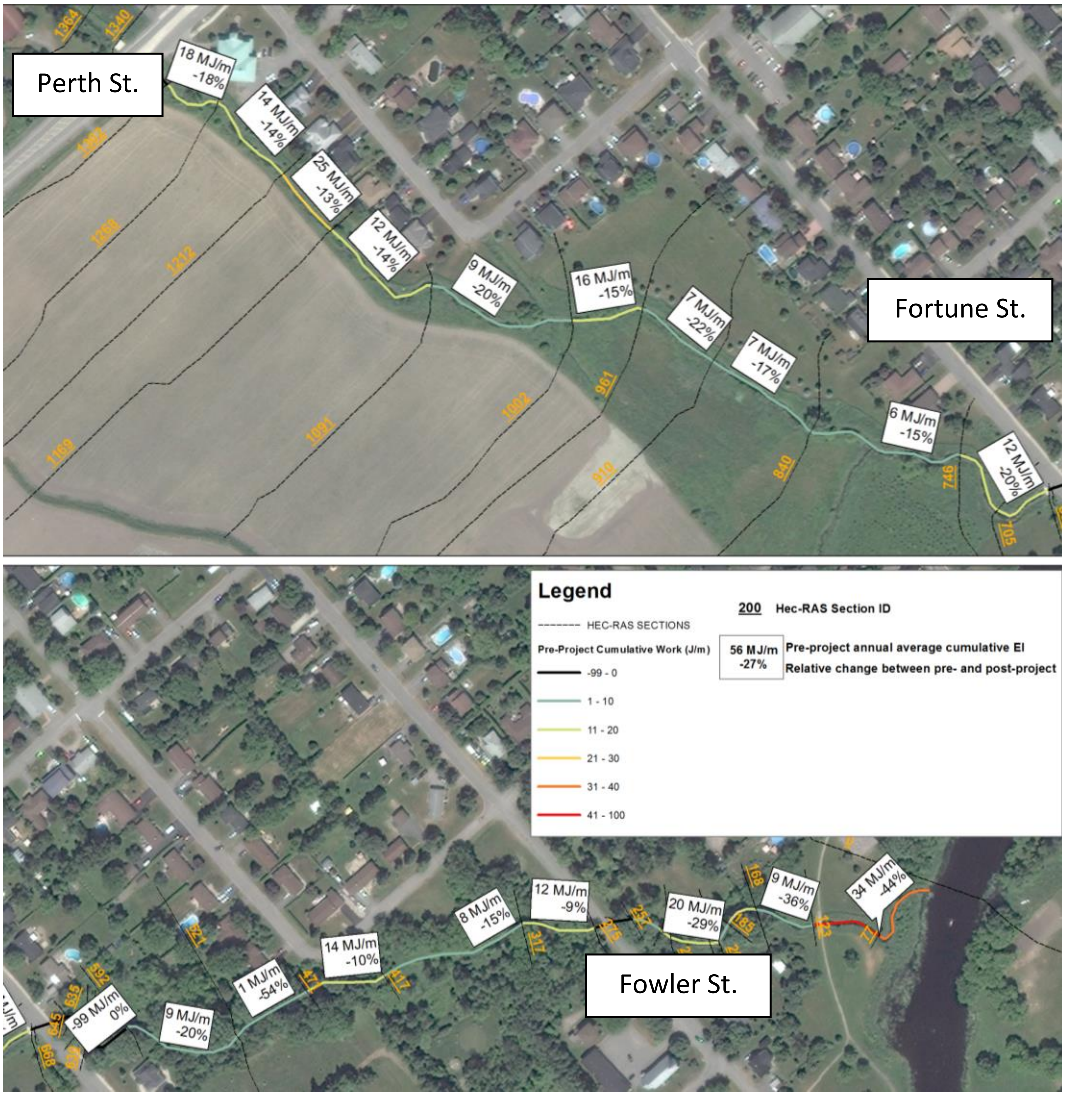


Figure 16 Annual average pre-development erosion index and change in erosion index over the 36-year simulation

The following table (Table 1) shows the annual average erosion index for existing and post-development conditions, alongside the annual hours above critical shear from the hydrologic analysis. It is important to note here that while patterns in total erosion threshold hours are the same between JFSA's 'continuous erosion analysis' and Coldwater's calculations, differences in the absolute numbers do exist. It is our understanding that the JFSA analysis was undertaken using a 1 hour timestep, while the Coldwater analysis was undertaken using a 15 minute timestep. JFSA's approach considers just one critical flow threshold, whereas the Coldwater approach looks at when a critical shear stress of 0.5 Pa occurs which varies spatially throughout the reach.

Looking at erosion simply in terms of hours above threshold gives the impression that erosion will be 12% worse under post-development conditions. This calculation reflects the fact that post-development will see a slight increase in the duration of low-flow conditions – but this calculation does not take into consideration the overall reduction in flow peaks.

The Erosion Index (indicating the erosional energy of the stream) is consistently reduced throughout the drain due to attenuation of flood peaks by implementation of the stormwater management plan. Looking at the entire study reach, Erosion Index is reduced by 21% relative to pre-project (existing) conditions.

Table 1 Average annual erosion index and average annual hours above critical shear

Station (Chainage u/s from Jock River)	Hours above 0.5 Pa threshold			Erosion Index (MJ/m)		
	Pre	Post	Δ	Pre	Post	Δ
77	108	101	-7%	34.2	18.9	-45%
123	278	312	12%	74.5	62.8	-16%
168	162	165	2%	9.4	6.0	-37%
185	220	239	9%	11.6	8.4	-27%
200	191	203	6%	6.9	4.9	-28%
201	218	238	9%	16.9	11.5	-32%
226	268	299	11%	19.8	13.8	-30%
235	255	282	11%	14.3	10.3	-28%
257	228	249	9%	9.1	6.6	-28%
317	476	628	32%	11.8	10.5	-10%
417	521	714	37%	8.4	7.0	-17%
471	714	1319	85%	14.5	12.6	-13%
521	19	12	-37%	0.2	0.1	-55%
592	393	489	24%	9.0	7.1	-21%
705	421	533	27%	16.1	13.0	-19%
746	415	524	26%	11.8	9.3	-21%
840	616	606	-2%	6.2	5.3	-15%
910	507	495	-2%	7.4	6.1	-17%
961	434	421	-3%	6.5	5.1	-22%
1002	1106	1105	0%	16.3	13.8	-15%
1091	486	473	-3%	8.7	7.0	-20%
1169	1118	1117	0%	11.7	10.0	-14%
1212	771	765	-1%	24.5	21.5	-13%
1268	1017	1014	0%	13.6	11.7	-14%
1302	965	961	0%	18.1	14.9	-18%
1312	69	58	-15%	0.6	0.4	-31%
Length-weighted annual averages:			+12.3%			-21.4%

6 Restoration and Protection Works

As described in JTB (2012), opportunities exist to implement restoration works in critical reaches in order to reduce erosion and to restore natural channel processes. Most notably these are required in the waters immediately upstream and downstream of the Fortune St. crossing as well as downstream of Fowler St.

As shown in Figure 17, conceptual restoration works have been developed for five specific sites. Bank restoration works are proposed both upstream of Fortune St. and downstream of Fowler St. This involves re-grading of the slopes, riprap bank protection and live stake plantings to re-establish vegetative cover.

The existing meander immediately downstream of Fortune St. is creating an erosional hotspot. Here, a channel re-alignment is recommended (Item C in the Figure 17) that will shift the channel away from the eroding bank. Cross-vanes will be used to develop riffle-pool structures that will stabilize the channel in its new location while also providing natural in-stream features.

The upstream wingwall for the Fowler St. crossing is presently being undercut, as identified in Item B in Figure 17, wingwall restoration works are proposed to remedy this situation.

Downstream of Fowler St., bank re-grading and revetment is required to protect the adjacent property on the west side of the stream. A small timber weir exists beneath the pedestrian bridge just upstream of this site and the remnants of a somewhat larger timber weir exist further downstream. It is quite possible that the loss of this downstream weir has significantly increased erosion upstream. We are proposing several cross-vane weir structures composed of quarrystone to replace and improve the function of these weirs. These structures will reduce channel slope in the area and help to stabilize the banks.

All of the proposed works will be undertaken within a framework of natural channel design. Even though riprap and slope re-grading will be required in several areas, measures will be incorporated in such a way that diversity and quality of aquatic habitat is enhanced through a combination of appropriate plantings and boulder features.

Preliminary cost estimates for these works are presented in Figure 18.

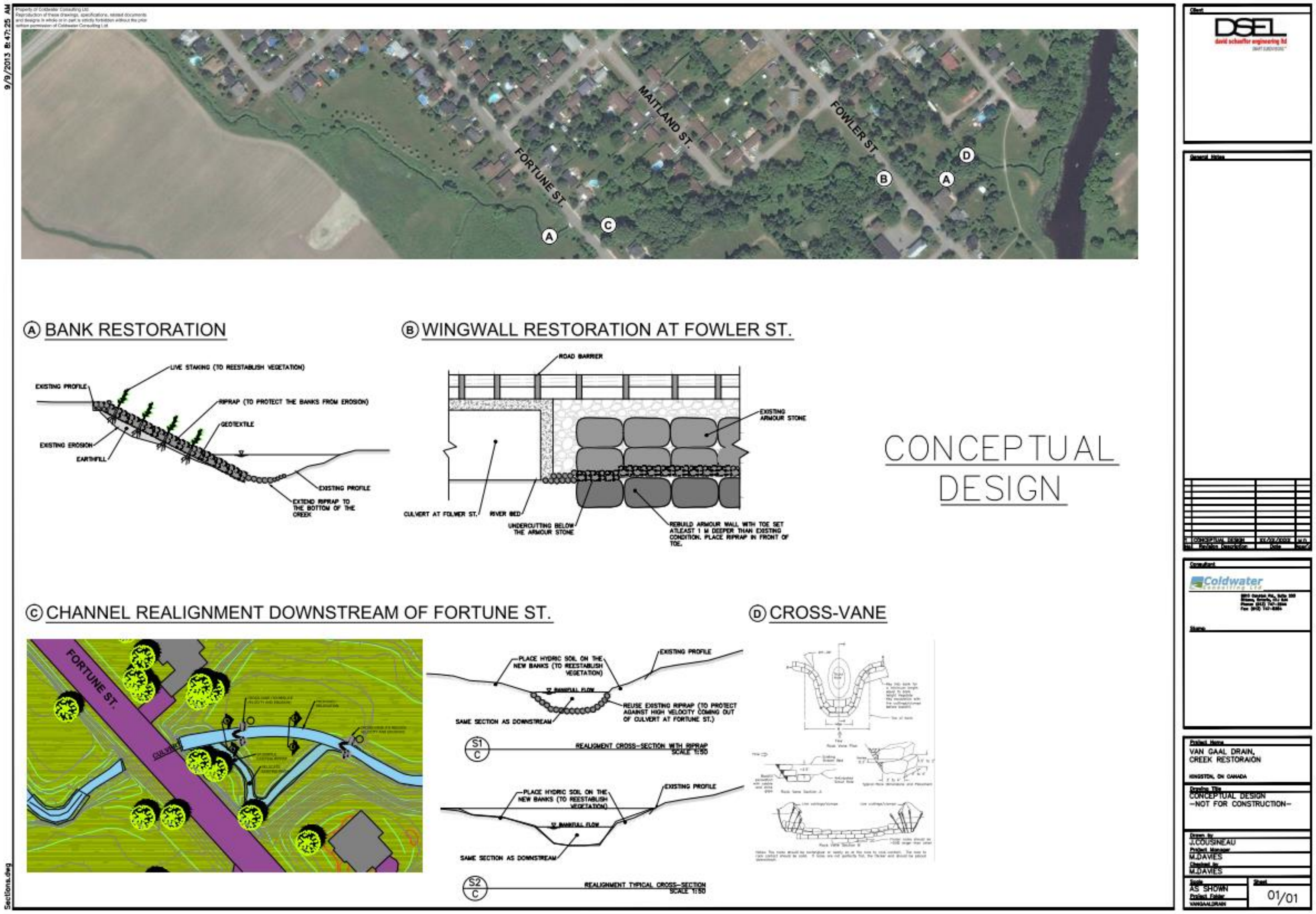
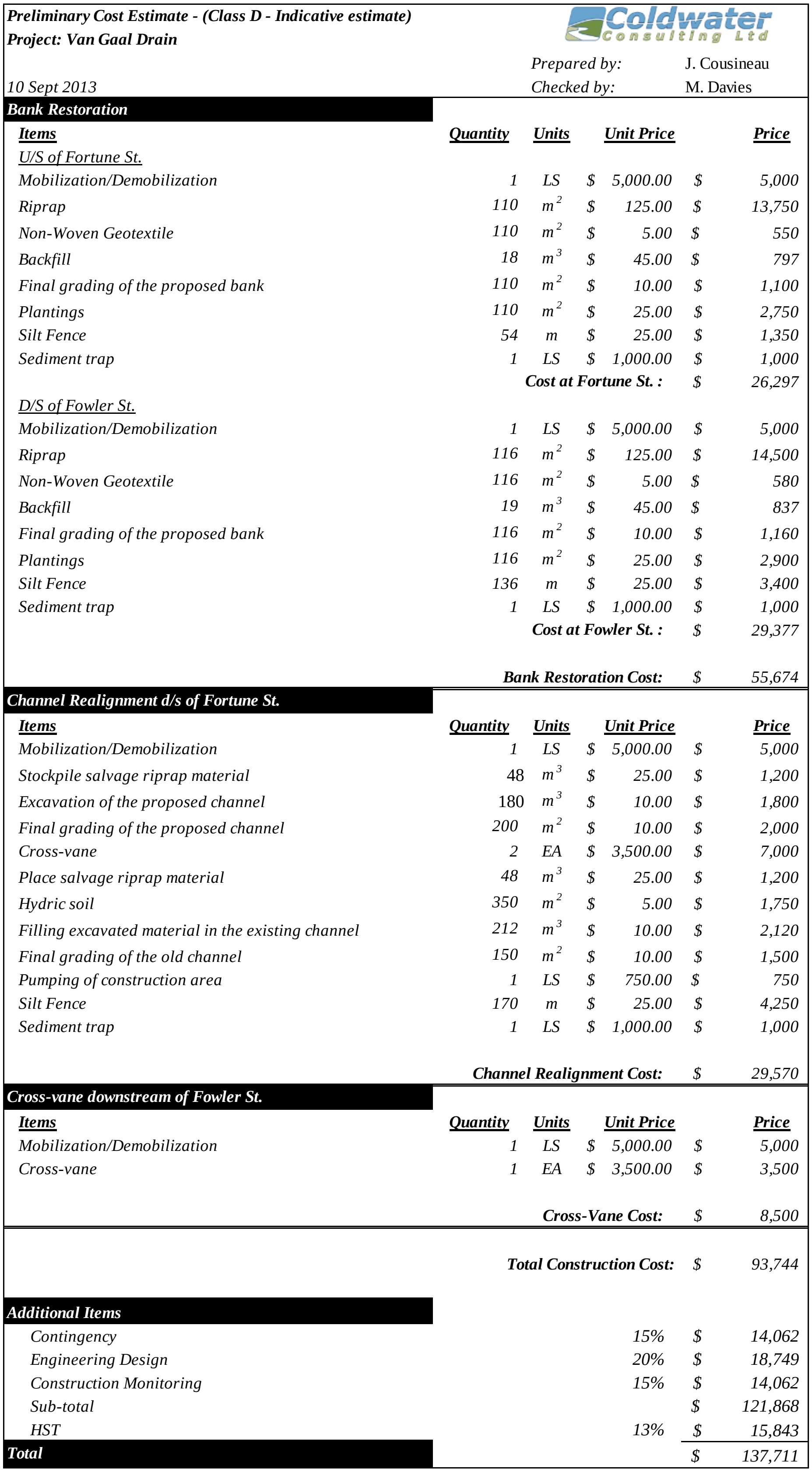


Figure 17 Restoration Conceptual Designs



Preliminary Cost Estimate - (Class D - Indicative estimate)

Project: Van Gaal Drain

Prepared by: J. Cousineau

10 Sept 2013 *Checked by:* M. Davies

Bank Restoration

Items	***Quantity***	***Units***	***Unit Price***	***Price***
U/S of Fortune St.				
Mobilization/Demobilization	*1*	*LS*	*$ 5,000.00*	*$ 5,000*
Riprap	*110*	*m^2*	*$ 125.00*	*$ 13,750*
Non-Woven Geotextile	*110*	*m^2*	*$ 5.00*	*$ 550*
Backfill	*18*	*m^3*	*$ 45.00*	*$ 797*
Final grading of the proposed bank	*110*	*m^2*	*$ 10.00*	*$ 1,100*
Plantings	*110*	*m^2*	*$ 25.00*	*$ 2,750*
Silt Fence	*54*	*m*	*$ 25.00*	*$ 1,350*
Sediment trap	*1*	*LS*	*$ 1,000.00*	*$ 1,000*
			Cost at Fortune St. :	*$ 26,297*
D/S of Fowler St.				
Mobilization/Demobilization	*1*	*LS*	*$ 5,000.00*	*$ 5,000*
Riprap	*116*	*m^2*	*$ 125.00*	*$ 14,500*
Non-Woven Geotextile	*116*	*m^2*	*$ 5.00*	*$ 580*
Backfill	*19*	*m^3*	*$ 45.00*	*$ 837*
Final grading of the proposed bank	*116*	*m^2*	*$ 10.00*	*$ 1,160*
Plantings	*116*	*m^2*	*$ 25.00*	*$ 2,900*
Silt Fence	*136*	*m*	*$ 25.00*	*$ 3,400*
Sediment trap	*1*	*LS*	*$ 1,000.00*	*$ 1,000*
			Cost at Fowler St. :	*$ 29,377*
			Bank Restoration Cost:	*$ 55,674*

Channel Realignment d/s of Fortune St.

Items	***Quantity***	***Units***	***Unit Price***	***Price***
Mobilization/Demobilization	*1*	*LS*	*$ 5,000.00*	*$ 5,000*
Stockpile salvage riprap material	48	*m^3*	*$ 25.00*	*$ 1,200*
Excavation of the proposed channel	180	*m^3*	*$ 10.00*	*$ 1,800*
Final grading of the proposed channel	*200*	*m^2*	*$ 10.00*	*$ 2,000*
Cross-vane	*2*	*EA*	*$ 3,500.00*	*$ 7,000*
Place salvage riprap material	*48*	*m^3*	*$ 25.00*	*$ 1,200*
Hydric soil	*350*	*m^2*	*$ 5.00*	*$ 1,750*
Filling excavated material in the existing channel	*212*	*m^3*	*$ 10.00*	*$ 2,120*
Final grading of the old channel	*150*	*m^2*	*$ 10.00*	*$ 1,500*
Pumping of construction area	*1*	*LS*	*$ 750.00*	*$ 750*
Silt Fence	*170*	*m*	*$ 25.00*	*$ 4,250*
Sediment trap	*1*	*LS*	*$ 1,000.00*	*$ 1,000*
			Channel Realignment Cost:	*$ 29,570*

Cross-vane downstream of Fowler St.

Items	***Quantity***	***Units***	***Unit Price***	***Price***
Mobilization/Demobilization	*1*	*LS*	*$ 5,000.00*	*$ 5,000*
Cross-vane	*1*	*EA*	*$ 3,500.00*	*$ 3,500*
			Cross-Vane Cost:	*$ 8,500*

	Total Construction Cost:	*$ 93,744*

Additional Items

Contingency	*15%*	*$ 14,062*
Engineering Design	*20%*	*$ 18,749*
Construction Monitoring	*15%*	*$ 14,062*
Sub-total		*$ 121,868*
HST	*13%*	*$ 15,843*
Total		*$ 137,711*

Figure 18 Planning level cost estimate - PRELIMINARY

7 Summary

The Van Gaal Drain is presently experiencing high levels of erosion. This erosion is expected to continue to erode until it reaches a new equilibrium, even without the proposed storm management plan.

From the erosion hazard model, the proposed storm water management plan is predicted to reduce erosion by an average of 21% - the drain will, however, continue to experience erosion.

Conceptual designs have been developed for protection and restoration measures that will protect critical areas – notably in the immediate vicinity of Fortune St. and downstream of Fowler St. Erosion sites upstream of the proposed SWM outfall could be left to naturally erode or could be improved – in large part, by excavating the drain to form a wider cross-section. Eroding sections in undeveloped reaches such as those between Fortune St. and Fowler St. are best left to re-shape naturally without the introduction of restoration works.

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APPENDIX E

Existing Hydrology / Hydraulics

From:	Bryan Willcott <bwillcott@jfsa.com>
Sent:	March-26-10 9:58 AM
To:	spichette@dsel.ca; Adam Fobert; Conway, Darlene
Cc:	jfsabourin@jfsa.com
Subject:	P709(02) Richmond 2,5, 10, 25 Year WSEL Results
Attachments:	Spring Storm WSEL Table.pdf; Summer Storm WSEL Table.pdf; Village of Richmond (2, 5, 10, 25, 100 Yr WSEL).pdf

Good day,

Please find the attached plan showing water surface elevations for the 2, 5, 10, 25 and 100 year Spring and Summer events. WSEL tables are also attached.

The calculated flows shown on the attached tables were prepared based upon the same methodology used for the 2009 Richmond Floodplain Mapping study, taking into consideration the timing of peak flows of the Van Gaal drain and Jock River. The calculcated WSELs were determined using the 2009 Richmond Floodplain HEC-RAS spring and summer models, which do not include features including the berm that has been constructed upstream of Perth Street or any modification to Fortune Street culvert.

Best Regards,

Bryan P. Willcott, B.Eng.
Water Resources EIT

JFSA Water Resources and Environmental Consultants

J.F. Sabourin and Associates Inc.
52 Springbrook Drive, Ottawa, ON K2S 1B9
tel.: 613.836.3884 ext. 223, fax: 613.836.0332, **www.jfsa.com**

Maximum Spring Scenario Results					
Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 3	3165	2.59	96.65
5 Year	Van Gaal Drain	Reach 3	3165	3.31	96.70
10 Year	Van Gaal Drain	Reach 3	3165	3.79	96.72
25 year	Van Gaal Drain	Reach 3	3165	4.37	96.74
100 year	Van Gaal Drain	Reach 3	3165	5.24	96.75
2 Year	Van Gaal Drain	Reach 3	3149	2.59	96.60
5 Year	Van Gaal Drain	Reach 3	3149	3.31	96.64
10 Year	Van Gaal Drain	Reach 3	3149	3.79	96.66
25 year	Van Gaal Drain	Reach 3	3149	4.37	96.68
100 year	Van Gaal Drain	Reach 3	3149	5.24	96.72
2 Year	Van Gaal Drain	Reach 3	3086	2.59	96.50
5 Year	Van Gaal Drain	Reach 3	3086	3.31	96.55
10 Year	Van Gaal Drain	Reach 3	3086	3.79	96.58
25 year	Van Gaal Drain	Reach 3	3086	4.37	96.60
100 year	Van Gaal Drain	Reach 3	3086	5.24	96.64
2 Year	Van Gaal Drain	Reach 3	3016	2.59	96.46
5 Year	Van Gaal Drain	Reach 3	3016	3.31	96.52
10 Year	Van Gaal Drain	Reach 3	3016	3.79	96.54
25 year	Van Gaal Drain	Reach 3	3016	4.37	96.56
100 year	Van Gaal Drain	Reach 3	3016	5.24	96.61
2 Year	Van Gaal Drain	Reach 3	2980	2.59	96.42
5 Year	Van Gaal Drain	Reach 3	2980	3.31	96.49
10 Year	Van Gaal Drain	Reach 3	2980	3.79	96.51
25 year	Van Gaal Drain	Reach 3	2980	4.37	96.54
100 year	Van Gaal Drain	Reach 3	2980	5.24	96.57
2 Year	Van Gaal Drain	Reach 3	2851	2.59	96.21
5 Year	Van Gaal Drain	Reach 3	2851	3.31	96.30
10 Year	Van Gaal Drain	Reach 3	2851	3.79	96.34
25 year	Van Gaal Drain	Reach 3	2851	4.37	96.38
100 year	Van Gaal Drain	Reach 3	2851	5.24	96.42
2 Year	Van Gaal Drain	Reach 3	2808	2.59	96.18
5 Year	Van Gaal Drain	Reach 3	2808	3.31	96.27
10 Year	Van Gaal Drain	Reach 3	2808	3.79	96.31
25 year	Van Gaal Drain	Reach 3	2808	4.37	96.35
100 year	Van Gaal Drain	Reach 3	2808	5.24	96.39
2 Year	Van Gaal Drain	Reach 3	2658	2.59	96.11
5 Year	Van Gaal Drain	Reach 3	2658	3.31	96.19
10 Year	Van Gaal Drain	Reach 3	2658	3.79	96.22
25 year	Van Gaal Drain	Reach 3	2658	4.37	96.25
100 year	Van Gaal Drain	Reach 3	2658	5.24	96.29

Maximum Spring Scenario Results

Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	2554	4.13	96.10
5 Year	Van Gaal Drain	Reach 2	2554	5.24	96.18
10 Year	Van Gaal Drain	Reach 2	2554	6.01	96.21
25 year	Van Gaal Drain	Reach 2	2554	6.94	96.24
100 year	Van Gaal Drain	Reach 2	2554	8.32	96.28
2 Year	Van Gaal Drain	Reach 2	2478	4.13	96.03
5 Year	Van Gaal Drain	Reach 2	2478	5.24	96.09
10 Year	Van Gaal Drain	Reach 2	2478	6.01	96.11
25 year	Van Gaal Drain	Reach 2	2478	6.94	96.13
100 year	Van Gaal Drain	Reach 2	2478	8.32	96.16
2 Year	Van Gaal Drain	Reach 2	2157	4.13	95.19
5 Year	Van Gaal Drain	Reach 2	2157	5.24	95.32
10 Year	Van Gaal Drain	Reach 2	2157	6.01	95.38
25 year	Van Gaal Drain	Reach 2	2157	6.94	95.43
100 year	Van Gaal Drain	Reach 2	2157	8.32	95.48
2 Year	Van Gaal Drain	Reach 2	2076	5.00	95.03
5 Year	Van Gaal Drain	Reach 2	2076	6.32	95.13
10 Year	Van Gaal Drain	Reach 2	2076	7.24	95.17
25 year	Van Gaal Drain	Reach 2	2076	8.38	95.21
100 year	Van Gaal Drain	Reach 2	2076	10.81	95.27
2 Year	Van Gaal Drain	Reach 2	1974	5.00	94.89
5 Year	Van Gaal Drain	Reach 2	1974	6.32	94.97
10 Year	Van Gaal Drain	Reach 2	1974	7.24	95.02
25 year	Van Gaal Drain	Reach 2	1974	8.38	95.06
100 year	Van Gaal Drain	Reach 2	1974	10.81	95.11
2 Year	Van Gaal Drain	Reach 2	1922	5.00	94.82
5 Year	Van Gaal Drain	Reach 2	1922	6.32	94.89
10 Year	Van Gaal Drain	Reach 2	1922	7.24	94.92
25 year	Van Gaal Drain	Reach 2	1922	8.38	94.95
100 year	Van Gaal Drain	Reach 2	1922	10.81	94.99
2 Year	Van Gaal Drain	Reach 2	1833	5.00	94.71
5 Year	Van Gaal Drain	Reach 2	1833	6.32	94.76
10 Year	Van Gaal Drain	Reach 2	1833	7.24	94.78
25 year	Van Gaal Drain	Reach 2	1833	8.38	94.80
100 year	Van Gaal Drain	Reach 2	1833	10.81	94.85
2 Year	Van Gaal Drain	Reach 2	1796	5.00	94.68
5 Year	Van Gaal Drain	Reach 2	1796	6.32	94.73
10 Year	Van Gaal Drain	Reach 2	1796	7.24	94.75
25 year	Van Gaal Drain	Reach 2	1796	8.38	94.77
100 year	Van Gaal Drain	Reach 2	1796	10.81	94.81

Maximum Spring Scenario Results					
Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	1735	5.00	94.64
5 Year	Van Gaal Drain	Reach 2	1735	6.32	94.68
10 Year	Van Gaal Drain	Reach 2	1735	7.24	94.69
25 year	Van Gaal Drain	Reach 2	1735	8.38	94.70
100 year	Van Gaal Drain	Reach 2	1735	10.81	94.72
2 Year	Van Gaal Drain	Reach 2	1728	5.00	94.64
5 Year	Van Gaal Drain	Reach 2	1728	6.32	94.67
10 Year	Van Gaal Drain	Reach 2	1728	7.24	94.68
25 year	Van Gaal Drain	Reach 2	1728	8.38	94.68
100 year	Van Gaal Drain	Reach 2	1728	10.81	94.69
2 Year	Van Gaal Drain	Reach 2	1717	5.00	94.42
5 Year	Van Gaal Drain	Reach 2	1717	6.32	94.55
10 Year	Van Gaal Drain	Reach 2	1717	7.24	94.59
25 year	Van Gaal Drain	Reach 2	1717	8.38	94.63
100 year	Van Gaal Drain	Reach 2	1717	10.81	94.69
2 Year	Van Gaal Drain	Reach 2	1615	5.00	94.29
5 Year	Van Gaal Drain	Reach 2	1615	6.32	94.40
10 Year	Van Gaal Drain	Reach 2	1615	7.24	94.46
25 year	Van Gaal Drain	Reach 2	1615	8.38	94.52
100 year	Van Gaal Drain	Reach 2	1615	10.81	94.61
2 Year	Van Gaal Drain	Reach 2	1555	5.00	94.22
5 Year	Van Gaal Drain	Reach 2	1555	6.32	94.33
10 Year	Van Gaal Drain	Reach 2	1555	7.24	94.39
25 year	Van Gaal Drain	Reach 2	1555	8.38	94.45
100 year	Van Gaal Drain	Reach 2	1555	10.81	94.55
2 Year	Van Gaal Drain	Reach 2	1488	5.00	94.16
5 Year	Van Gaal Drain	Reach 2	1488	6.32	94.26
10 Year	Van Gaal Drain	Reach 2	1488	7.24	94.31
25 year	Van Gaal Drain	Reach 2	1488	8.38	94.36
100 year	Van Gaal Drain	Reach 2	1488	10.81	94.45
2 Year	Van Gaal Drain	Reach 2	1416	5.00	94.02
5 Year	Van Gaal Drain	Reach 2	1416	6.32	94.10
10 Year	Van Gaal Drain	Reach 2	1416	7.24	94.17
25 year	Van Gaal Drain	Reach 2	1416	8.38	94.25
100 year	Van Gaal Drain	Reach 2	1416	10.81	94.41
2 Year	Van Gaal Drain	Reach 2	1400	5.00	94.02
5 Year	Van Gaal Drain	Reach 2	1400	6.32	94.11
10 Year	Van Gaal Drain	Reach 2	1400	7.24	94.16
25 year	Van Gaal Drain	Reach 2	1400	8.38	94.23
100 year	Van Gaal Drain	Reach 2	1400	10.81	94.36

Maximum Spring Scenario Results					
Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	1364	5.00	93.99
5 Year	Van Gaal Drain	Reach 2	1364	6.32	94.06
10 Year	Van Gaal Drain	Reach 2	1364	7.24	94.11
25 year	Van Gaal Drain	Reach 2	1364	8.38	94.18
100 year	Van Gaal Drain	Reach 2	1364	10.81	94.31
2 Year	Van Gaal Drain	Reach 2	1340	5.79	93.96
5 Year	Van Gaal Drain	Reach 2	1340	7.32	94.01
10 Year	Van Gaal Drain	Reach 2	1340	8.34	94.05
25 year	Van Gaal Drain	Reach 2	1340	9.65	94.10
100 year	Van Gaal Drain	Reach 2	1340	11.62	94.21
2 Year	Van Gaal Drain	Reach 2	1312	6.08	93.94
5 Year	Van Gaal Drain	Reach 2	1312	7.69	93.98
10 Year	Van Gaal Drain	Reach 2	1312	8.76	94.01
25 year	Van Gaal Drain	Reach 2	1312	10.15	94.04
100 year	Van Gaal Drain	Reach 2	1312	3.44	94.13
2 Year	Van Gaal Drain	Reach 2	1302	6.08	93.91
5 Year	Van Gaal Drain	Reach 2	1302	7.69	93.97
10 Year	Van Gaal Drain	Reach 2	1302	8.76	94.00
25 year	Van Gaal Drain	Reach 2	1302	10.15	94.04
100 year	Van Gaal Drain	Reach 2	1302	12.20	94.15
2 Year	Van Gaal Drain	Reach 2	1268	6.08	93.87
5 Year	Van Gaal Drain	Reach 2	1268	7.69	93.93
10 Year	Van Gaal Drain	Reach 2	1268	8.76	93.96
25 year	Van Gaal Drain	Reach 2	1268	10.15	94.00
100 year	Van Gaal Drain	Reach 2	1268	12.20	94.14
2 Year	Van Gaal Drain	Reach 2	1212	6.08	93.78
5 Year	Van Gaal Drain	Reach 2	1212	7.69	93.85
10 Year	Van Gaal Drain	Reach 2	1212	8.76	93.89
25 year	Van Gaal Drain	Reach 2	1212	10.15	93.94
100 year	Van Gaal Drain	Reach 2	1212	3.44	94.12
2 Year	Van Gaal Drain	Reach 2	1169	6.08	93.70
5 Year	Van Gaal Drain	Reach 2	1169	7.69	93.76
10 Year	Van Gaal Drain	Reach 2	1169	8.76	93.80
25 year	Van Gaal Drain	Reach 2	1169	10.15	93.87
100 year	Van Gaal Drain	Reach 2	1169	3.44	94.12
2 Year	Van Gaal Drain	Reach 2	1091	6.08	93.57
5 Year	Van Gaal Drain	Reach 2	1091	7.69	93.64
10 Year	Van Gaal Drain	Reach 2	1091	8.76	93.69
25 year	Van Gaal Drain	Reach 2	1091	10.15	93.78
100 year	Van Gaal Drain	Reach 2	1091	3.44	94.12

Maximum Spring Scenario Results					
Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	1002	6.08	93.38
5 Year	Van Gaal Drain	Reach 2	1002	7.69	93.49
10 Year	Van Gaal Drain	Reach 2	1002	8.76	93.59
25 year	Van Gaal Drain	Reach 2	1002	10.15	93.71
100 year	Van Gaal Drain	Reach 2	1002	3.44	94.12
2 Year	Van Gaal Drain	Reach 2	961	6.08	93.28
5 Year	Van Gaal Drain	Reach 2	961	7.69	93.41
10 Year	Van Gaal Drain	Reach 2	961	8.76	93.52
25 year	Van Gaal Drain	Reach 2	961	10.15	93.69
100 year	Van Gaal Drain	Reach 2	961	3.44	94.11
2 Year	Van Gaal Drain	Reach 2	910	6.08	93.22
5 Year	Van Gaal Drain	Reach 2	910	7.69	93.38
10 Year	Van Gaal Drain	Reach 2	910	8.76	93.49
25 year	Van Gaal Drain	Reach 2	910	10.15	93.68
100 year	Van Gaal Drain	Reach 2	910	3.44	94.11
2 Year	Van Gaal Drain	Reach 2	840	6.08	93.18
5 Year	Van Gaal Drain	Reach 2	840	7.69	93.35
10 Year	Van Gaal Drain	Reach 2	840	8.76	93.48
25 year	Van Gaal Drain	Reach 2	840	10.15	93.67
100 year	Van Gaal Drain	Reach 2	840	3.44	94.11
2 Year	Van Gaal Drain	Reach 1	746	7.86	93.17
5 Year	Van Gaal Drain	Reach 1	746	9.97	93.35
10 Year	Van Gaal Drain	Reach 1	746	11.34	93.48
25 year	Van Gaal Drain	Reach 1	746	13.11	93.67
100 year	Van Gaal Drain	Reach 1	746	4.06	94.12
2 Year	Van Gaal Drain	Reach 1	705	7.86	93.14
5 Year	Van Gaal Drain	Reach 1	705	9.97	93.34
10 Year	Van Gaal Drain	Reach 1	705	11.34	93.47
25 year	Van Gaal Drain	Reach 1	705	13.11	93.66
100 year	Van Gaal Drain	Reach 1	705	4.06	94.11
2 Year	Van Gaal Drain	Reach 1	668	7.86	93.05
5 Year	Van Gaal Drain	Reach 1	668	9.97	93.28
10 Year	Van Gaal Drain	Reach 1	668	11.34	93.42
25 year	Van Gaal Drain	Reach 1	668	13.11	93.63
100 year	Van Gaal Drain	Reach 1	668	4.06	94.11
2 Year	Van Gaal Drain	Reach 1	666	7.86	92.64
5 Year	Van Gaal Drain	Reach 1	666	9.97	92.83
10 Year	Van Gaal Drain	Reach 1	666	11.34	93.01
25 year	Van Gaal Drain	Reach 1	666	3.31	93.43
100 year	Van Gaal Drain	Reach 1	666	4.06	94.10

Maximum Spring Scenario Results

Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Moore Drain	Reach 1	298	1.07	93.79
5 Year	Moore Drain	Reach 1	298	1.36	93.81
10 Year	Moore Drain	Reach 1	298	1.55	93.82
25 year	Moore Drain	Reach 1	298	1.78	93.83
100 year	Moore Drain	Reach 1	298	0.32	94.11
2 Year	Moore Drain	Reach 1	130	1.07	93.18
5 Year	Moore Drain	Reach 1	130	1.36	93.36
10 Year	Moore Drain	Reach 1	130	1.55	93.48
25 year	Moore Drain	Reach 1	130	1.78	93.67
100 year	Moore Drain	Reach 1	130	0.32	94.11
2 Year	Joys Road Trib	Reach 1	705	1.57	97.24
5 Year	Joys Road Trib	Reach 1	705	2.00	97.39
10 Year	Joys Road Trib	Reach 1	705	2.29	97.49
25 year	Joys Road Trib	Reach 1	705	2.64	97.60
100 year	Joys Road Trib	Reach 1	705	3.17	97.79
2 Year	Joys Road Trib	Reach 1	664	1.57	97.26
5 Year	Joys Road Trib	Reach 1	664	2.00	97.40
10 Year	Joys Road Trib	Reach 1	664	2.29	97.49
25 year	Joys Road Trib	Reach 1	664	2.64	97.61
100 year	Joys Road Trib	Reach 1	664	3.17	97.79
2 Year	Joys Road Trib	Reach 1	635	1.57	97.19
5 Year	Joys Road Trib	Reach 1	635	2.00	97.31
10 Year	Joys Road Trib	Reach 1	635	2.29	97.40
25 year	Joys Road Trib	Reach 1	635	2.64	97.50
100 year	Joys Road Trib	Reach 1	635	3.17	97.67
2 Year	Joys Road Trib	Reach 1	622	1.57	97.08
5 Year	Joys Road Trib	Reach 1	622	2.00	97.14
10 Year	Joys Road Trib	Reach 1	622	2.29	97.18
25 year	Joys Road Trib	Reach 1	622	2.64	97.21
100 year	Joys Road Trib	Reach 1	622	3.17	97.26
2 Year	Joys Road Trib	Reach 1	602	1.57	97.07
5 Year	Joys Road Trib	Reach 1	602	2.00	97.14
10 Year	Joys Road Trib	Reach 1	602	2.29	97.17
25 year	Joys Road Trib	Reach 1	602	2.64	97.22
100 year	Joys Road Trib	Reach 1	602	3.17	97.27
2 Year	Joys Road Trib	Reach 1	322	1.57	96.54
5 Year	Joys Road Trib	Reach 1	322	2.00	96.58
10 Year	Joys Road Trib	Reach 1	322	2.29	96.61
25 year	Joys Road Trib	Reach 1	322	2.64	96.65
100 year	Joys Road Trib	Reach 1	322	3.17	96.71

Maximum Spring Scenario Results					
Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Joys Road Trib	Reach 1	275	1.57	96.31
5 Year	Joys Road Trib	Reach 1	275	2.00	96.39
10 Year	Joys Road Trib	Reach 1	275	2.29	96.44
25 year	Joys Road Trib	Reach 1	275	2.64	96.49
100 year	Joys Road Trib	Reach 1	275	3.17	96.56
2 Year	Joys Road Trib	Reach 1	30	1.57	96.12
5 Year	Joys Road Trib	Reach 1	30	2.00	96.19
10 Year	Joys Road Trib	Reach 1	30	2.29	96.22
25 year	Joys Road Trib	Reach 1	30	2.64	96.26
100 year	Joys Road Trib	Reach 1	30	3.17	96.29

Maximum Summer Scenario Results

Return Period	River	Reach	River Station	Q Total (m^3/s)	W.S. Elev (m)
2 Year	Van Gaal Drain	Reach 3	3165	1.33	96.50
5 Year	Van Gaal Drain	Reach 3	3165	2.15	96.61
10 Year	Van Gaal Drain	Reach 3	3165	2.74	96.67
25 year	Van Gaal Drain	Reach 3	3165	3.51	96.72
100 year	Van Gaal Drain	Reach 3	3165	4.81	96.75
2 Year	Van Gaal Drain	Reach 3	3149	1.33	96.46
5 Year	Van Gaal Drain	Reach 3	3149	2.15	96.57
10 Year	Van Gaal Drain	Reach 3	3149	2.74	96.62
25 year	Van Gaal Drain	Reach 3	3149	3.51	96.66
100 year	Van Gaal Drain	Reach 3	3149	4.81	96.72
2 Year	Van Gaal Drain	Reach 3	3086	1.33	96.35
5 Year	Van Gaal Drain	Reach 3	3086	2.15	96.47
10 Year	Van Gaal Drain	Reach 3	3086	2.74	96.53
25 year	Van Gaal Drain	Reach 3	3086	3.51	96.58
100 year	Van Gaal Drain	Reach 3	3086	4.81	96.65
2 Year	Van Gaal Drain	Reach 3	3016	1.33	96.26
5 Year	Van Gaal Drain	Reach 3	3016	2.15	96.41
10 Year	Van Gaal Drain	Reach 3	3016	2.74	96.48
25 year	Van Gaal Drain	Reach 3	3016	3.51	96.54
100 year	Van Gaal Drain	Reach 3	3016	4.81	96.61
2 Year	Van Gaal Drain	Reach 3	2980	1.33	96.21
5 Year	Van Gaal Drain	Reach 3	2980	2.15	96.36
10 Year	Van Gaal Drain	Reach 3	2980	2.74	96.44
25 year	Van Gaal Drain	Reach 3	2980	3.51	96.51
100 year	Van Gaal Drain	Reach 3	2980	4.81	96.57
2 Year	Van Gaal Drain	Reach 3	2851	1.33	95.90
5 Year	Van Gaal Drain	Reach 3	2851	2.15	96.10
10 Year	Van Gaal Drain	Reach 3	2851	2.74	96.21
25 year	Van Gaal Drain	Reach 3	2851	3.51	96.31
100 year	Van Gaal Drain	Reach 3	2851	4.81	96.41
2 Year	Van Gaal Drain	Reach 3	2808	1.33	95.86
5 Year	Van Gaal Drain	Reach 3	2808	2.15	96.07
10 Year	Van Gaal Drain	Reach 3	2808	2.74	96.18
25 year	Van Gaal Drain	Reach 3	2808	3.51	96.28
100 year	Van Gaal Drain	Reach 3	2808	4.81	96.38
2 Year	Van Gaal Drain	Reach 3	2658	1.33	95.78
5 Year	Van Gaal Drain	Reach 3	2658	2.15	95.99
10 Year	Van Gaal Drain	Reach 3	2658	2.74	96.10
25 year	Van Gaal Drain	Reach 3	2658	3.51	96.19
100 year	Van Gaal Drain	Reach 3	2658	4.81	96.28

Maximum Summer Scenario Results

Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	2554	1.95	95.77
5 Year	Van Gaal Drain	Reach 2	2554	3.20	95.98
10 Year	Van Gaal Drain	Reach 2	2554	4.11	96.09
25 year	Van Gaal Drain	Reach 2	2554	5.28	96.18
100 year	Van Gaal Drain	Reach 2	2554	7.27	96.27
2 Year	Van Gaal Drain	Reach 2	2478	1.95	95.72
5 Year	Van Gaal Drain	Reach 2	2478	3.20	95.92
10 Year	Van Gaal Drain	Reach 2	2478	4.11	96.02
25 year	Van Gaal Drain	Reach 2	2478	5.28	96.09
100 year	Van Gaal Drain	Reach 2	2478	7.27	96.16
2 Year	Van Gaal Drain	Reach 2	2157	1.95	94.93
5 Year	Van Gaal Drain	Reach 2	2157	3.20	95.12
10 Year	Van Gaal Drain	Reach 2	2157	4.11	95.25
25 year	Van Gaal Drain	Reach 2	2157	5.28	95.37
100 year	Van Gaal Drain	Reach 2	2157	7.27	65.47
2 Year	Van Gaal Drain	Reach 2	2076	2.80	94.74
5 Year	Van Gaal Drain	Reach 2	2076	4.41	94.97
10 Year	Van Gaal Drain	Reach 2	2076	5.53	95.08
25 year	Van Gaal Drain	Reach 2	2076	6.96	95.17
100 year	Van Gaal Drain	Reach 2	2076	9.54	95.26
2 Year	Van Gaal Drain	Reach 2	1974	2.80	94.61
5 Year	Van Gaal Drain	Reach 2	1974	4.41	94.84
10 Year	Van Gaal Drain	Reach 2	1974	5.53	94.93
25 year	Van Gaal Drain	Reach 2	1974	6.96	95.01
100 year	Van Gaal Drain	Reach 2	1974	9.54	95.10
2 Year	Van Gaal Drain	Reach 2	1922	2.80	94.55
5 Year	Van Gaal Drain	Reach 2	1922	4.41	94.79
10 Year	Van Gaal Drain	Reach 2	1922	5.53	94.86
25 year	Van Gaal Drain	Reach 2	1922	6.96	94.92
100 year	Van Gaal Drain	Reach 2	1922	9.54	94.99
2 Year	Van Gaal Drain	Reach 2	1833	2.80	94.47
5 Year	Van Gaal Drain	Reach 2	1833	4.41	94.69
10 Year	Van Gaal Drain	Reach 2	1833	5.53	94.74
25 year	Van Gaal Drain	Reach 2	1833	6.96	94.78
100 year	Van Gaal Drain	Reach 2	1833	9.54	94.85
2 Year	Van Gaal Drain	Reach 2	1796	2.80	94.44
5 Year	Van Gaal Drain	Reach 2	1796	4.41	94.66
10 Year	Van Gaal Drain	Reach 2	1796	5.53	94.71
25 year	Van Gaal Drain	Reach 2	1796	6.96	94.74
100 year	Van Gaal Drain	Reach 2	1796	9.54	94.80

Maximum Summer Scenario Results

Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	1735	2.80	94.41
5 Year	Van Gaal Drain	Reach 2	1735	4.41	94.63
10 Year	Van Gaal Drain	Reach 2	1735	5.53	94.67
25 year	Van Gaal Drain	Reach 2	1735	6.96	94.68
100 year	Van Gaal Drain	Reach 2	1735	9.54	94.71
2 Year	Van Gaal Drain	Reach 2	1728	2.80	94.41
5 Year	Van Gaal Drain	Reach 2	1728	4.41	94.62
10 Year	Van Gaal Drain	Reach 2	1728	5.53	94.66
25 year	Van Gaal Drain	Reach 2	1728	6.96	94.67
100 year	Van Gaal Drain	Reach 2	1728	9.54	94.69
2 Year	Van Gaal Drain	Reach 2	1717	2.80	94.15
5 Year	Van Gaal Drain	Reach 2	1717	4.41	94.37
10 Year	Van Gaal Drain	Reach 2	1717	5.53	94.49
25 year	Van Gaal Drain	Reach 2	1717	6.96	94.59
100 year	Van Gaal Drain	Reach 2	1717	9.54	94.69
2 Year	Van Gaal Drain	Reach 2	1615	2.80	94.04
5 Year	Van Gaal Drain	Reach 2	1615	4.41	94.24
10 Year	Van Gaal Drain	Reach 2	1615	5.53	94.36
25 year	Van Gaal Drain	Reach 2	1615	6.96	94.46
100 year	Van Gaal Drain	Reach 2	1615	9.54	94.60
2 Year	Van Gaal Drain	Reach 2	1555	2.80	93.99
5 Year	Van Gaal Drain	Reach 2	1555	4.41	94.19
10 Year	Van Gaal Drain	Reach 2	1555	5.53	94.29
25 year	Van Gaal Drain	Reach 2	1555	6.96	94.40
100 year	Van Gaal Drain	Reach 2	1555	9.54	94.53
2 Year	Van Gaal Drain	Reach 2	1488	2.80	93.96
5 Year	Van Gaal Drain	Reach 2	1488	4.41	94.13
10 Year	Van Gaal Drain	Reach 2	1488	5.53	94.23
25 year	Van Gaal Drain	Reach 2	1488	6.96	94.33
100 year	Van Gaal Drain	Reach 2	1488	9.54	94.45
2 Year	Van Gaal Drain	Reach 2	1416	2.80	93.89
5 Year	Van Gaal Drain	Reach 2	1416	4.41	94.03
10 Year	Van Gaal Drain	Reach 2	1416	5.53	94.10
25 year	Van Gaal Drain	Reach 2	1416	6.96	94.20
100 year	Van Gaal Drain	Reach 2	1416	9.54	94.39
2 Year	Van Gaal Drain	Reach 2	1400	2.80	93.89
5 Year	Van Gaal Drain	Reach 2	1400	4.41	94.03
10 Year	Van Gaal Drain	Reach 2	1400	5.53	94.11
25 year	Van Gaal Drain	Reach 2	1400	6.96	94.20
100 year	Van Gaal Drain	Reach 2	1400	9.54	94.36

Maximum Summer Scenario Results

Return Period	River	Reach	River Station	Q Total (m^3/s)	W.S. Elev (m)
2 Year	Van Gaal Drain	Reach 2	1364	2.80	93.87
5 Year	Van Gaal Drain	Reach 2	1364	4.41	94.00
10 Year	Van Gaal Drain	Reach 2	1364	5.53	94.07
25 year	Van Gaal Drain	Reach 2	1364	6.96	94.16
100 year	Van Gaal Drain	Reach 2	1364	9.54	94.31
2 Year	Van Gaal Drain	Reach 2	1340	3.64	93.85
5 Year	Van Gaal Drain	Reach 2	1340	5.57	93.97
10 Year	Van Gaal Drain	Reach 2	1340	6.92	94.03
25 year	Van Gaal Drain	Reach 2	1340	8.58	94.09
100 year	Van Gaal Drain	Reach 2	1340	11.43	94.21
2 Year	Van Gaal Drain	Reach 2	1312	3.87	93.84
5 Year	Van Gaal Drain	Reach 2	1312	5.93	93.95
10 Year	Van Gaal Drain	Reach 2	1312	7.38	94.00
25 year	Van Gaal Drain	Reach 2	1312	9.17	94.05
100 year	Van Gaal Drain	Reach 2	1312	12.20	94.14
2 Year	Van Gaal Drain	Reach 2	1302	3.87	93.81
5 Year	Van Gaal Drain	Reach 2	1302	5.93	93.92
10 Year	Van Gaal Drain	Reach 2	1302	7.38	93.98
25 year	Van Gaal Drain	Reach 2	1302	9.17	94.04
100 year	Van Gaal Drain	Reach 2	1302	12.20	94.15
2 Year	Van Gaal Drain	Reach 2	1268	3.87	93.75
5 Year	Van Gaal Drain	Reach 2	1268	5.93	93.88
10 Year	Van Gaal Drain	Reach 2	1268	7.38	93.94
25 year	Van Gaal Drain	Reach 2	1268	9.17	94.01
100 year	Van Gaal Drain	Reach 2	1268	12.20	94.14
2 Year	Van Gaal Drain	Reach 2	1212	3.87	93.61
5 Year	Van Gaal Drain	Reach 2	1212	5.93	93.78
10 Year	Van Gaal Drain	Reach 2	1212	7.38	93.85
25 year	Van Gaal Drain	Reach 2	1212	9.17	93.93
100 year	Van Gaal Drain	Reach 2	1212	12.20	94.10
2 Year	Van Gaal Drain	Reach 2	1169	3.87	93.53
5 Year	Van Gaal Drain	Reach 2	1169	5.93	93.70
10 Year	Van Gaal Drain	Reach 2	1169	7.38	93.77
25 year	Van Gaal Drain	Reach 2	1169	9.17	93.85
100 year	Van Gaal Drain	Reach 2	1169	12.20	94.04
2 Year	Van Gaal Drain	Reach 2	1091	3.87	93.40
5 Year	Van Gaal Drain	Reach 2	1091	5.93	93.57
10 Year	Van Gaal Drain	Reach 2	1091	7.38	93.65
25 year	Van Gaal Drain	Reach 2	1091	9.17	93.75
100 year	Van Gaal Drain	Reach 2	1091	12.20	93.97

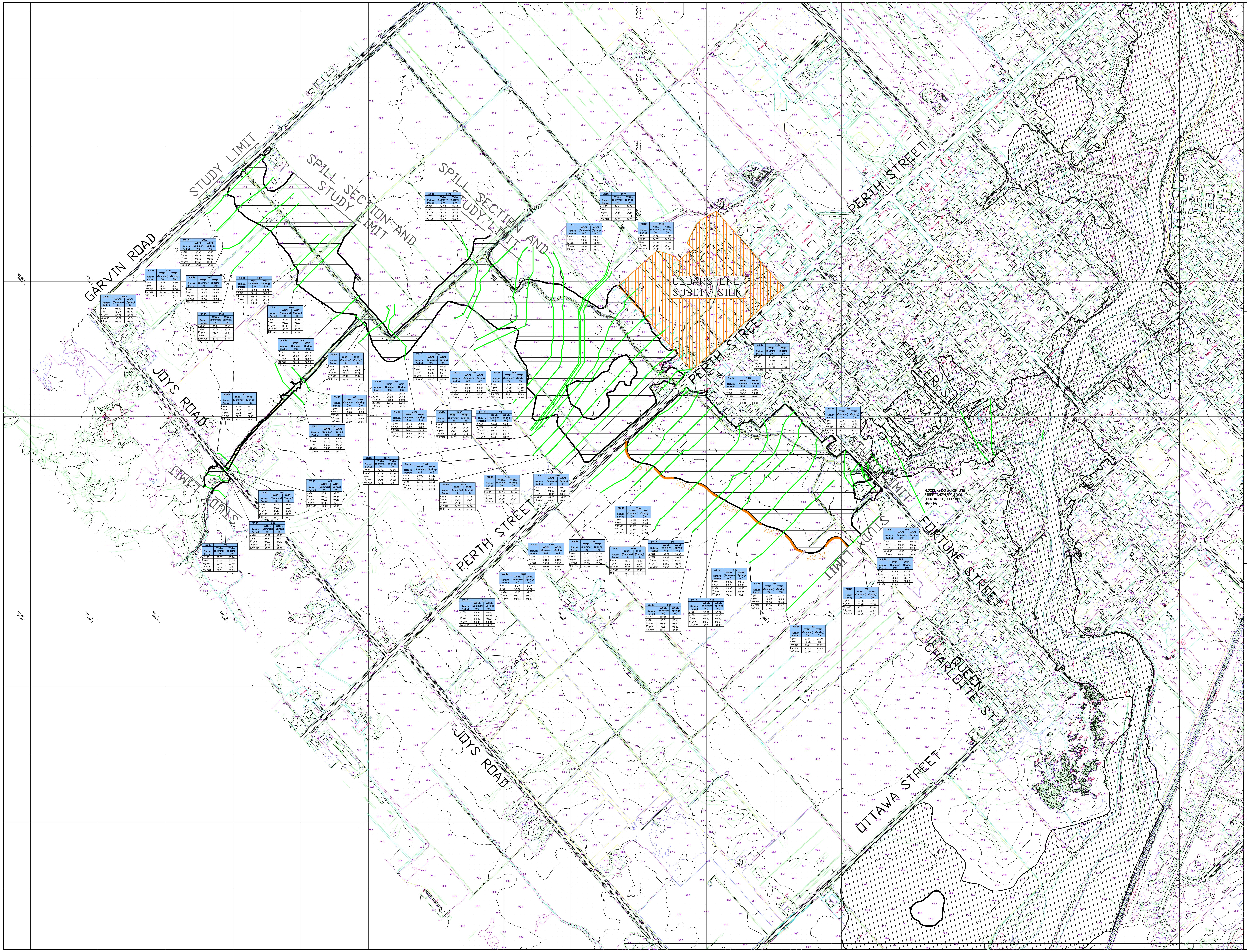
Maximum Summer Scenario Results

Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Van Gaal Drain	Reach 2	1002	3.87	93.21
5 Year	Van Gaal Drain	Reach 2	1002	5.93	93.39
10 Year	Van Gaal Drain	Reach 2	1002	7.38	93.50
25 year	Van Gaal Drain	Reach 2	1002	9.17	93.65
100 year	Van Gaal Drain	Reach 2	1002	12.20	93.92
2 Year	Van Gaal Drain	Reach 2	961	3.87	93.14
5 Year	Van Gaal Drain	Reach 2	961	5.93	93.31
10 Year	Van Gaal Drain	Reach 2	961	7.38	93.44
25 year	Van Gaal Drain	Reach 2	961	9.17	93.61
100 year	Van Gaal Drain	Reach 2	961	12.20	93.92
2 Year	Van Gaal Drain	Reach 2	910	3.87	93.07
5 Year	Van Gaal Drain	Reach 2	910	5.93	93.25
10 Year	Van Gaal Drain	Reach 2	910	7.38	93.40
25 year	Van Gaal Drain	Reach 2	910	9.17	93.59
100 year	Van Gaal Drain	Reach 2	910	12.20	93.91
2 Year	Van Gaal Drain	Reach 2	840	3.87	93.00
5 Year	Van Gaal Drain	Reach 2	840	5.93	93.22
10 Year	Van Gaal Drain	Reach 2	840	7.38	93.38
25 year	Van Gaal Drain	Reach 2	840	9.17	93.58
100 year	Van Gaal Drain	Reach 2	840	12.20	93.91
2 Year	Van Gaal Drain	Reach 1	746	5.36	92.96
5 Year	Van Gaal Drain	Reach 1	746	8.09	93.20
10 Year	Van Gaal Drain	Reach 1	746	10.02	93.37
25 year	Van Gaal Drain	Reach 1	746	12.40	93.57
100 year	Van Gaal Drain	Reach 1	746	16.38	93.90
2 Year	Van Gaal Drain	Reach 1	705	5.36	92.88
5 Year	Van Gaal Drain	Reach 1	705	8.09	93.16
10 Year	Van Gaal Drain	Reach 1	705	10.02	93.34
25 year	Van Gaal Drain	Reach 1	705	12.40	93.55
100 year	Van Gaal Drain	Reach 1	705	16.38	93.89
2 Year	Van Gaal Drain	Reach 1	668	5.36	92.72
5 Year	Van Gaal Drain	Reach 1	668	8.09	93.04
10 Year	Van Gaal Drain	Reach 1	668	10.02	93.25
25 year	Van Gaal Drain	Reach 1	668	12.40	93.48
100 year	Van Gaal Drain	Reach 1	668	16.38	93.84
2 Year	Van Gaal Drain	Reach 1	666	5.36	92.48
5 Year	Van Gaal Drain	Reach 1	666	8.09	92.66
10 Year	Van Gaal Drain	Reach 1	666	10.02	92.80
25 year	Van Gaal Drain	Reach 1	666	12.40	93.03
100 year	Van Gaal Drain	Reach 1	666	16.38	93.32

Maximum Summer Scenario Results					
Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Moore Drain	Reach 1	298	0.67	93.68
5 Year	Moore Drain	Reach 1	298	1.06	93.78
10 Year	Moore Drain	Reach 1	298	1.32	93.81
25 year	Moore Drain	Reach 1	298	1.64	93.83
100 year	Moore Drain	Reach 1	298	2.17	93.90
2 Year	Moore Drain	Reach 1	130	0.67	93.00
5 Year	Moore Drain	Reach 1	130	1.06	93.22
10 Year	Moore Drain	Reach 1	130	1.32	93.38
25 year	Moore Drain	Reach 1	130	1.64	93.58
100 year	Moore Drain	Reach 1	130	2.17	93.91
2 Year	Joys Road Trib	Reach 1	705	0.66	97.07
5 Year	Joys Road Trib	Reach 1	705	1.11	97.14
10 Year	Joys Road Trib	Reach 1	705	1.44	97.19
25 year	Joys Road Trib	Reach 1	705	1.87	97.35
100 year	Joys Road Trib	Reach 1	705	2.62	97.59
2 Year	Joys Road Trib	Reach 1	664	0.66	96.97
5 Year	Joys Road Trib	Reach 1	664	1.11	97.12
10 Year	Joys Road Trib	Reach 1	664	1.44	97.22
25 year	Joys Road Trib	Reach 1	664	1.87	97.36
100 year	Joys Road Trib	Reach 1	664	2.62	97.60
2 Year	Joys Road Trib	Reach 1	635	0.66	96.94
5 Year	Joys Road Trib	Reach 1	635	1.11	97.07
10 Year	Joys Road Trib	Reach 1	635	1.44	97.16
25 year	Joys Road Trib	Reach 1	635	1.87	97.28
100 year	Joys Road Trib	Reach 1	635	2.62	97.50
2 Year	Joys Road Trib	Reach 1	622	0.66	96.91
5 Year	Joys Road Trib	Reach 1	622	1.11	97.00
10 Year	Joys Road Trib	Reach 1	622	1.44	97.06
25 year	Joys Road Trib	Reach 1	622	1.87	97.13
100 year	Joys Road Trib	Reach 1	622	2.62	97.21
2 Year	Joys Road Trib	Reach 1	602	0.66	96.90
5 Year	Joys Road Trib	Reach 1	602	1.11	96.99
10 Year	Joys Road Trib	Reach 1	602	1.44	97.05
25 year	Joys Road Trib	Reach 1	602	1.87	97.12
100 year	Joys Road Trib	Reach 1	602	2.62	97.21
2 Year	Joys Road Trib	Reach 1	322	0.66	96.41
5 Year	Joys Road Trib	Reach 1	322	1.11	96.49
10 Year	Joys Road Trib	Reach 1	322	1.44	96.53
25 year	Joys Road Trib	Reach 1	322	1.87	96.57
100 year	Joys Road Trib	Reach 1	322	2.62	96.65

Maximum Summer Scenario Results

Return Period	*River*	*Reach*	*River Station*	*Q Total (m^3/s)*	*W.S. Elev (m)*
2 Year	Joys Road Trib	Reach 1	275	0.66	96.18
5 Year	Joys Road Trib	Reach 1	275	1.11	96.23
10 Year	Joys Road Trib	Reach 1	275	1.44	96.28
25 year	Joys Road Trib	Reach 1	275	1.87	96.37
100 year	Joys Road Trib	Reach 1	275	2.62	96.50
2 Year	Joys Road Trib	Reach 1	30	0.66	95.78
5 Year	Joys Road Trib	Reach 1	30	1.11	96.00
10 Year	Joys Road Trib	Reach 1	30	1.44	96.11
25 year	Joys Road Trib	Reach 1	30	1.87	96.20
100 year	Joys Road Trib	Reach 1	30	2.62	96.29



LEGEND :

- 100 Yr Floodlines
- Cross Section Locations
- ▨ Cedarstone Subdivision

Water Surface Elevation Tables

XS ID	xxx	
Return Period	WSEL (Summer) (m)	WSEL (Spring) (m)
2 year	XXX	XXX
5 year	XXX	XXX
10 year	XXX	XXX
25 year	XXX	XXX
100 year	XXX	XXX

SCALE : 1: 5000

0 100 200 300 400 500 m

N

J.F. Sabourin & Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS
OTTAWA (613) 836–3884
GATINEAU (819) 243–6858

CLIENT :

MATTAMY homes

PROJECT :

Van Gaal and Arbuckle Drain 2, 5, 10, 25, and 100 Yr WSEL

No.	BY	DATE	DESCRIPTION	BY
1	CB	03/25/2010	ISSUED FOR REVIEW	CB

WSEL for the 2, 5, 10, 25, and 100 Year Design Storms

DESIGNED:
DRAWN: CB
VERIFIED: BW
APPROVED: JFS

DRAWING REF.

DATE	PROJECT No.
Mar/10	709(02)

APPENDIX F

Hydrogeology

Golder Associates

TECHNICAL MEMORANDUM

DATE August 11, 2011 **PROJECT No.** 08-1122-0078 (9600)

TO Ms. Susan Murphy
Mattamy Homes

FROM Dale Holtze
Brian Byerley

EMAIL Dale_Holtze@Golder.com
Brian_Byerley@Golder.com

GROUNDWATER MONITORING PROGRAM
PROPOSED MATTAMY HOMES DEVELOPMENT
RICHMOND (OTTAWA), ONTARIO

This memo presents the results of a one year groundwater monitoring program conducted by Golder Associates Ltd. (Golder) at the site of the proposed Mattamy Homes (Mattamy) development in the Village of Richmond (in the City of Ottawa), Ontario (see Key Plan: Figure 1). The results of previous hydrogeological investigation conducted at the site by Golder are discussed in the Technical Memorandum entitled: Hydrogeological Investigation Proposed Mattamy Homes Development Richmond (Ottawa), Ontario, dated July 16, 2010, which includes a summary of the drilling of eight boreholes equipped with monitoring and/or multi-level monitoring wells, description of subsurface conditions and hydraulic conductivity testing performed in all groundwater monitors.

Monitoring Program

The groundwater monitoring program included monthly groundwater level measurements from April 2010 to April 2011 at monitoring wells MW10-1, MW10-2, MW10-3, MW10-4, MW10-5, MW10-6, MW10-7 and MW10-8, located as shown on Figure 2. Groundwater monitor MW10-5 was observed to be damaged in June 2010 (MW10-5A) and July 2010 (MW10-5B); therefore groundwater levels are not available for the remainder of the monitoring program at this location.

Groundwater levels at monitor MW10-6A (bedrock) was measured continuously (two hour interval) using a transducer from March 19 to April 21, 2011 in order to assess the hydrogeological response of the shallow overburden and bedrock aquifers related to the spring melt and precipitation recharge events. Average daily climatic data was obtained from Environment Canada database at the Ottawa-Macdonald Cartier International Airport. Local rain gauge data for the period between August 1 to October 3, 2010, was also provided by the City of Ottawa. Monitor MW10-6B (overburden) was frozen during that time period and thus could not provide useful data.

Results

Monthly groundwater levels and description of the subsurface conditions at each monitoring well are presented in Table 1. Groundwater levels are subject to seasonal fluctuations ranging from 0.5 metres (MW10-4A) to 1.8 metres (MW10-6B) in overburden monitoring wells (see Figures 2 and 3) and 0.8 metres (MW10-3A) to

1.3 metres (MW10-8) in bedrock monitoring wells (see Figure 4). The groundwater levels generally vary from less than 0.1 to 2 metres below ground surface, with the exception of bedrock monitor MW10-3A where water levels were up to 0.2 metres above ground surface, indicating artesian conditions in the upper bedrock. Continuous groundwater level measurements at monitors MW10-6A (bedrock) and MW10-6B (overburden) in Spring 2011 are presented in Figure 5.

Discussion

Groundwater levels are subject to seasonal variability: minimum groundwater levels are encountered in the summer and winter months and maximum levels occur in spring and fall.

The greatest fluctuations in groundwater levels occurred at MW10-7 and MW10-6B (overburden monitors) and at MW10-6A and MW10-8 (bedrock monitors). Shallow groundwater levels are influenced significantly by infiltration, which vary according to precipitation, snowmelt and evapotranspiration.

The horizontal groundwater flow direction within the overburden is interpreted to be controlled by local surface water drainage features (i.e. Jock River, drains and ditches).

Multi-level monitors installed within the overburden and bedrock were used to assess vertical hydraulic gradients. In general, the hydraulic gradients indicated groundwater was observed to consistent upward flow from the bedrock to the overburden at MW10-3 and downward at MW10-6.

If you have any questions regarding the content of this memo, please contact the undersigned.

Yours truly,

GOLDER ASSOCIATES LTD.

Dale Holtze

Dale Holtze, B.Sc., GIT.
Environmental Consultant

for Brian Byerley, M.Sc., P.Eng.
Senior Hydrogeologist/Associate

DH/BTB/sg
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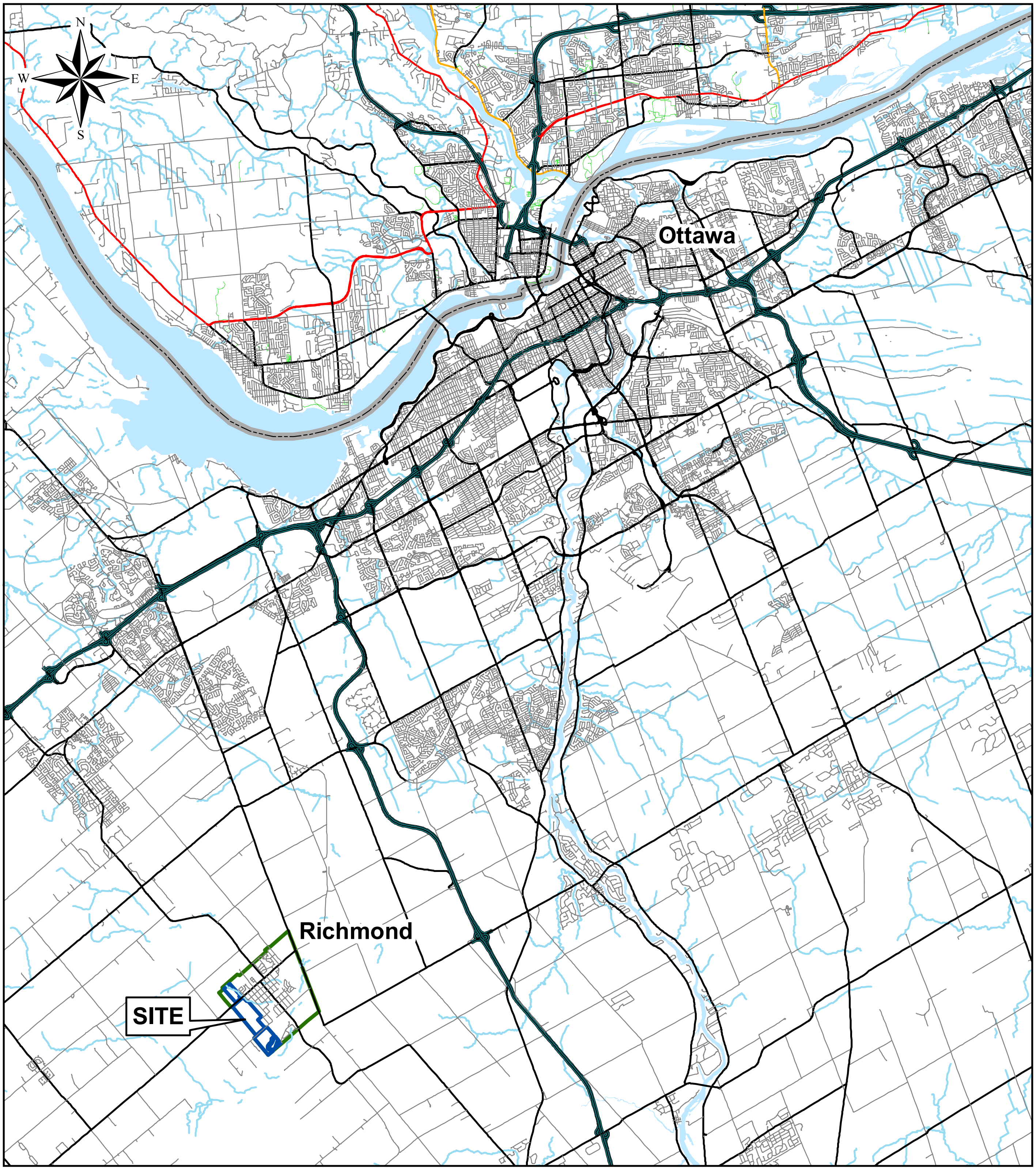
Attachments: Table 1 – Groundwater Monitoring Data
Figure 1: Key Plan
Figure 2: Spring 2011 Groundwater Levels
Figure 3: Groundwater Levels in Overburden Monitoring Wells April 2010 to April 2011
Figure 4: Groundwater Levels in Bedrock Monitoring Wells April 2010 to April 2011
Figure 5: Continuous Measurements of Groundwater Levels at MW10-6 March and April 2011

Table 1: Groundwater Monitoring Data

Well ID	Ground Surface Elevation (geodetic)	Screen depth (middle of screen) (mbgs)	Soil/rock at depth of well screen	Groundwater Level (mbgs)						
				Apr.29 & 30, 2010	May 14, 2010	Jun.16, 2010	Jul. 15, 2010	Aug. 20, 2011	Sept. 16, 2010	Oct. 1, 2010
MW10-1A	94.55	1.11	Grey silty Clay	0.73	0.78	0.80	0.81	0.78	0.81	0.61
MW10-1B	94.55	1.21	Grey brown silty Clay (weathered crust)	0.74	0.77	0.80	0.79	0.79	0.81	0.62
MW10-2	94.90	2.12	Grey brown silty find Sand	0.73	0.67	0.86	0.66	0.96	0.94	0.17
MW10-3A	94.0	4.55	Fresh grey Dolomite	0.22	0.26	0.58	0.28	0.23	0.25	-0.09[1]
MW10-3B	94.0	2.12	Grey brown silty Clay (weathered crust)/grey brown fine sandy Silt	0.82	0.81	0.87	0.79	0.87	0.85	0.44
MW10-4A	94.34	3.03	Grey brown fine sandy Silt	0.48	0.47	0.56	0.49	0.64	0.62	0.11
MW10-4B	94.34	1.21	Grey brown silty Clay (weathered crust)	0.47	0.49	0.56	0.49	0.64	0.62	0.08
MW10-5A	95.65	3.03	Glacial Till	0.82	0.79	--[2]	--[2]	--[2]	--[2]	--[2]
MW10-5B	95.65	1.21	Grey brown silty find Sand	0.89	0.88	1.33	--[2]	--[2]	--[2]	--[2]
MW10-6A	95.67	4.24	Fresh grey Dolomite	1.38	1.38	1.98	1.34	1.31	1.52	0.66
MW10-6B	95.67	1.52	Grey brown silty Sand trace Clay	1.29	1.28	1.84	1.59	1.12	1.55	0.31
MW10-7	95.36	2.42	Grey brown silty fine Sand	0.51	0.49	1.09	0.98	0.38	0.47	0.03
MW10-8	93.32	2.42	Weathered to fresh grey Dolomite	1.02	1.15	1.52	1.39	1.24	1.29	0.23

Well ID	Ground Surface Elevation (geodetic)	Screen depth (middle of screen) (mbgs)	Soil/rock at depth of well screen	Groundwater Level (mbgs)						
				Oct 18, 2010	Nov. 19, 2010	Dec. 15, 2010	Jan. 18, 2011	Feb. 14, 2011	Mar. 15, 2011	Apr. 15, 2011
MW10-1A	94.55	1.11	Grey silty Clay	0.77	0.72	0.78	1.00	1.21	0.89	0.64
MW10-1B	94.55	1.21	Grey brown silty Clay (weathered crust)	0.75	0.72	0.78	0.97	1.17	0.90	0.66
MW10-2	94.90	2.12	Grey brown silty find Sand	0.59	0.34	0.55	0.78	0.89	0.56	0.13
MW10-3A	94.0	4.55	Fresh grey Dolomite	0.08	-0.06[1]	--[3]	--[3]	--[3]	--[3]	-0.21[1]
MW10-3B	94.0	2.12	Grey brown silty Clay (weathered crust)/grey brown fine sandy Silt	0.72	0.56	0.73	0.90	0.77	0.69	0.29
MW10-4A	94.34	3.03	Grey brown fine sandy Silt	0.47	0.27	0.44	0.53	0.64	0.41	0.21
MW10-4B	94.34	1.21	Grey brown silty Clay (weathered crust)	0.46	0.25	0.44	0.56	0.65	--[3]	0.26
MW10-5A	95.65	3.03	Glacial Till	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
MW10-5B	95.65	1.21	Grey brown silty find Sand	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
MW10-6A	95.67	4.24	Fresh grey Dolomite	0.84	0.67	0.68	1.22	1.44	0.56	0.46
MW10-6B	95.67	1.52	Grey brown silty Sand trace Clay	0.62	0.26	0.50	1.16	1.42	--[3]	0.05
MW10-7	95.36	2.42	Grey brown silty fine Sand	0.06	0.04	0.03	--[3]	--[3]	--[3]	0.02
MW10-8	93.32	2.42	Weathered to fresh grey Dolomite	0.31	0.26	0.27	0.80	0.81	--[3]	0.19

Notes: [1] Artesian conditions exist. Groundwater level above ground surface.
[2] Monitoring well MW 10-5 A and B vandalized and groundwater levels not available after May 14, 2010.
[3] Groundwater in monitoring well frozen. Depth to groundwater level could not be measured.



REFERENCE

Digital base map data supplied by DMTI Spatial Inc. CANMAP, 2005
Projection: Transverse Mercator   Datum: NAD 83   Coordinate System: UTM Zone 18

5,000   0   5,000
SCALE   1:200,000

Golder Associates
Ottawa, Ontario

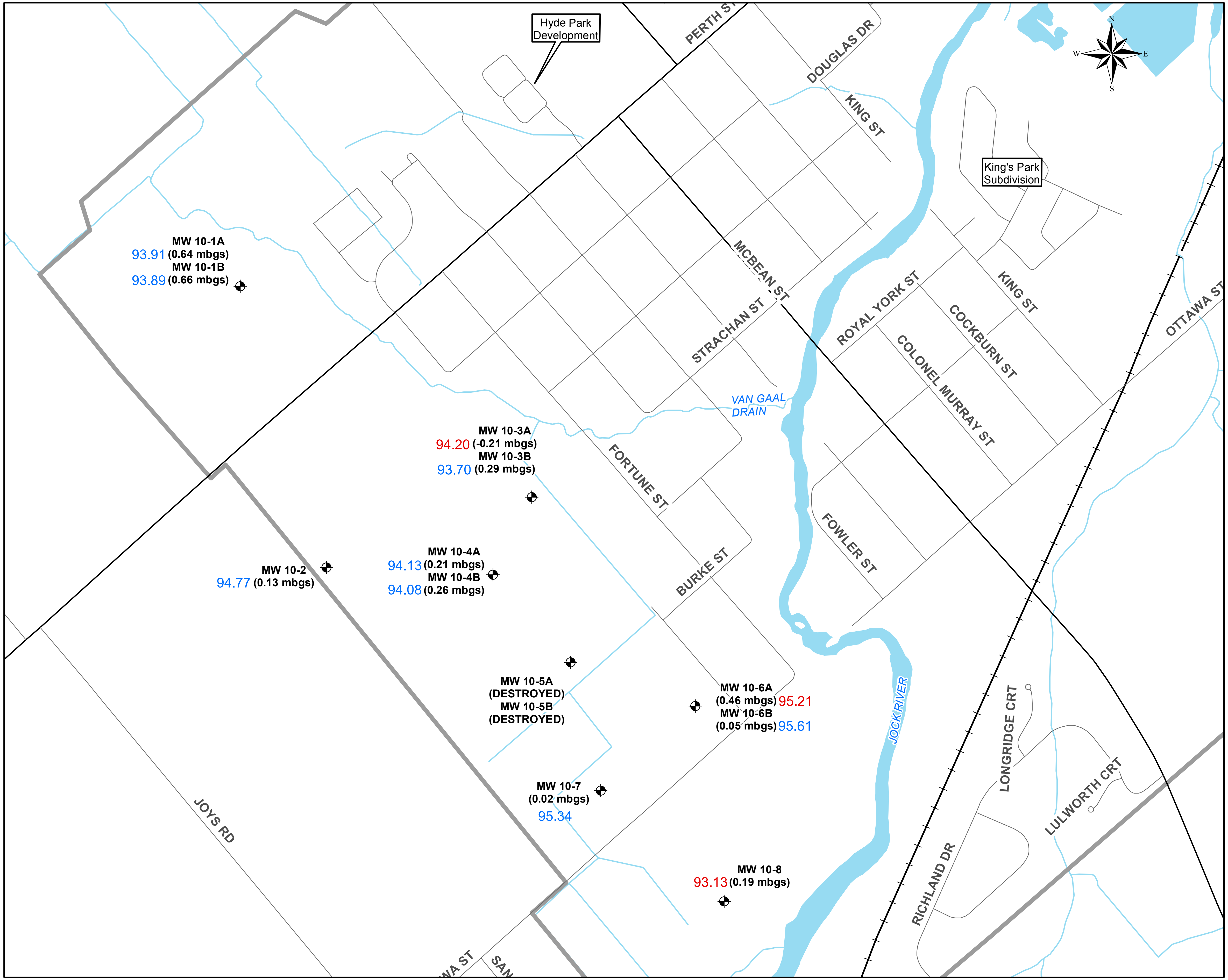
DATE	10 Aug 2011
DESIGN	JPAO
GIS	ABD
CHECK	DH
REVIEW	BTB

PROJECT No. 08-1122-0078
SCALE AS SHOWN   REV. 0

TITLE
KEY PLAN

PROJECT
MATTAMY HOMES
VILLAGE OF RICHMOND

FIGURE: 1



LEGEND

⊕ MONITORING WELL (CURRENT INVESTIGATION)
(0.64 mbgs) - SPRING 2011 GROUNDWATER LEVEL, metres below ground surface

— MAJOR ROAD

— LOCAL ROAD

┼ RAILWAY

▬ RICHMOND VILLAGE BOUNDARY

— WATERCOURSE

▆ SURFACE WATER

95.34 SPRING 2011 GROUNDWATER ELEVATION IN OVERBURDEN

95.34 SPRING 2011 GROUNDWATER ELEVATION IN BEDROCK

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. TECHNICAL MEMO NO. 08-1122-0078 PHASE 9600

REFERENCE

DIGITAL DATA PROVIDED BY ONTARIO MINISTRY OF NATURAL RESOURCES, USED UNDER LICENSE © QUEEN'S PRINTER OF ONTARIO, 2008.
WATER WELLS PROVIDED BY ONTARIO MINISTRY OF ENVIRONMENT, 2006.
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: UTM ZONE 18

250 0 250

SCALE 1:10,000 METRES

PROJECT

MATTAMY HOMES
VILLAGE OF RICHMOND

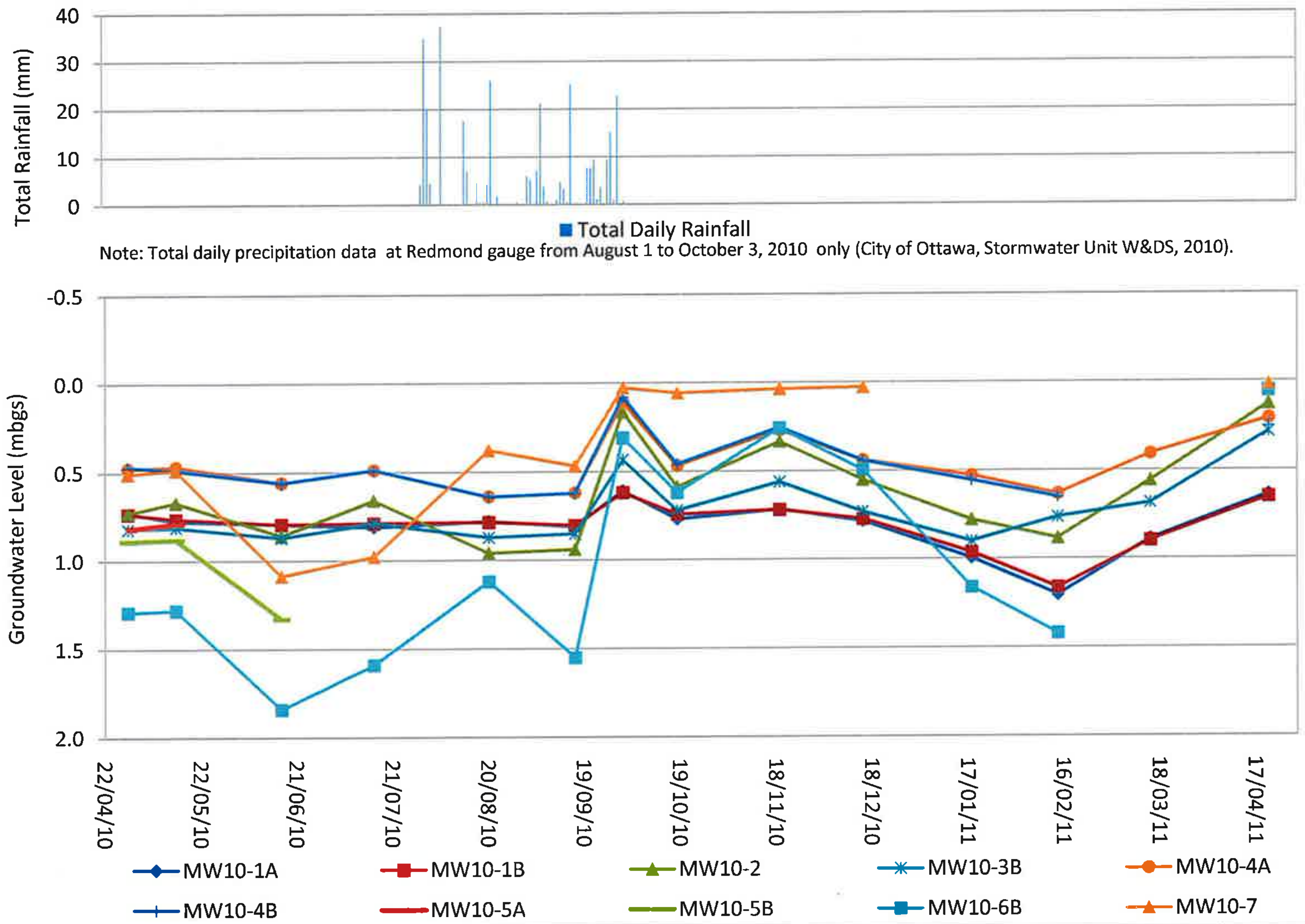
TITLE

SPRING 2011 GROUNDWATER LEVELS

Golder Associates
Ottawa, Ontario

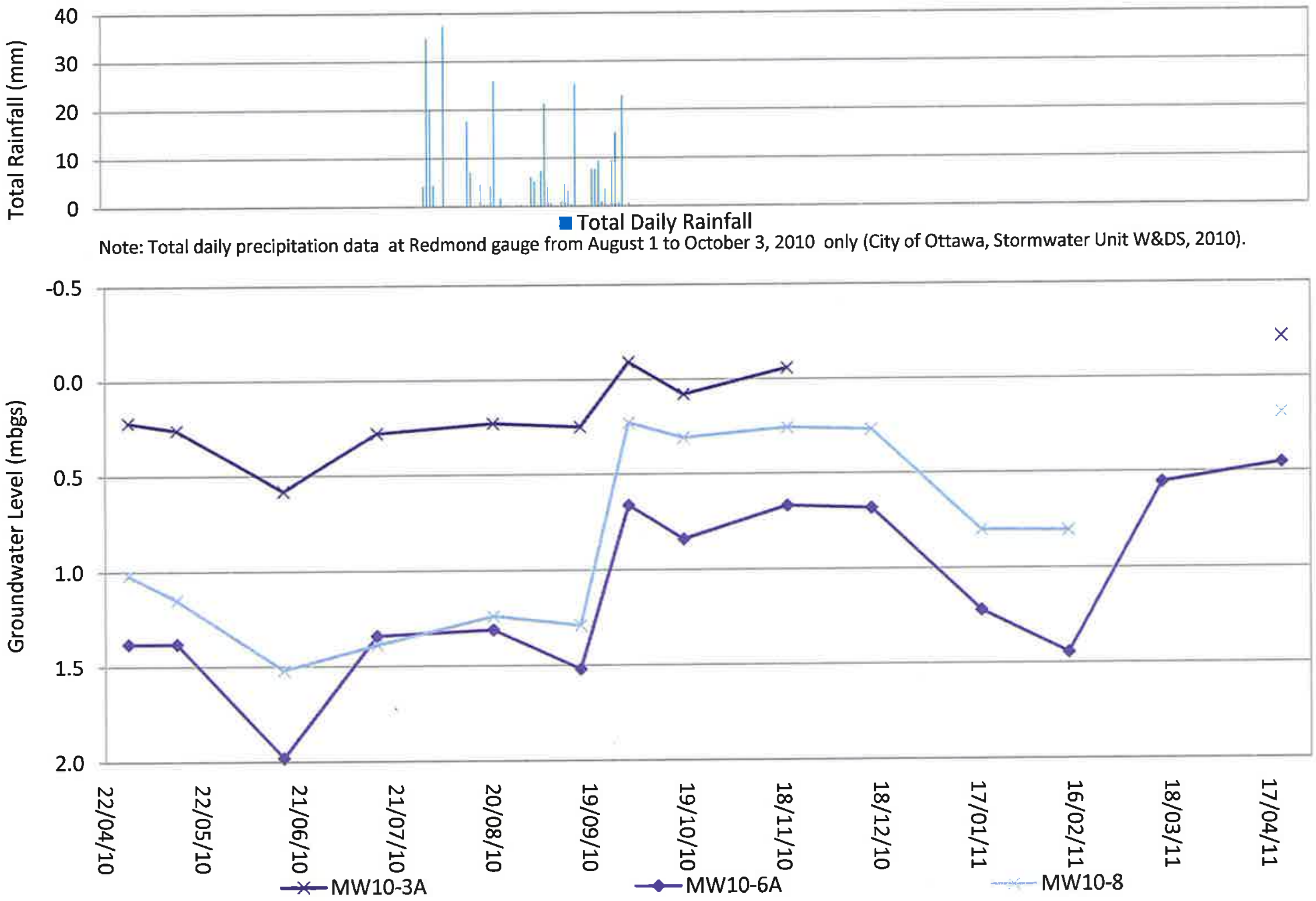
PROJECT No. 08-1122-0078			SCALE AS SHOWN	REV. 0
DESIGN	DH	10 Aug. 2011	FIGURE 2	
GIS	BR	10 Aug. 2011		
CHECK	DH	10 Aug. 2011		
REVIEW	JPAO	11 Aug. 2011		

Path: N:\Active\2008\1122 - Environmental\08-1122-0078 Mattamy Richmond\GIS\mxd\Phase 9600\0811220078-9600-02.mxd



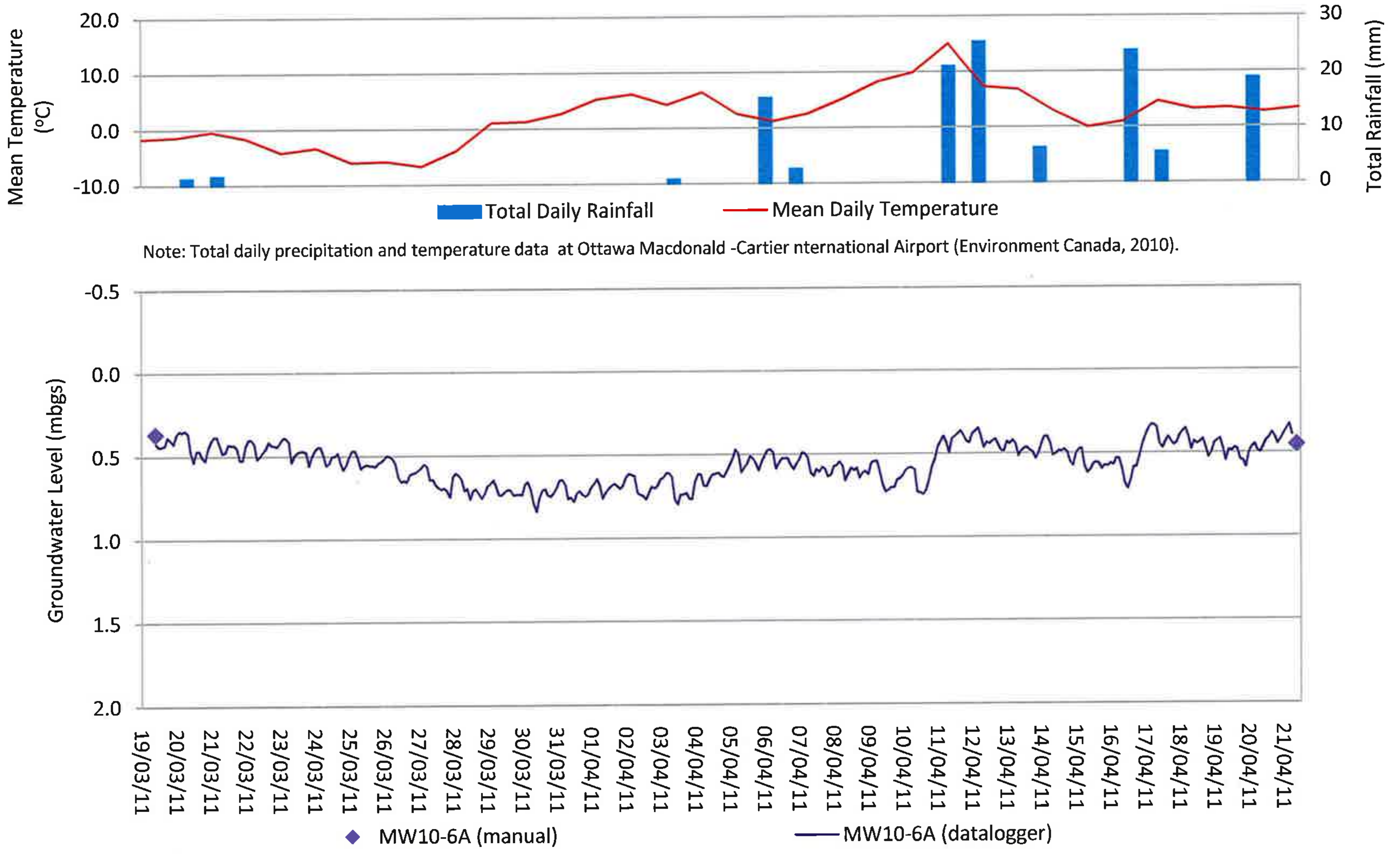
Note: Total daily precipitation data at Redmond gauge from August 1 to October 3, 2010 only (City of Ottawa, Stormwater Unit W&DS, 2010).

Golder Associates	Date: August 2011 Project: 08-1122-0078 (9600)	Drawn: DH Checked: BTB	Groundwater Levels in Overburden Monitoring Wells April 2010 to April 2011	Figure 3



Note: Total daily precipitation data at Redmond gauge from August 1 to October 3, 2010 only (City of Ottawa, Stormwater Unit W&DS, 2010).

Golder Associates	Date: August 2011 Project: 08-1122-0078 (9600)	Drawn: DH Checked: BTB	Groundwater Levels in Bedrock Monitoring Wells April 2010 to April 2011	Figure 4



Note: Total daily precipitation and temperature data at Ottawa Macdonald -Cartier nternational Airport (Environment Canada, 2010).

Notes: MW10-6A is situated in the upper bedrock.

Golder Associates	Date: August 2011 Project: 08-1122-0078 (9600)	Drawn: DH Checked: BTB	Continuous Measurements of Groundwater Levels at MW10-6 During March and April 2011	Figure 5

DATE November 4, 2013 **PROJECT No.** 12-1127-0062 (8000)

TO Frank Cairo
Richmond Village (South) Limited

CC Mike Green
Mattamy (Jock River) Limited

FROM Brian Byerley, M.Sc., P.Eng. **EMAIL** bbyerley@golder.com

BASEMENT DEPTHS, PROPOSED DEVELOPMENT
RICHMOND VILLAGE WESTERN DEVELOPMENT LANDS
OTTAWA, ONTARIO

1.0 INTRODUCTION

This memo provides an assessment regarding proposed basement depths for the Richmond Village (South) and Mattamy (Jock River) developments in the Richmond Village Western Development Lands, in the southwest part of the City of Ottawa.

This issue has been the subject of extensive study and several memos/reports over the past several years. The intent of this current memo is to:

- Compile the applicable information from these previous reports; and,
- Provide a comprehensive recommendation regarding basement depths and, in particular, to document why the proposed founding levels, and the use of sump pumps, are feasible from an engineering/technical perspective.

2.0 BACKGROUND

The proposed development site is approximately 132 hectares (325 acres) in size and is located along the western edge of the Village of Richmond (see Figure 1). The site is legally described as Lot 22, Concessions II, III and IV, Geographic Township of Goulbourn (Village of Richmond). The site boundary is shown on Figure 2.

For the purposes of this memo, and for simplicity of description, the site is described as extending along a north-south axis, with the south limit being adjacent to the Jock River.

The northern part of the site (about two thirds of the area) is actively farmed (corn, wheat and beans) while the southern portion currently consists of fallow fields.

The site is presently zoned for future residential development.

The surrounding lands to the north and west of the site are beyond the Village boundary and are primarily used for agricultural purposes. The lands to the east of the site are within the Village boundary and consist of existing low density residential developments. The Jock River forms the south boundary of the site.

The site is crossed by two existing roadways (Perth Street and Ottawa Street) which approximately divide the site into one-third parcels (north, central, and south).

The ground surface topography across the site is gently sloping, with ground elevations varying from approximately 98 to 94 metres above sea level (masl). The lowest portion of the site is located in the area of Perth Street (between the north and central parcels). To the north of Perth Street, the site is very nearly flat (i.e., just a slight increase in elevation to the north). To the south, the site rises up to Ottawa Street and then to the height-of-land which exists between Ottawa Street and the Jock River.

Berms currently exist along the eastern site boundary, in the vicinity of the Van Gaal Drain, which prevent proper surface water drainage in wet times of the year.

Based on published geological mapping, the subsurface conditions at the site, in a simplified form, can be summarized as follows:

- Overburden soils (see also Figure 3):
    - North half of site (i.e., north parcel and north half of central parcel): marine clay (i.e., Champlain Sea clay);
    - South half of central parcel: fine grained sandy soil (likely existing as a ‘cap’ over the clay underlying layer); and,
    - South parcel: shallow bedrock.
- Bedrock:
    - Dolomite bedrock of the Oxford Formation; and,
    - Bedrock surface outcropping in the south part of the site, near the Jock River, but sloping down to the north and reaching depths of up to 15 metres beneath the north parcel.

Jacques Whitford Limited conducted a preliminary geotechnical investigation on this site, the results of which were produced in a report dated June 22, 2007. Due to the presence of compressible clay soils beneath portions of the site, the geotechnical report recommended grade raise restrictions which generally vary between 1.0 and 2.0 metres (except for the south part of the site, where clay is absent, and a maximum permissible grade raise of 4.0 metres was specified). For the clay areas, the maximum permissible grade raise relates to the capacity of the clay soil to support the weight of grade raise fill without undergoing significant consolidation/compression, which would lead to the settlement of structures, services, and roadways built on the site.

3.0 PURPOSE OF CURRENT ASSESSMENT

From a site development perspective, it is understood that there are several interrelated and opposing challenges associated with the grading design for this site:

- In accordance with the aforementioned geotechnical report, the site grade raise needs to be limited. This type of geotechnical restriction is a common challenge for site development in the Ottawa area, due to the extensive presence of the sensitive and compressible Champlain Sea clay (i.e., Leda clay) deposit. Where the geotechnical permissible grade raises cannot be accommodated, very costly measures can be required, such as the use of expanded polystyrene (EPS) Geofoam lightweight fill blocks for filling around the houses.
- Conversely, if the grading is kept too low, then the footings would be deeper (for conventional houses with basements), below the groundwater level and on soft wet clay (typically grey in colour), which has very little

capacity to support the footings. House footings are ideally constructed in the shallower clay, which is above the 'normal' water level and is therefore generally drier and stiff, since it has been 'weathered' by drying and exposure to air, to form a brown 'crust' (which is about 2 to 3 metres thick on this site). A minimum level of filling, of generally at least 0.5 to 1.0 metres, is therefore commonly needed if the footings will be constructed in the drier upper portions of the brown clay crust (since a standard-depth basement is about 2.4 metres deep relative to the finished grade around the house).

- From the perspective of the economics of developing the site, it is important to approximately achieve a 'cut-fill' balance. That is, the volume of soil excavated to make the excavations down to footing level should ideally equal the soil volume needed on each lot, around the house, to fill to the design finished grade level. If the footing levels are established too shallow, then fill material needs to be imported to the site at significant expense. It is understood that, given the development density, size, and location of this site, any grade raise in excess of 1.3 metres (which corresponds to a footing depth of about 1 metre below existing grade) will result in incremental imported fill costs that can greatly impact the feasibility of the development.

Given the above competing constraints, it is understood that the 'ideal' footing depth (i.e., feasible maximum basement depth) for this site is about 1.0 to 1.3 metres below existing grade elevations.

There is, however, an additional challenge related to the site grading, which is the focus of this memo. Because of the stormwater drainage outlet for this site (to the proposed stormwater management ponds which will ultimately discharge to the Jock River via creeks and existing drainage ditches), it is not feasible to construct house basements at 1.0 to 1.3 metres depth and also provide gravity drainage of the foundation drains (i.e., weeping tile) to the storm sewer system during storm events. Due to the elevations of the storm sewer outlets, the storm sewers will need to be installed at a relatively shallow level, with the obverts and hydraulic grade line (HGL) being above the footing level (although the *invert* levels of the storm sewers will still be below the footing levels).

It is therefore proposed to provide these houses with sump pumps. The sump pumps and weeping tile system will collect groundwater inflows during those times of the year when the groundwater level rises above the footing level and will pump to the storm sewer system. A sketch of the proposed arrangement is provided in Appendix A.

This proposed design, with the use of sump pumps, is similar to what is used for rural housing/developments and what is also understood to currently be used by all/most of the existing houses in Richmond Village. In particular, this system is consistent with what is used in the adjacent existing Richmond Oaks development, which directly abuts the east side of the site. It is understood that the use of similar sump pump systems is also common for urban developments in other municipalities of Ontario.

The City of Ottawa does not currently have clear guidelines on the use of sump pumps for *urban* house construction. In the absence of such guidelines, the acceptability of the proposed footing levels has been evaluated based on geotechnical and hydrogeological assessments, as discussed further in following sections of this memo.

It is understood that, for this site, the City of Ottawa is looking for justification to support the desire to establish footing levels at the 1.0 to 1.3 metre depths which have been proposed (and which are needed to make this development feasible). As discussed above, a *shallower* footing arrangement is not feasible for this site due to the geotechnical restrictions on the permissible grade raise and the large cost/quantity of fill material that would need to be imported to the site (since the site would not have a cut-fill 'balance'). Therefore, it is necessary to document why the proposed founding levels (with the use of sump pumps) are indeed feasible from an engineering/technical perspective.

In the absence of detailed City of Ottawa guidelines, the acceptable founding depths for the use of sump pumps have been evaluated by means of four separate assessments:

1) By the undertaking of technical studies (i.e., groundwater modelling) focused on the expected operating conditions that will apply to the sump pumps;
2) By comparison of the design to the City of Ottawa's practices that are currently applied to individual rural residences, including examination of test pits excavated across the site to directly observe the soil and groundwater conditions;
3) By comparison of the proposed design to other developments with consistent conditions; and,
4) By comparison to the City of Ottawa Sewer Design Guidelines (October 2012).

In the absence of any specified performance objectives by the City of Ottawa, the design team has proposed the following design criteria:

- The footing/basement depths should be selected such that the sump pumps would not operate continuously, but rather, would only need to operate during limited time periods, such as during wet seasons (e.g., spring and fall) or during significant rain events; and,
- When the sump pump is required to operate, the groundwater inflow rate to the foundation drains and the corresponding necessary pumping rate should be well within the capacity of a typical sump pump.

The ultimate objective of these design criteria is to avoid basement flooding, either during power failures or due to overwhelming of the pumping system. It is considered that, provided the above criteria are met, the risk of basement flooding is reasonably small, such as would be accepted by homeowners, insurers, etc., and would be consistent with normal sump pump usage in Ontario.

An added consideration in this assessment is the effect that developing the site will have on the long-term groundwater levels. Much of eastern Ontario is underlain by Champlain Sea clay, which is a soil with a low hydraulic conductivity. As a result, and due to the relatively flat topography prevalent in the area, the natural groundwater levels in Eastern Ontario tend to be relatively shallow. This is also the case for this site, where the shallow groundwater levels reflect the current agricultural land uses and poorly drained conditions which have been created (due to the aforementioned berms which currently prevent the free-drainage of surface water).

However, urban and suburban development is well known to create conditions which lead to long term groundwater level *lowering*. The installation of sewer pipes within a 'surround' of granular material (as needed to support and install the pipes) creates an inherent subsurface drainage system. In addition, natural infiltration is reduced by development, since much of the post-development surface area is relatively impermeable (e.g., roofs, asphalt roadways, etc.) and rainwater is conveyed rapidly to the storm sewer system. This resulting combined effect of subsurface drainage and reduced infiltration causing groundwater level lowering is well known to local geotechnical engineers and, in fact, measures are sometimes implemented to prevent the groundwater level lowering from being excessive (because, in some cases, the lowering can lead to ground settlement). In the case of this site, the use of a sump pump drainage system is made even more feasible when viewed in the context that the post-development groundwater levels will end up lower than the pre-development levels. Considering the rather shallow footing levels that are proposed (at only about 1.0 to 1.3 metres depth), there is little likelihood of groundwater levels persisting above that level after full build-out of the development.

The following sections provide further detail on each of the four assessments, as described above, of the conditions on this site, in terms of using sump pumps.

It should be noted that the assessments discussed in this memo are focused on the north and central portions of the site, which are those parts underlain by clay and the surficial sandy deposit. For the south portion of the site, which is adjacent to the Jock River and where bedrock is near surface, it is proposed to set the footing levels such that the footings and foundation drains should not be below the 100-year flood level in the Jock River.

4.0 TECHNICAL STUDY

Golder Associates Ltd. (Golder Associates) has undertaken a multi-staged numerical modelling program to simulate the hydrogeologic conditions at this site, with the objective of quantitatively evaluating the future groundwater levels and potential inflows to a sump pump system, based on the proposed conceptual design.

4.1 Hydrogeologic Subsurface Investigation

For this assessment to be meaningful, it was necessary to first carry out a supplementary subsurface investigation and monitoring plan to:

- Better evaluate the subsurface stratigraphy on the site;
- Evaluate the hydraulic conductivity of the various strata (e.g., of the clay, sand, and bedrock); and,
- Evaluate the groundwater levels and, if possible, the range of groundwater level variations.

The subsurface investigation was carried out by Golder Associates in April 2010 and included the drilling of eight boreholes across the site, as shown on Figure 3. The boreholes were advanced to depths varying from about 4 to 6 metres below present ground surface. Some of the boreholes were also advanced/cored into the shallower bedrock that exists on the south part of the site. Monitoring wells were installed in the boreholes, including wells at multiple depths in some of the boreholes.

‘Rising head’ testing was carried out in the monitoring wells to evaluate the hydraulic conductivity of the soil and bedrock strata. This testing involved rapidly pumping down the water level in the well (or conversely *raising* the water level in the well) and then monitoring the rate at which the water level recovered.

The investigation program also included monitoring the groundwater levels in the monitoring wells over a 13 month time period, between April 2010 and April 2011, plus two follow-up monitoring sessions in May 2012 and July 2013 (see Table 1).

In summary, the results of the subsurface investigation, hydrogeologic testing, and groundwater level monitoring indicated/confirmed the following:

- The hydraulic conductivities of the key strata are as follows:
    - Surficial weathered brown silty clay crust: 4×10^{-6} to 1×10^{-5} m/s;
    - Underlying unweathered grey silty clay: 5×10^{-6} m/s;
    - Surficial sand and silt (central portion of site): 1×10^{-6} to 7×10^{-6} m/s;
    - Glacial till: 5×10^{-6} m/s; and,
    - Dolomite bedrock: 5×10^{-6} to 1×10^{-4} m/s.

- The groundwater levels varied as follows:
  - From near ground surface to 1.3 metres below ground surface in the spring and fall (average 0.5 metres below ground surface); and,
  - From 0.3 to 1.8 metres below ground surface in the summer and winter (average 0.9 metres below ground surface).

4.2 Numerical Model Construction

Two separate numerical groundwater flow models were developed (using the MODFLOW commercial software). The two models were as follows:

- The first model is referred to as the “Long Term Drainage Model”. It was made to be representative of a large portion of the proposed development area and was used to predict the long term (i.e., steady state) groundwater levels that will ultimately be created at the site, for the post-development condition; and,
- The second model is referred to as the “100-year Storm Event Model”. It was made to be representative of a single lot and was used to evaluate the short-term sump pump response to the 100-year storm and the spring freshet (i.e., during periods of high rain water infiltration and groundwater levels).

The details and findings of these two models are discussed separately below.

4.3 Long Term Drainage Model

The details regarding the construction of the Long Term Drainage Model are summarized as follows:

- This model was used to predict the long term groundwater levels that that will exist over a representative large section of the proposed development, for the post-development condition, and the model therefore covers an area of approximately 700 by 900 metres in size.
- The model topography and size was selected to be representative of the conditions within the central portion of the proposed development, and to represent the conditions for the construction of the first houses (approximately 150 houses in total).
- The hydraulic conductivity values (for horizontal flow) were selected based on the aforementioned testing results, as follows:
  - Overburden: 5 x10^{-6} m/s (for all soil types);
  - Upper weathered bedrock (2 metre thick layer): 5x10^{-5} m/s; and,
  - Deeper bedrock: 5x10^{-7} m/s.

These values are representative of the average measured values from the subsurface investigation.

- The natural (pre-development) groundwater flow was modelled to be to the north-east, towards an existing drainage ditch (un-named tributary).
- The model boundary conditions and parameters were calibrated such that the initial groundwater levels corresponded to the highest recorded groundwater levels from the aforementioned monitoring (which were the water levels recorded in the spring of 2011).

- The effects of developing the site were simulated as follows:
  - The model simulated the installation of higher hydraulic conductivity (i.e., 'free draining') granular material as the granular 'surround' of the storm sewers, based on the alignments and invert depths of the storm sewer system as designed by DSEL, which range from about 2.0 to 2.5 metres below the future roadway surface. [*Note: It is understood that the extent of any higher conductivity pipe surround material would be subject to City approval.*]
  - The proposed 'Pond 1' storm water management pond (SWMP) was included in the model, since it will form a groundwater discharge point. The pond water level in the model was set to the 'normal operating level' of the pond (92.35 masl).
  - Infiltration to the water table from surface (recharge) was set to be consistent with current conditions (i.e., with the existing soil and vegetation cover). This represents a 'conservative' condition, since, as discussed previously, infiltration will actually be reduced due to the impermeable surfaces present in the developed condition (e.g., roofs, pavement, etc.).

The results of the modelling are summarized as follows:

- The long term (i.e., steady-state) water table within the proposed development would be at greater than approximately 1.9 metres depth (relative to the roadway level), which is below the proposed footing and foundation drain level.
- These conditions would be achieved after complete build-out of the storm sewer network. However, the effect of dewatering during construction (i.e., pumping from the trenches while the sewers are installed) and the reduced infiltration that will exist after development would decrease the time required to achieve steady-state conditions.

These results are considered to be applicable to all those portions of the site underlain by silt and clay, which includes essentially the north half of the site.

4.4 100-Year Storm Event Model

The details regarding the construction of the 100-year Storm Event Model are summarized as follows:

- This model focused on making a prediction of the sump pump response of a single house to transient/high groundwater levels, such as would occur during the 100-year storm and during the spring freshet (i.e., thaw). The model was therefore constructed to focus on 'smaller scale' conditions analogous to the development of a single lot/house, and covers an area measuring 10 metres by 14 metres (which allowed more detailed refinement of the model structure around the foundation drains).
- The model ground level and boundaries were selected to correspond to an area that is close to, and therefore hydraulically connected to, the Pond 1 SWMP, such that the groundwater levels could be evaluated during filling of the pond up to the 100-year storm event level.
- The hydraulic conductivities of the strata were consistent with the Long Term Drainage Model, as discussed previously.
- The foundation drain elevation was set at 2.4 metres depth below the finished grade around the house (which is about 2.1 metres below the roadway surface), consistent with conventional basement construction and slightly deeper than currently proposed.

- The conditions during a 100-year storm event were simulated by raising the groundwater level in the storm sewer trench backfill, up to the design storm level in the pond. To further simulate the additional impact of the 100-year storm occurring concurrently with the spring freshet, the recharge was set at 2000 millimetres per year during the same 24 hour period. Two additional simulations were completed in which the recharge was increased to 4000 millimetres per year and 8000 millimetres per year to further evaluate the sensitivity of the model to this parameter. The latter two values are well in excess of the possible actual level of infiltration, even during the spring thaw.

The results of the modelling are summarized as follows:

- The calculated groundwater inflow to the foundation drain is shown on Figure 4.
- The peak anticipated inflow to the foundation drain during the 100-year storm event ranged from 1.4 to 2.0 m^3/day for the range of recharge values simulated (2000 to 8000 millimetres per year). These flows represent the maximum expected sump pump pumping rate due to groundwater inflow.
- Following the storm event, flow to the foundation drains ended within 12 hours (as also shown on Figure 4).

Included in Appendix B is a data sheet for a typical sump pump, such as would be installed in these houses. The capacity of the sump pump, as noted in the Performance Data plot on the second sheet in the attachment, is about 100 Litres per minute for the total 'head' that this pump will need to overcome. This pumping rate is equivalent to more than 140 m^3/day, and therefore is well above the anticipated maximum required pumping rate determined from the modelling.

In the event of power outages, the proposed back-up power for the sump pump configuration will provide an additional level of protection. An assessment of the back-up pump endurance is provided in the DSEL memo provided in Appendix C.

4.5 Modelling Conclusions

The results of this modelling exercise, even if considered to provide a conceptual/qualitative rather than precise quantitative findings, provide the following three key conclusions:

- Consistent with local experience and common knowledge of geotechnical engineers and hydrogeologists, developing this site with an urban residential subdivision will result in a lowering of the groundwater level, and the resulting water table will be below the planned depth of the footings and foundation drains. As a result, sump pumps for this development would not be expected to operate continuously.
- The time required for the groundwater levels to be lowered will not be long, on the order of one year after build-out of the storm sewer network. The time would in fact likely be less, since the modelling did not consider the dewatering that would be carried out during installation of the sewer systems (i.e., the pumping from the trenches during installation of the site services).
- Even during periods of high infiltration, high groundwater levels, and high water levels in the on-site SWMPs, the rate of inflow to the foundation drains would be modest and well below the pumping capacity of a normal sump pump.

5.0 COMPARISON TO CURRENT CITY RURAL PRACTICE USING TEST PIT OBSERVATIONS

Although the City of Ottawa does not have detailed guidelines regarding the use of sump pumps for urban developments (and the corresponding founding levels and site grading design), there is understood to be an ‘unwritten’ protocol for establishing the suitable grading for individual rural houses. It is our understanding that the protocol for rural houses is as follows:

- The homeowner’s septic system contractor has a test pit excavated at the site.
- A representative of the Ottawa Septic Office (which is managed by the local Conservation Authority) inspects the test pit and establishes where the groundwater level is located. This level is established either by observed seepage or by the colour change in the soil, with the soil being browner above the water table and greyer below. The absorption trenches of the septic system then need to be constructed 0.9 metres above that level.
- The City of Ottawa’s building inspector uses that groundwater level to set the deepest allowable footing level for the house.

This protocol appears to operate from the practical perspective that the basement should be located above the groundwater level that is established based on observations made in an actual excavation at the site just prior to issuance of the building permit.

A series of test pits was thus excavated across this site in late July 2013 under observation by Golder Associates. In total, 19 test pits were excavated across the site, and were extended to depths ranging from 1.4 to 2.2 metres. City of Ottawa staff were present for excavation of several of the test pits.

The test pits were spread across the site, including the clay (north), sandy (south-central), and shallow bedrock (south) parts of the site.

The conditions observed in the test pits are summarized as follows:

- In general, groundwater seepage was observed to be at about 1.0 to 1.3 metres depth.
- A locally deeper groundwater level was observed in one test pit excavated in the sandy portion of the site, at 1.8 metres depth.
- In both the clay and sandy portions of the site, the rate of groundwater inflow to the test pits was similar and very modest. From a practical perspective, there was no noticeable difference in the rate of inflow between the clay and the sand. The sandy deposit was observed to be very fine grained (i.e., to be very ‘silty’) while the clay has very fine fissures. The result is that the conditions in the two deposits are similar (similar hydraulic conductivity), resulting in minor groundwater inflow.
- From a geotechnical/constructability perspective, it was also observed that the clay deposit at 1 metre depth was sufficiently dry and stiff to form a suitable subgrade for footing construction; i.e., the clay soil at this depth was dry to touch (did not wet the hand) and dry enough to stand on without the clay being slippery.

Based on these conditions, it is considered that a founding level at about 1 metre depth would be entirely acceptable from both a geotechnical and hydrogeological perspective. It would not be expected that, with basements constructed to this depth, the sump pumps would be required to operate other than during the spring/fall or during rain events. In addition to providing a very direct and practical assessment of the appropriate founding depth (i.e., for which sump pumps would have to work only intermittently), this exercise also emulated what is understood to be the City of Ottawa's practice for establishing the lowest founding depth for rural houses.

In regards to this assessment, the following two additional points should be noted:

- These test pits were excavated during a period of raining weather; and,
- The surface water drainage on the lower-lying portions of the site, near Perth Street, has been intentionally blocked by the construction of berms. That condition, which impedes the free runoff of rainwater to the ditches and adjacent municipal drains, has likely created a condition of increased recharge and of groundwater levels that are elevated and unrepresentative of the levels that would otherwise exist.

Figures 5a through 5g present plots of groundwater elevations at the monitoring wells, the proposed USF and storm sewer invert elevations in the vicinity of the monitoring wells, and schematics showing a foundation wall and footing drain. Only at MW10-4 and MW10-7 are the groundwater levels consistently above the proposed USF elevations. However, at these locations the storm sewer invert will be below the proposed USF elevations (as shown on Figures 5a through 5g). Therefore, because the granular material surrounding the storm sewers will control the groundwater levels (i.e. will lower the water table), the normal water table elevation will be below the USFs and will rise above the USFs only during spring/fall and rain events. [*Note: it is our understanding that groundwater level monitoring will be undertaken during site development, to monitor the effects of site grading and the installation of sewer infrastructure. DSEL will review and possibly increase USF elevations in the vicinity of MW10-4 and MW10-7 at the detailed grading design stage, based on the groundwater level monitoring results.*]

In summary, it is concluded that, based on the conditions directly observed in actual test pits at the site, a founding depth of up to 1 metre is entirely feasible and appropriate from the perspective of the operation of sump pumps (i.e., having only intermittent/seasonal flow that is well within the pump's capacity). This assessment is based on:

- The depths at which water seepage was observed in the test pits, which was at or below that level;
- The observed modest rates of groundwater inflow, even for the test pits excavated in the sandy soil; and,
- What the acceptable founding levels would be if this work had been carried out for setting the founding level of a rural house, based on our understanding of the City of Ottawa's current protocol.

6.0 COMPARISON TO OTHER DEVELOPMENTS

The proposed founding depth of 1 metre is very common and consistent with other developments in the Ottawa area, both for those equipped with sump pumps as well as those with gravity controlled foundation drainage systems. In fact, for the latter case, houses are frequently founded at greater depth in the clay, because a deeper founding level is required when the permissible grade raise is low, as established by the geotechnical conditions (i.e., based on the capacity of the clay to support the weight of the grading fill). To our knowledge, we are unaware of any difficulties in the City of Ottawa with basements that are installed to their full 2.4 metre depth in these clay deposits, even when located below the groundwater level. Although drainage systems may initially convey a continuous flow of groundwater, the flow is not sustained because the groundwater level is ultimately lowered to at/below footing level, such that the foundation drains only operate during wet seasons. These conditions are considered representative of the performance expected at this site. That is, there should be little to no actual day-to-day sustained flow into the foundation drains. The sump pumps will only be required to operate intermittently, during storms and wet seasons.

There is an existing development which is located immediately adjacent to the east side of this site which is useful for comparison purposes. The following details are understood about that development, which is known as Richmond Oaks:

- The ground conditions are understood to be similar (largely consisting of clay and a shallow water table);
- Based upon plans available, the founding depths of the houses appear to range from 0.8 to 1.1 metres below original ground surface, and the site grade raise is approximately 1.2 metres, which is consistent with what is proposed for this site. The houses are serviced with sump pumps which discharge to storm sewers; and,
- The footing level is below the storm sewer HGL.

We are not aware of any issues regarding basement flooding or the performance of the foundation drainage system and sump pumps in this development. Considering the similarity between the two developments, it would not be unreasonable to permit the use of sump pumps on the currently proposed site.

More generally, it is understood that essentially all of the houses in the entire Village of Richmond use foundation drainage systems with sump pumps. It is understood that DSEL contacted the insurance industry to obtain data on the history of claims related to basement flooding and that inquiry revealed no history of such claims. By extension, there would therefore appear to be no technical reason, from a hydrogeologic perspective, to not similarly permit the use of sump pumps for this site.

Thus, there appears to be no history of basement flooding problems in Richmond that would indicate issues with sump pump operations.

7.0 CITY OF OTTAWA SEWER DESIGN GUIDELINES

The October 2012 City of Ottawa Sewer Design Guidelines make reference to sump pumps in Section 5.7.1 (Connections General), which states, “Three types of systems are available…(2) Sump pit and pump discharging to a ditch.” Although the proposal is to discharge to a storm sewer, as the development will not be serviced with ditches, the guideline allows for the use of sump pumps. Section 6.4.3 (Water Table Considerations) states, “All development shall ensure that each foundation footing is above the average water table elevation.” As discussed in Sections 4.0, 5.0 and 6.0 of this memorandum, developing this site with an urban residential subdivision will result in a lowering of the groundwater level, as is common for developments on clay in the City of Ottawa, such that foundation footings will be above the average water table elevation.

8.0 SUMMARY AND CONCLUSIONS

This memo has summarized the results of four separate assessments regarding the basement/founding levels and the use of sump pumps in this development. It is our opinion, based on these three assessments, that the conditions on this site are suitable to have the footings and foundation drains constructed at up to 1 metre depth. More specifically, it has been shown that the following design/performance criteria would be met:

- The sump pumps would not operate continuously; and,
- When required to operate, the pumping rate would be well within the capacity of the pumps.

These conclusions have been based upon the following:

- Hydrogeologic analyses have shown that, within about a year of the site being developed (and probably sooner), the steady-state groundwater level would be lowered to below the footing level;

- Hydrogeologic analyses have also shown that, during periods of high precipitation/infiltration, when groundwater levels and surface water levels would be high (such that the sump pumps would be required to operate), the rates of inflow to the foundation drains would be well below the capacity of a normal sump pump;
- Test pits excavated across the site have shown that groundwater inflow was only observed below about 1.0 to 1.3 metres depth, the clay above that level is rather dry, and the actual rates of groundwater inflow to the test pits were modest. Furthermore, this assessment methodology was consistent with the practice which has been successfully used for setting the basement levels of individual rural houses;
- The adjacent development has similarly been constructed with sump pumps, with the basements at similar depths, and there are no known troubles or history of basement flooding;
- The entire Village of Richmond is understood to be serviced with sump pumps, and insurance records show no history of basement flooding; and,
- The proposed use of sump pits and pumps is consistent with the current City of Ottawa Sewer Use Guidelines.

At a very simple and conceptual level, it must be recognized that the proposed 1 metre founding depth is very shallow. It is also well known to local geotechnical engineers and hydrogeologists that development of sites with clay or low permeability soils results in groundwater level lowering in the long term. Therefore, given the low hydraulic conductivity of the soils and the shallow founding depths proposed on this site, there is no reason to believe that basement flooding due to groundwater inflow should be of concern.

9.0 CLOSURE

We trust that this memo is adequate for your current needs. Please contact the undersigned if you have any questions.

Yours truly,

GOLDER ASSOCIATES LTD.

for
Mike Cunningham, P.Eng.
Associate, Geotechnical Engineer

Brian Byerley, M.Sc., P.Eng.
Principal, Senior Hydrogeologist

B. T. BYERLEY
Nov 4/13

MIC/BTB/sg
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Attachments:

Important Information and Limitations of this Memorandum
Table 1 – Groundwater Monitoring Data
Figure 1 – Key Plan
Figure 2 – Site Plan
Figure 3 – Surficial Geology
Figure 4 – Simulated Flows to Foundation Drain (100-Year Storm Model)
Figures 5a through 5g – Groundwater vs Proposed USF
Appendix A – Sump Pump Detail
Appendix B – Sump Pump Data Sheet
Appendix C – DSEL memorandum dated October 30, 2013

IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, Richmond Village (South) Limited and Mattamy (Jock River) Limited. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

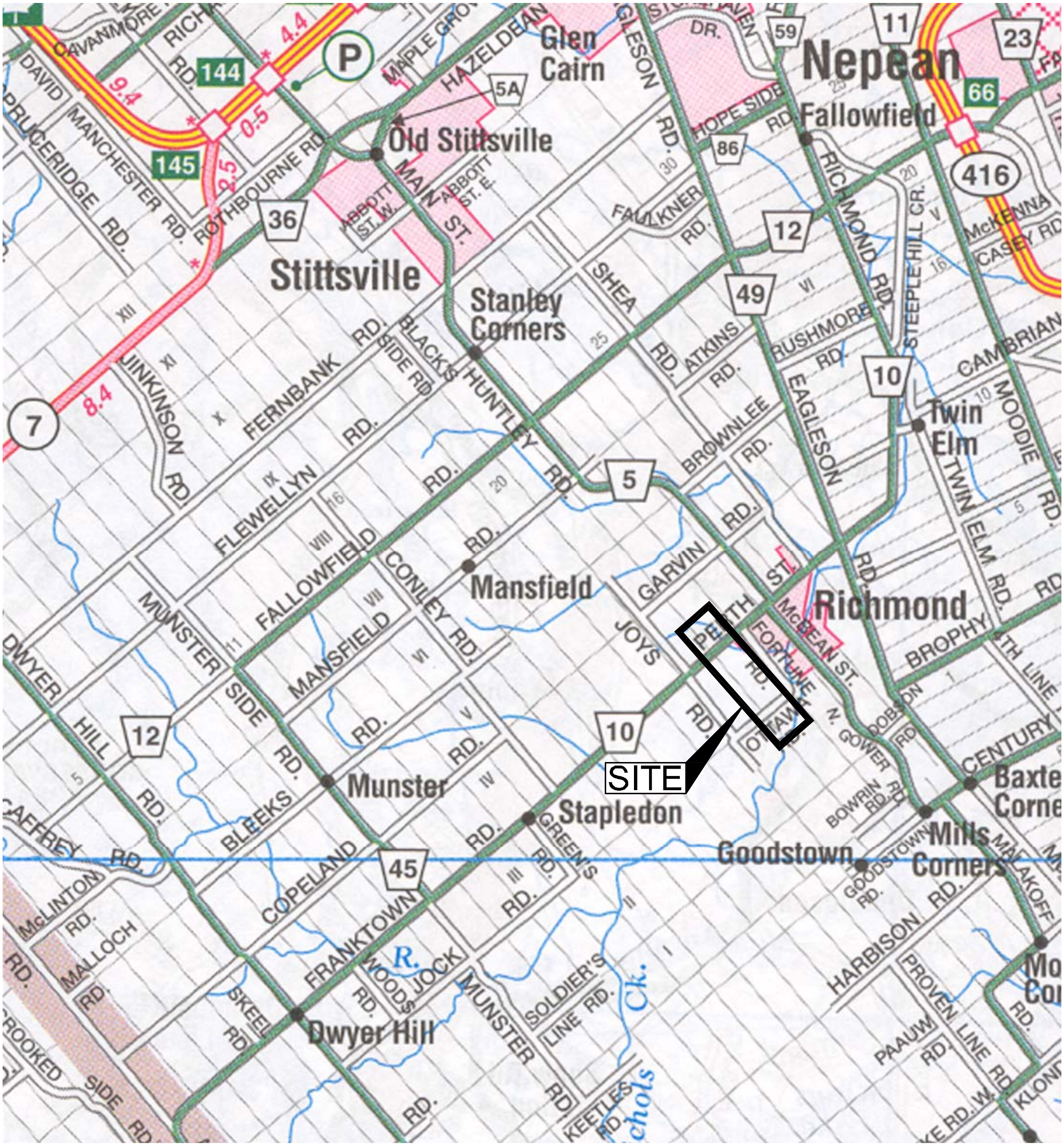
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

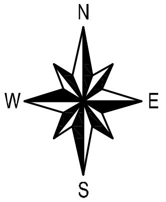
Table 1: Groundwater Monitoring Data

Well ID	Ground Surface Elevation (geodetic)[1]	Screen depth (middle of screen) (mbgs)	Soil/rock at depth of well screen	Groundwater Depths (mbgs)															
				Apr.29 & 30, 2010	May 14, 2010	Jun.16, 2010	Jul. 15, 2010	Aug. 20, 2010	Sept. 16, 2010	Oct. 1, 2010	Oct 18, 2010	Nov. 19, 2010	Dec. 15, 2010	Jan. 18, 2011	Feb. 14, 2011	Mar. 15, 2011	Apr. 21, 2011	May 1, 2012	July 31, 2013
MW10-1A	94.55	3.15	Grey silty Clay	0.67	0.72	0.74	0.75	0.72	0.75	0.55	0.71	0.66	0.73	0.94	1.15	0.83	0.58	0.77	0.92
MW10-1B	94.55	1.21	Grey brown silty Clay (weathered crust)	0.66	0.69	0.72	0.72	0.71	0.73	0.55	0.67	0.65	0.70	0.89	1.09	0.83	0.58	0.75	1.23
MW10-2	94.90	2.12	Grey brown silty fine Sand	0.70	0.64	0.83	0.63	0.93	0.91	0.14	0.56	0.31	0.53	0.75	0.86	0.53	0.10	--[5]	0.82
MW10-3A	93.99	4.55	Fresh grey Dolomite	0.18	0.22	0.54	0.24	0.19	0.21	-0.13[2]	0.04	-0.09[2]	--[4]	--[4]	--[4]	--[4]	-0.25[2]	-0.09	0.08
MW10-3B	93.99	2.12	Grey brown silty Clay (weathered crust) / grey brown fine sandy Silt	0.81	0.80	0.86	0.78	0.86	0.84	0.42	0.71	0.55	0.72	0.89	0.75	0.67	0.27	0.61	0.68
MW10-4A	94.34	3.03	Grey brown fine sandy Silt	0.41	0.40	0.49	0.42	0.57	0.55	0.04	0.40	0.19	0.37	0.46	0.56	0.34	0.14	--[5]	0.46
MW10-4B	94.34	1.21	Grey brown silty Clay (weathered crust)	0.40	0.42	0.49	0.42	0.57	0.55	0.01	0.39	0.18	0.37	0.48	0.58	--[4]	0.18	0.46	0.61
MW10-5A	94.82	3.03	Glacial Till	0.81	0.78	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]
MW10-5B	94.82	1.21	Grey brown silty fine Sand	0.85	0.84	1.29	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]	--[3]
MW10-6A	95.67	4.24	Fresh grey Dolomite	1.36	1.36	1.96	1.32	1.29	1.50	0.64	0.82	0.65	0.66	1.20	1.42	0.53	0.43	0.94	1.16
MW10-6B	95.67	1.52	Grey brown silty Sand trace Clay	1.27	1.26	1.82	1.57	1.10	1.53	0.29	0.60	0.23	0.47	1.14	1.40	--[4]	0.03	0.85	1.15
MW10-7	95.36	2.42	Grey brown silty fine Sand	0.46	0.44	1.04	0.93	0.33	0.42	-0.02[2]	0.01	-0.01[2]	-0.02[2]	--[4]	--[4]	--[4]	-0.03[2]	--[5]	0.44
MW10-8	96.32	2.42	Weathered to fresh grey Dolomite	0.98	1.11	1.48	1.35	1.20	1.25	0.19	0.27	0.22	0.23	0.76	0.77	--[4]	0.15	0.40	0.77

Notes:

Groundwater depth measurements revised in September 2013 to reflect surveyed top of casing and ground surface elevations (From May 14, 2010). Previously presented data reflected manually measured height of casing at time of well construction.
Ground surface elevations at MW10-8 and MW10-5 revised in September 2013 as per surveyed elevations
1: Survey completed on May 14, 2010 by J.D. Barnes Limited (Ottawa).
2: Artesian conditions exist. Groundwater level above ground surface.
3: Monitoring well MW 10-5 A and B vandalized and groundwater levels not available.
4: Groundwater in monitoring well frozen. Depth to groundwater level could not be measured.
5: Only select wells were monitored in May 2012 as a component of a hydraulic response testing program.





NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. TECHNICAL MEMORANDUM No. 12-1127-0062-8000

Golder Associates
Ottawa, Ontario, Canada

SCALE 1:100,000
DATE 2013-10-16
DESIGN
CAD JM
CHECK MIC
REVIEW BTB

FILE No. 1211270062-8000-01.dwg
PROJECT No. 12-1127-0062
REV.

TITLE

KEY PLAN

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

FIGURE 1

PLOT DATE: October 16, 2013
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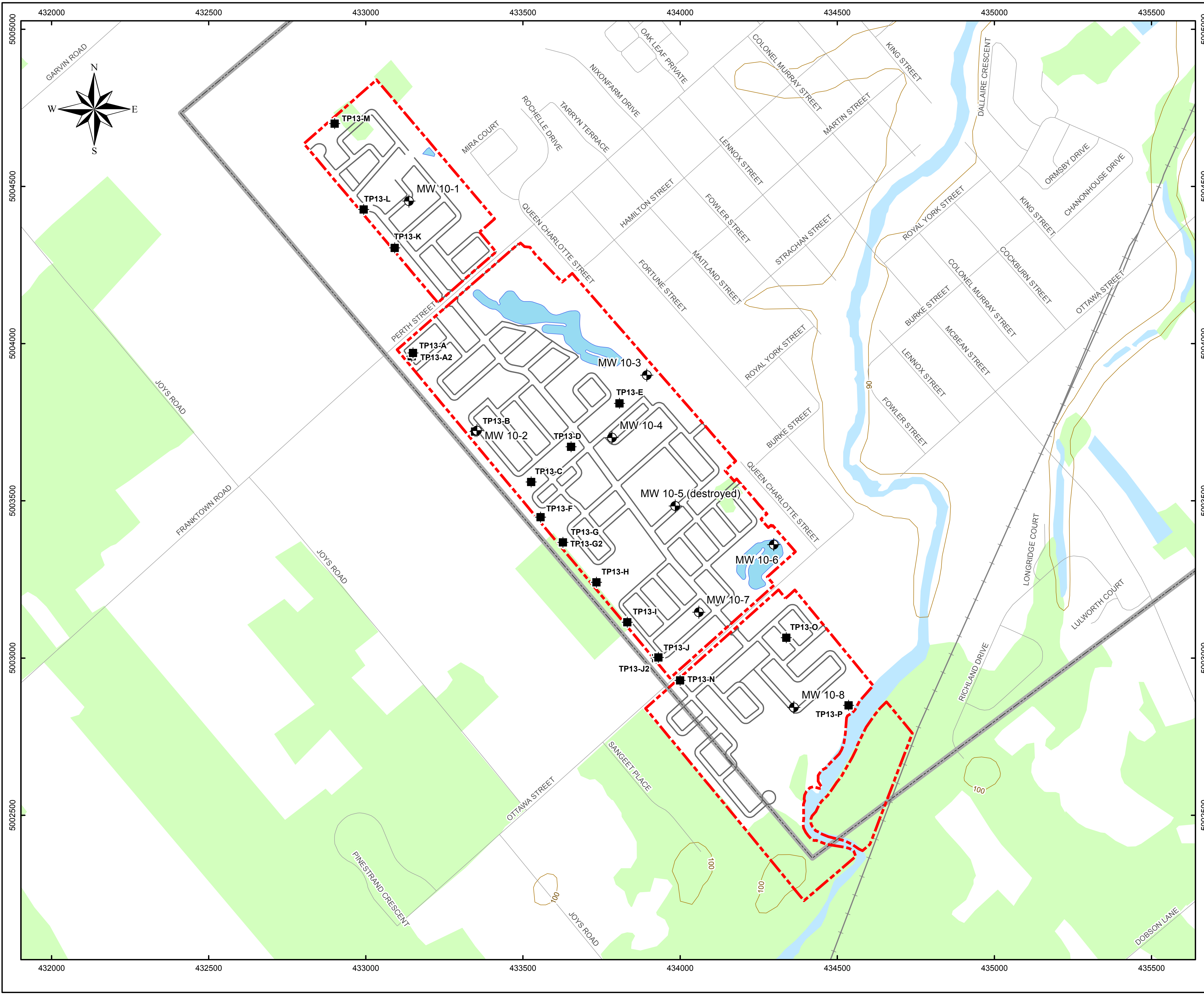

LEGEND

- ⊕ MONITORING WELL LOCATION
- ■ TEST PIT LOCATION (JULY 2013)
- RAILWAY
- PROPOSED ROADS
- ROAD
- TOPOGRAPHIC CONTOUR (m)
- 🟦 PROPOSED STORMWATER MANAGEMENT POND
- ▭ RICHMOND VILLAGE BOUNDARY
- 🟦 WATERBODY
- 🟩 VEGETATION
- 🟥 SITE BOUNDARY

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES TECHNICAL MEMORANDUM No. 12-1127-0062-8000

REFERENCE

BASE DATA - CANVEC PROVIDED BY HER MAJESTY THE QUEEN IN RIGHT OF CANADA, DEPARTMENT OF NATURAL RESOURCES
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: UTM ZONE 18

350 175 0 350 Meters

1:12,000

PROJECT

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

TITLE

SITE PLAN

Golder Associates
Ottawa, Ontario

PROJECT No. 12-1127-0062			SCALE AS SHOWN
DESIGN	NB	2012-05	
GIS	JEM/BR	2013-09	
CHECK	MIC	2013-10-16	
REVIEW	BTB	2013-10-16	

REV. 0.0

FIGURE 2

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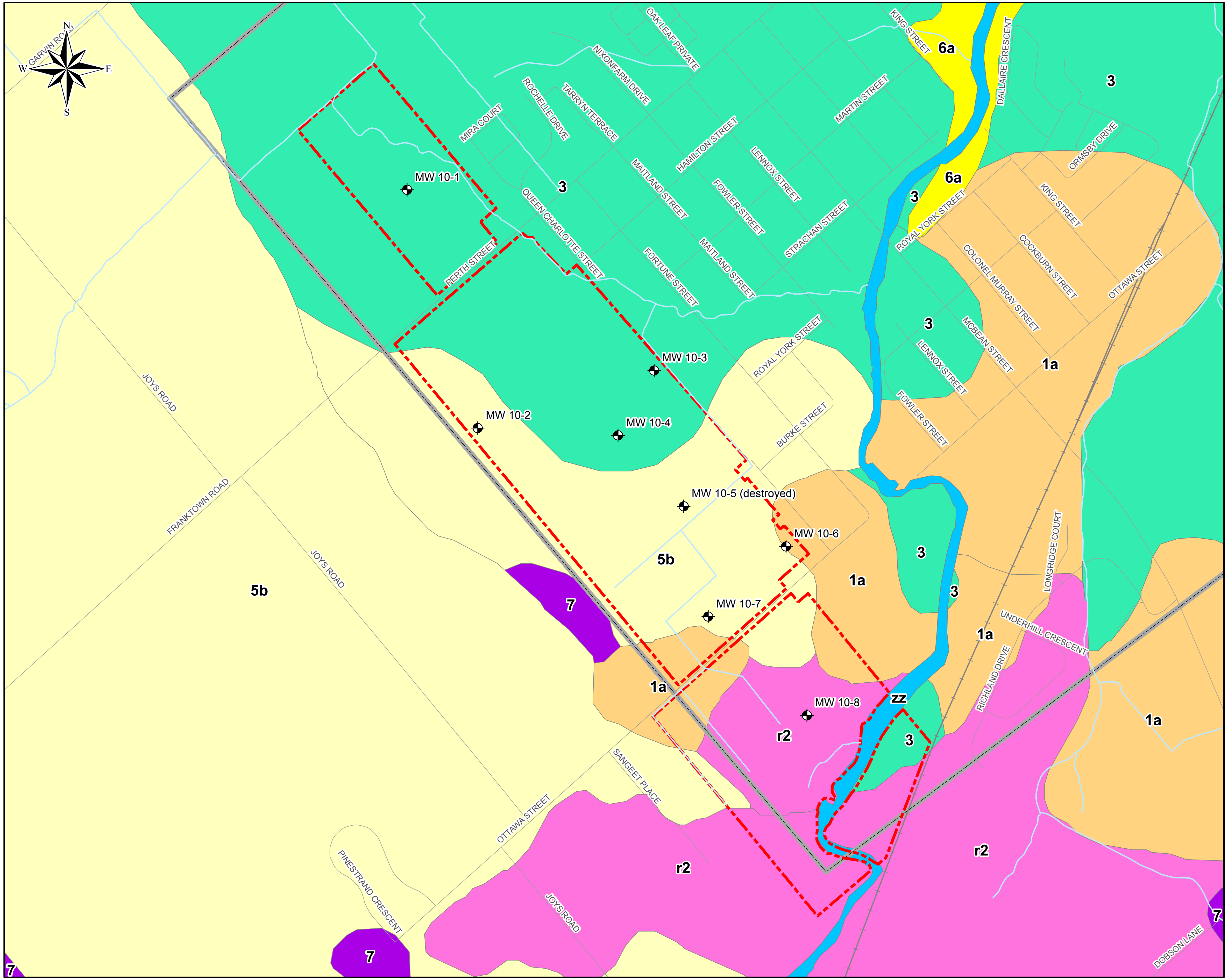

LEGEND

⊕ MONITORING WELL LOCATION
— WATERCOURSE
┼ RAILWAY
— ROADWAY
▭ PROPOSED DEVELOPMENT LANDS
▭ RICHMOND VILLAGE BOUNDARY

SURFICIAL GEOLOGY

🟪 7 Organic Deposits: Muck & Peat
🟨 6a Alluvial Deposits: Silty Sand, Silt, Sand & Clay
🟨 5a Nearshore Sediments: Gravel, Sand & Boulders
🟨 5b Nearshore Sediments: Fine To Medium Grained Sand
🟩 3 Offshore Marine Deposits: Clay, Silty Clay & Silt
🟧 1a Till, Plain With Local Relief <5m
🟩 3a Offshore Marine Deposits: Clay, Silt Underlying Erosional Terraces
🟫 1c Till, Hummocky To Rolling With Local Relief 5 To 10m
🟪 r2 Bedrock: Limestone, Dolomite, Sandstone & Local Shale
🟦 zz Water

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES TECHNICAL MEMORANDUM. 12-1127-0062-8000

REFERENCE

BASE DATA - CANVEC PROVIDED BY HER MAJESTY THE QUEEN IN RIGHT OF CANADA, DEPARTMENT OF NATURAL RESOURCES
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: UTM ZONE 18

475 237.5 0 475
Meters
1:12,000

PROJECT

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

TITLE

SURFICIAL GEOLOGY

Golder Associates
Ottawa, Ontario

PROJECT No. 12-1127-0062			SCALE AS SHOWN	REV. 0.0
DESIGN	NB	2012-05	**FIGURE 3**	
GIS	ABD	2012-07		
CHECK	MIC	2013-10-16		
REVIEW	BTB	2013-10-16		

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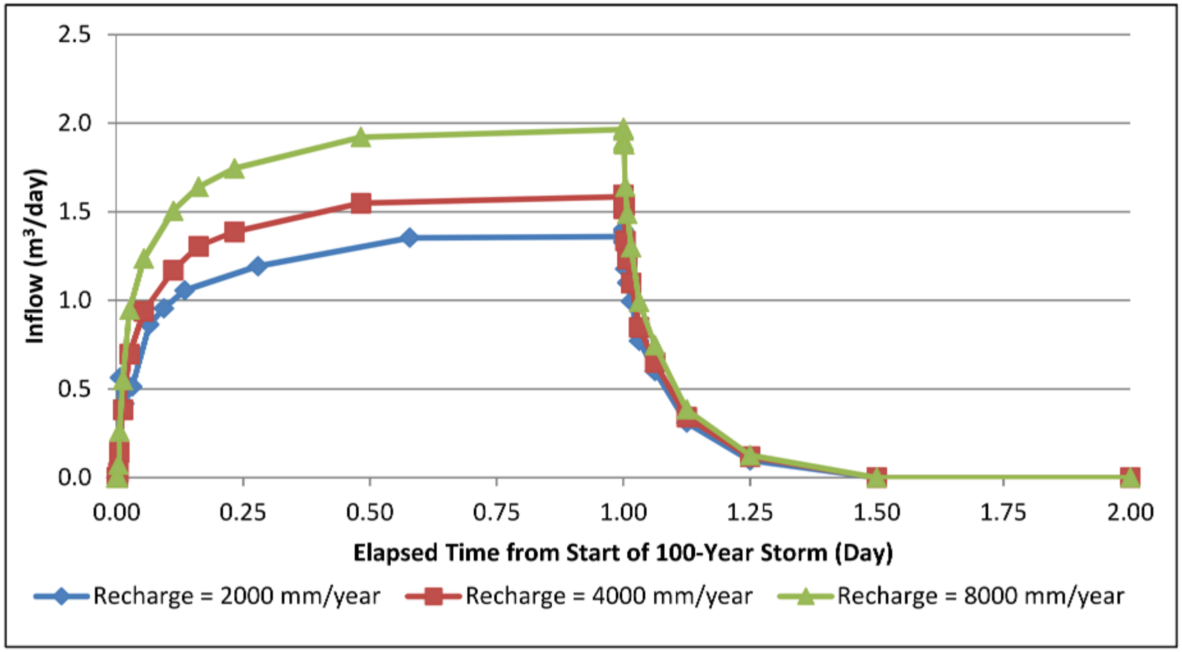


NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH
THE ACCOMPANYING GOLDER ASSOCIATES LTD.
TECHNICAL MEMORANDUM No. 12-1127-0062-8000

Golder Associates
Ottawa, Ontario, Canada

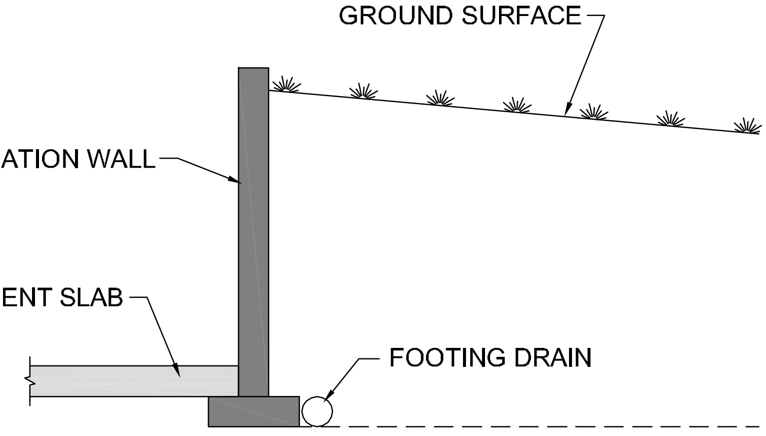
RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

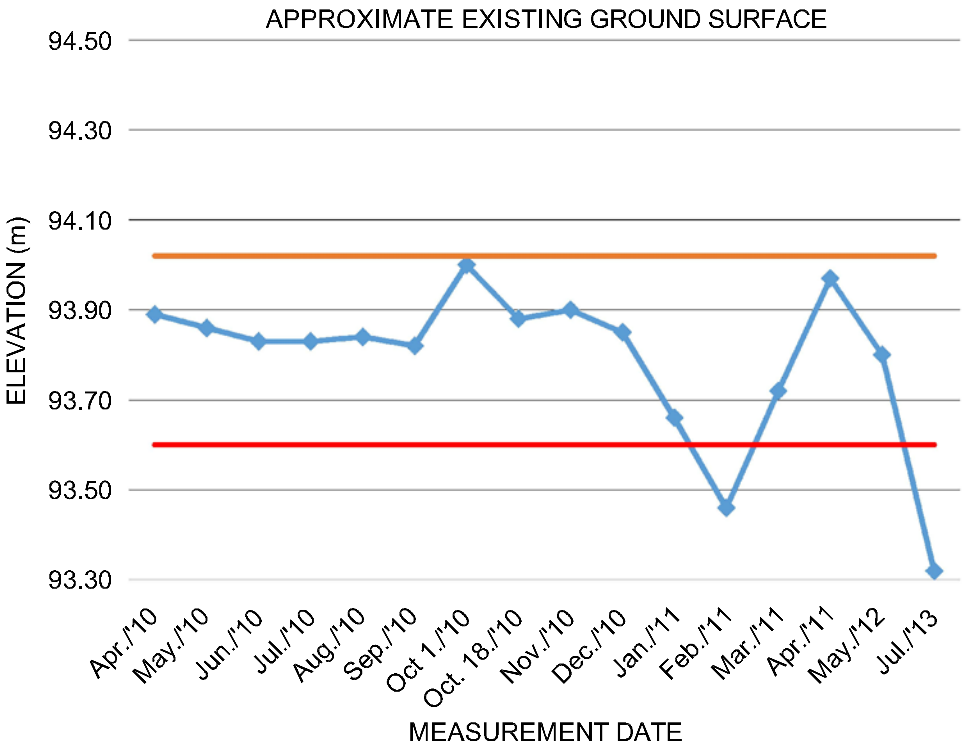
SIMULATED FLOWS TO FOUNDATION DRAIN (100-YEAR STORM MODEL)

FIGURE 4

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

PLOT DATE: November 1, 2013
FILENAME: N:\Active\2012\1127 – Geosciences\12-1127-0062 Richmond Village Drainage and Sump Design\Spatial_IM\CAD\Phase 8000\1211270062-8000-05.dwg





LEGEND

◆ MW10-1

🟧 ESTIMATED USF @ MW10-1

🟥 STORM SEWER INVERT

NOTE

1. CONCEPTUAL USF ELEVATIONS PROVIDED BY DSEL

2. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

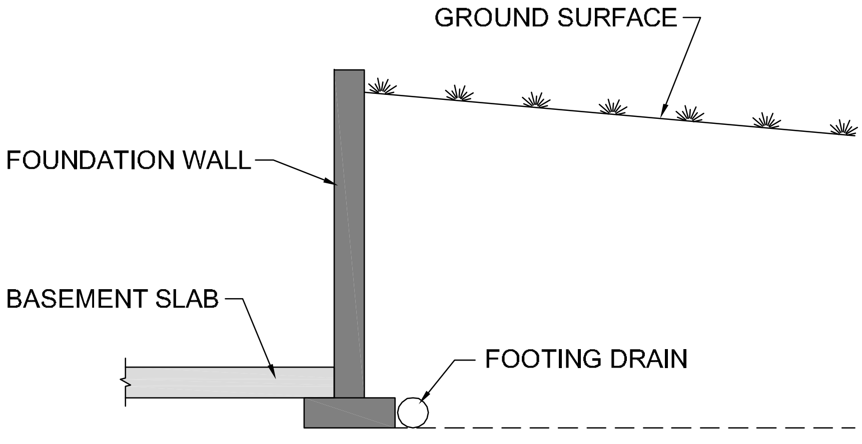
GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-1

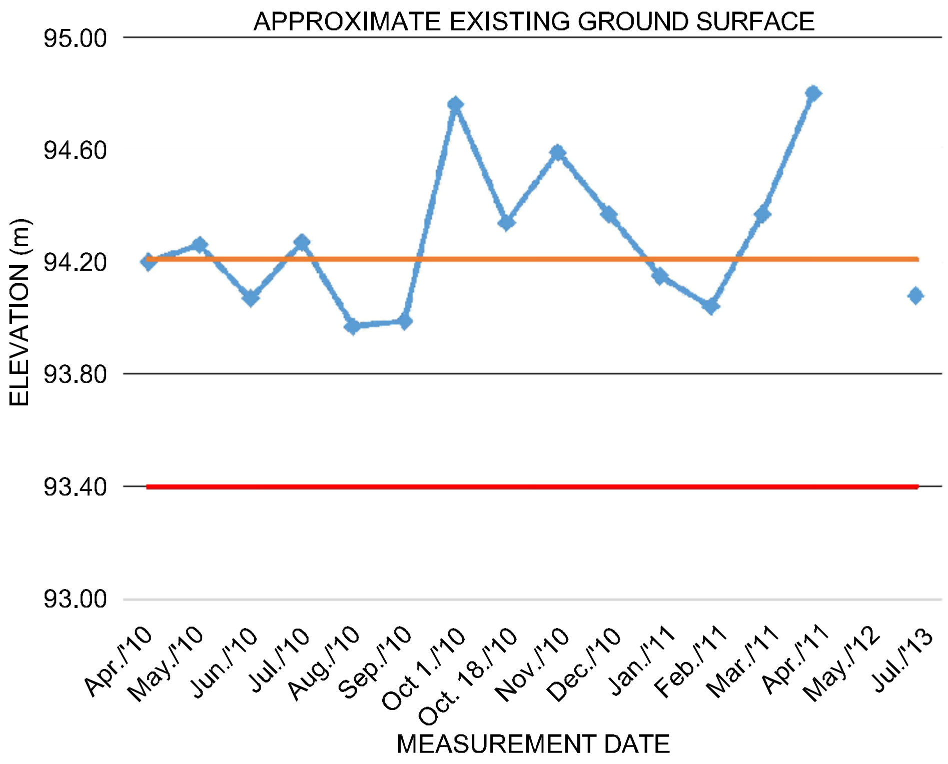
FIGURE 5A

Golder Associates
Ottawa, Ontario, Canada

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

PLOT DATE: November 1, 2013
FILENAME: N:\Active\2012\1127 – Geosciences\12-1127-0062 Richmond Village Drainage and Sump Design\Spatial_IM\CAD\Phase 8000\1211270062-8000-05.dwg





LEGEND

- MW10-2
- ESTIMATED USF @ MW10-2
- STORM SEWER INVERT

NOTE

1. CONCEPTUAL USF ELEVATIONS PROVIDED BY DSEL
2. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

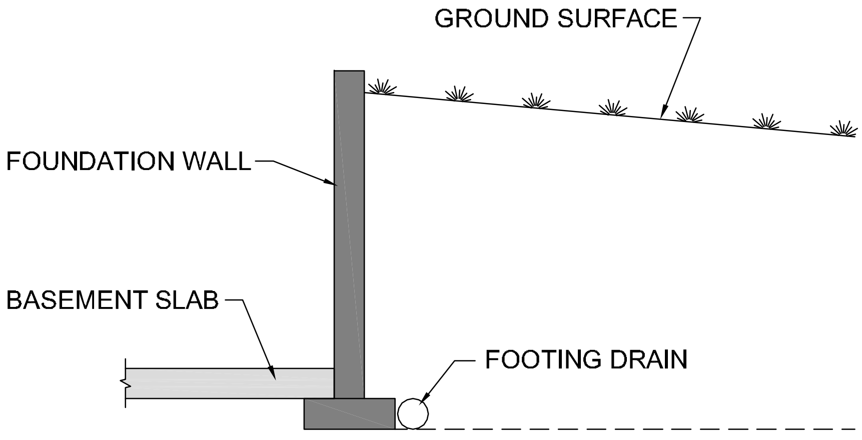
Golder Associates
Ottawa, Ontario, Canada

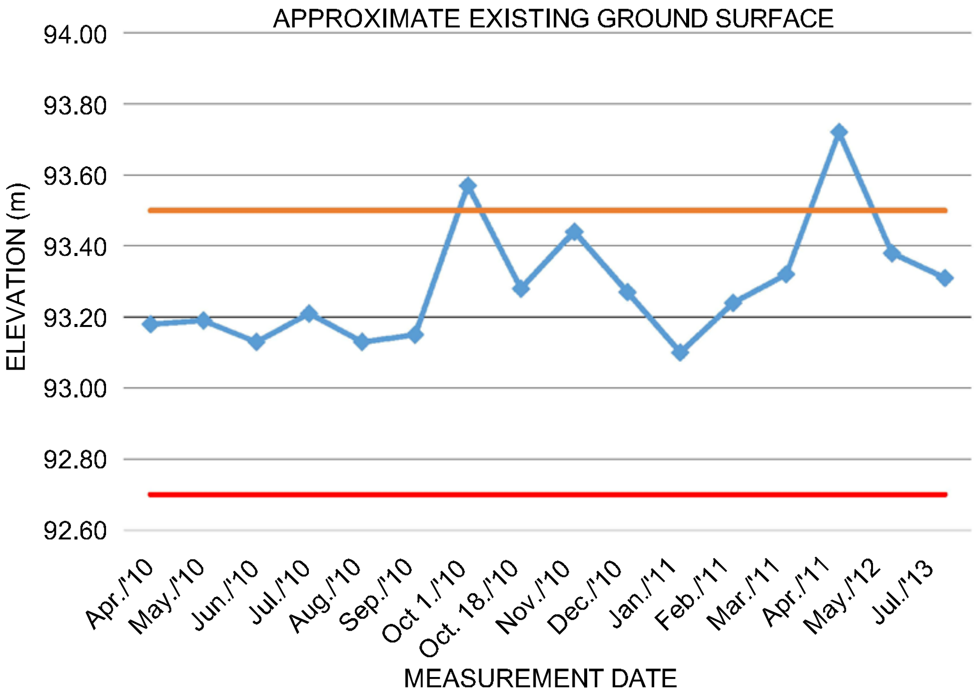
RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-2

FIGURE 5B

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM





LEGEND

- ◆ MW10-3
- 🟧 ESTIMATED USF @ MW10-3
- 🟥 STORM SEWER INVERT

NOTE

1. CONCEPTUAL USF ELEVATIONS PROVIDED BY DSEL
2. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

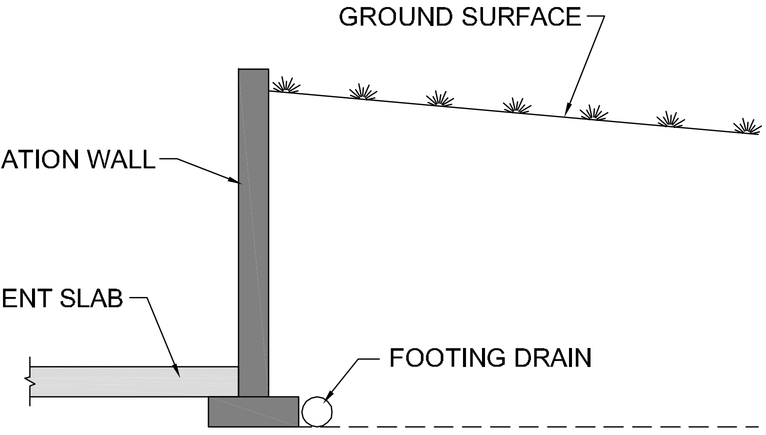
Golder Associates
Ottawa, Ontario, Canada

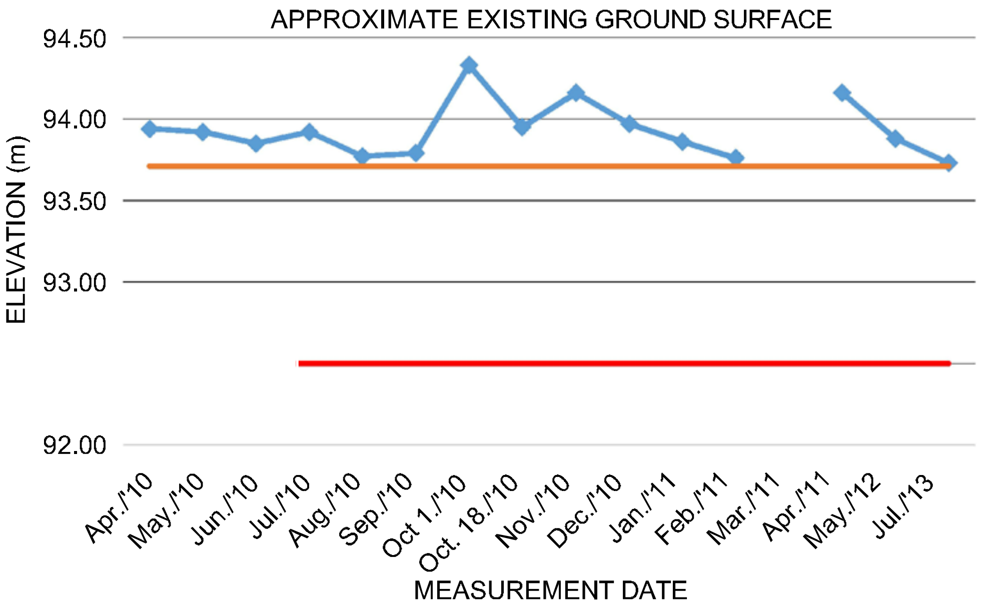
PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-3

FIGURE 5C





LEGEND

- ◆ MW10-4
- 🟧 ESTIMATED USF @ MW10-4
- 🟥 STORM SEWER INVERT

NOTE

1. CONCEPTUAL USF ELEVATIONS PROVIDED BY DSEL

2. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

3. IT IS RECOMMENDED THAT AREA SPECIFIC USF RAISES IN EXCESS OF THE GENERIC USF ELEVATION SHOWN BE REVIEWED AT THE DETAILED DESIGN STAGE TO BRING USF ELEVATIONS IN LINE WITH ANY FUTURE MONITORED GROUNDWATER ELEVATIONS.

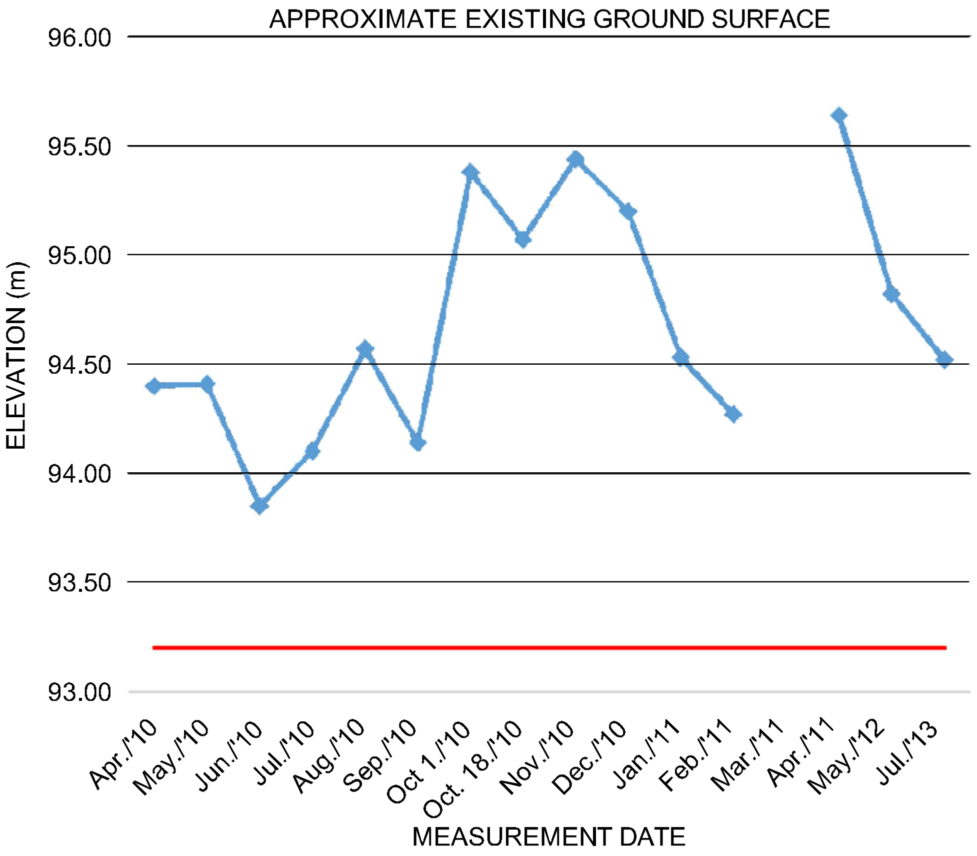
Golder Associates
Ottawa, Ontario, Canada

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-4

FIGURE 5D



LEGEND

◆	MW10-6
🟥	POND 2 PERMANENT POOL

NOTE

1. CONCEPTUAL USF ELEVATIONS PROVIDED BY DSEL
2. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

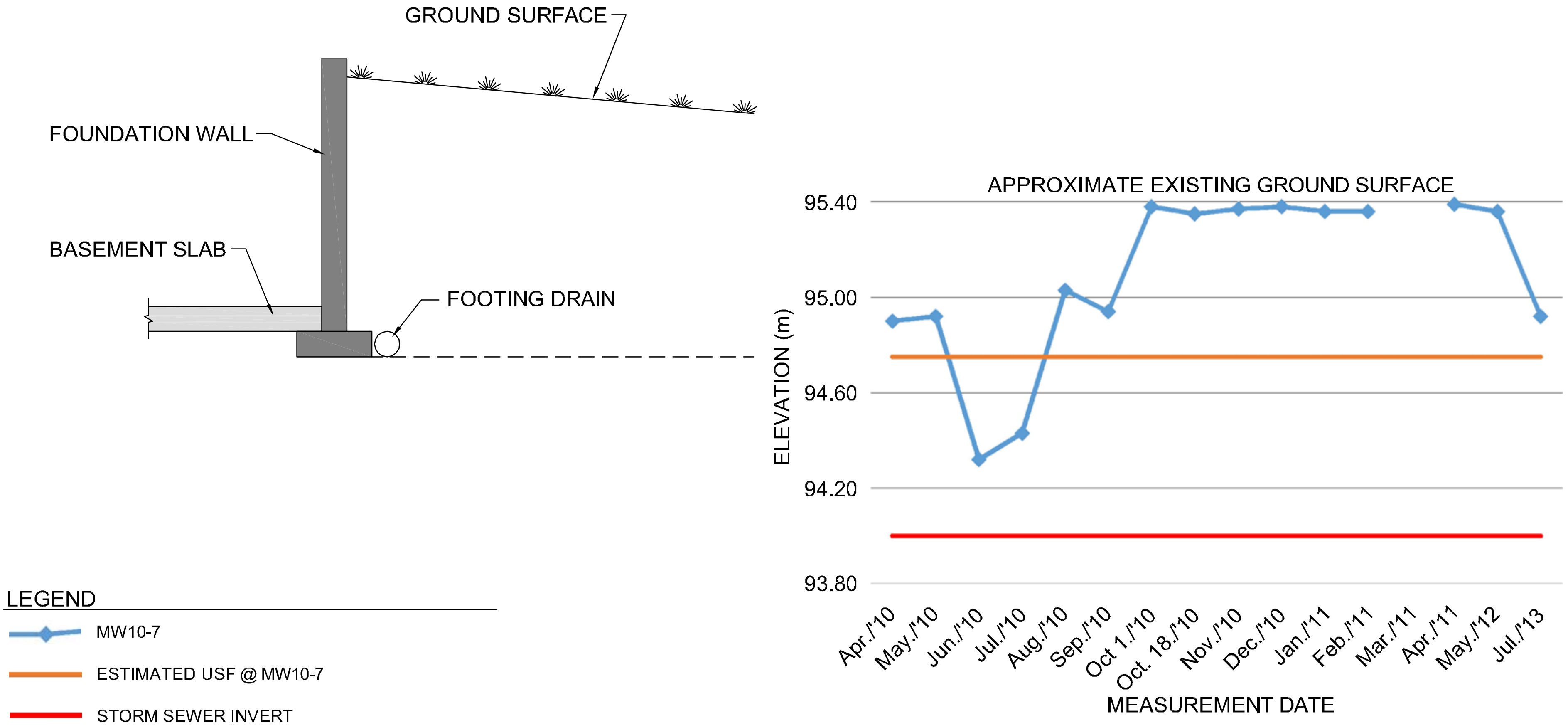
Golder Associates
Ottawa, Ontario, Canada

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-6

FIGURE 5E



NOTE

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2. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

3. IT IS RECOMMENDED THAT AREA SPECIFIC USF RAISES IN EXCESS OF THE GENERIC USF ELEVATION SHOWN BE REVIEWED AT THE DETAILED DESIGN STAGE TO BRING USF ELEVATIONS IN LINE WITH ANY FUTURE MONITORED GROUNDWATER ELEVATIONS.

Golder Associates
Ottawa, Ontario, Canada

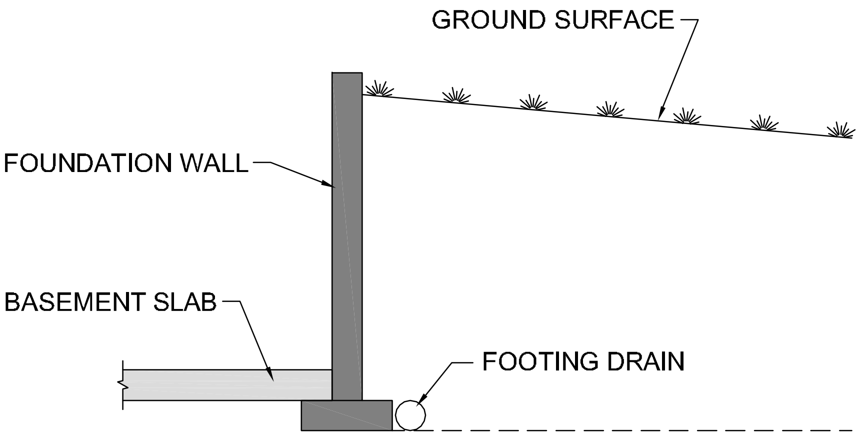
RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

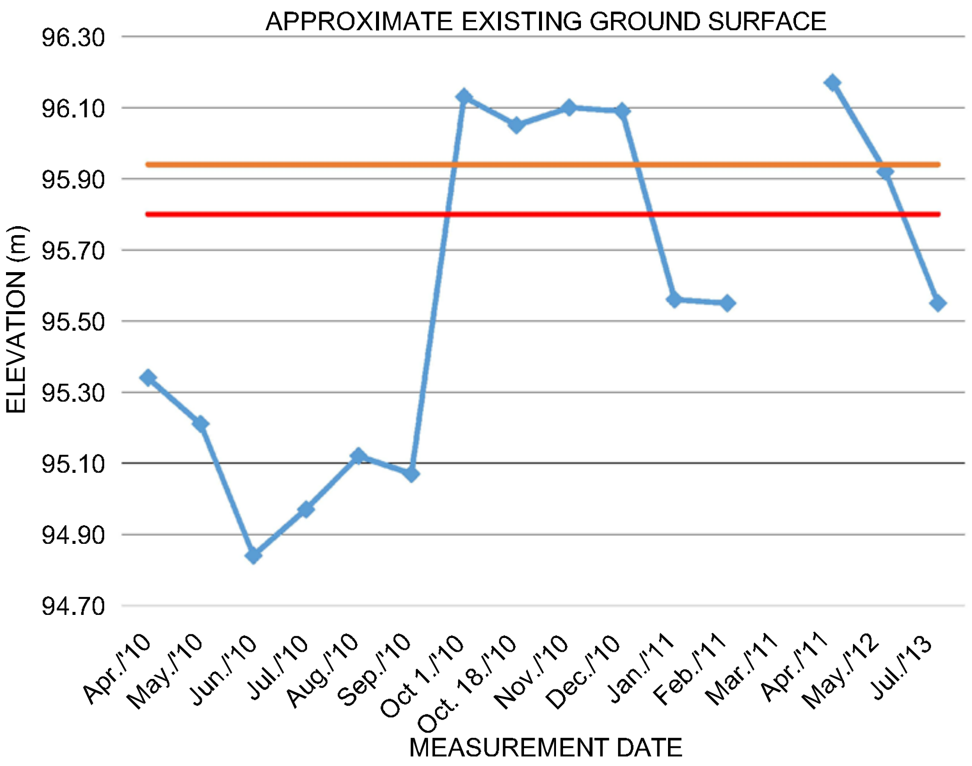
GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-7

FIGURE 5F

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

PLOT DATE: November 1, 2013
FILENAME: N:\Active\2012\1127 – Geosciences\12-1127-0062 Richmond Village Drainage and Sump Design\Spatial_IM\CAD\Phase 8000\1211270062-8000-05.dwg





LEGEND

◆	MW10-8
🟧	ESTIMATED USF @ MW10-8
🟥	STORM SEWER INVERT

NOTE

1. THESE GROUNDWATER LEVELS ARE IN BEDROCK
2. NO OVERBURDEN GROUNDWATER LEVELS WERE MEASURED
3. CONCEPTUAL USF ELEVATIONS PROVIDED BY DSEL
4. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1127-0062-8000

Golder Associates
Ottawa, Ontario, Canada

PROJECT No. 12-1127-0062 DATE 2013-10-16 CAD JM

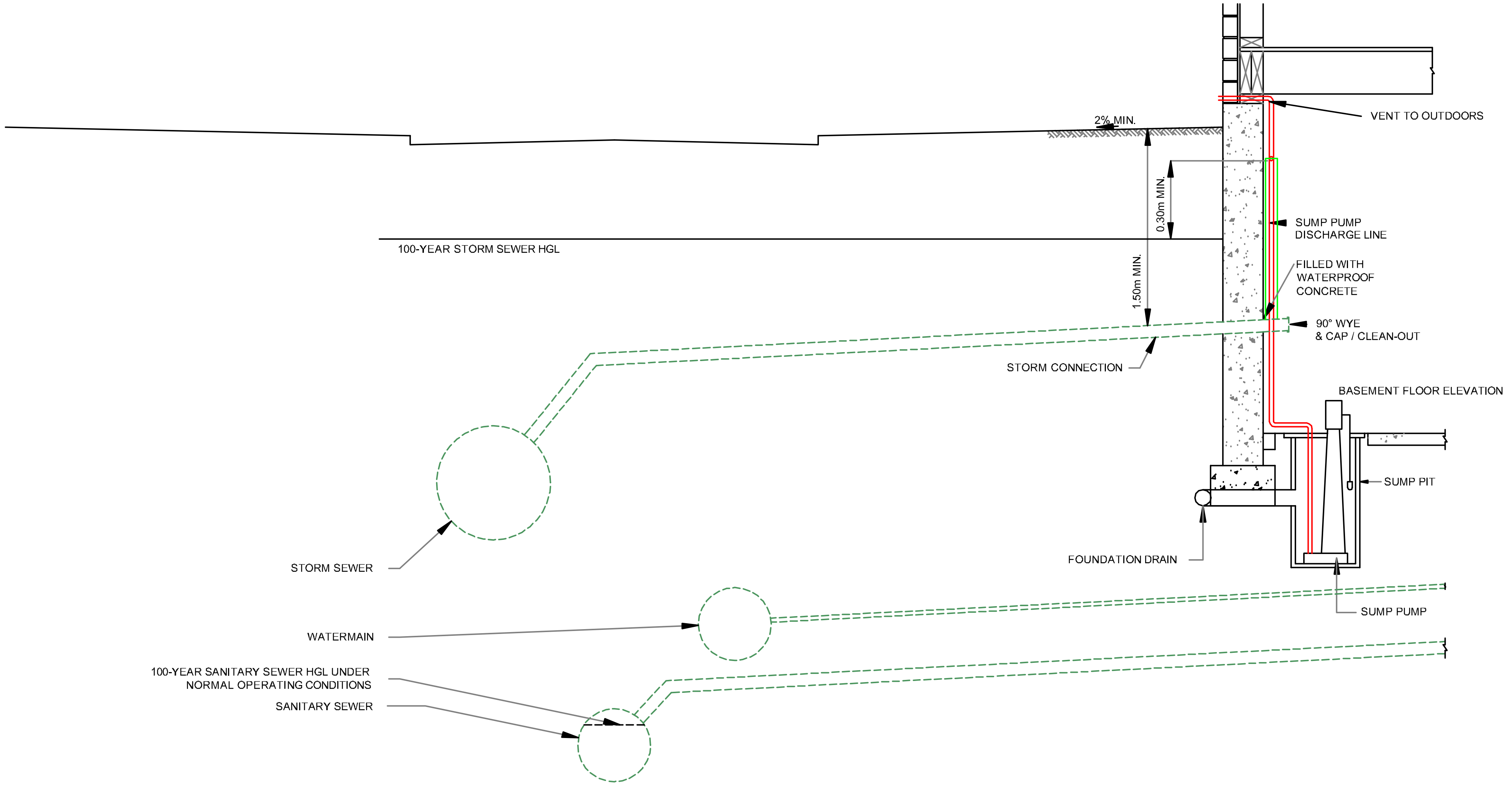
RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

GROUNDWATER VS. PROPOSED USF IN THE VICINITY OF MONITROING WELL MW10-8

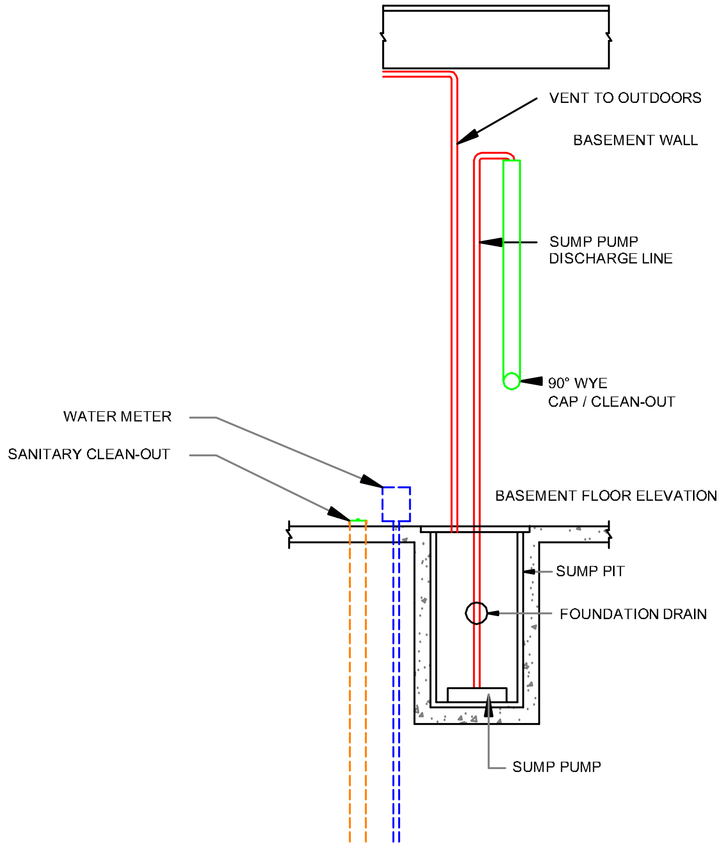
FIGURE 5G

APPENDIX A

Sump Pump Detail



SECTION VIEW



FRONT VIEW

DSEL
david schaeffer engineering ltd

120 Iber Road, Unit 203
Stittsville, Ontario, K2S 1E9
Tel. (613) 836-0856
Fax. (613) 836-7183
www.DSEL.ca

MATTAMY - VILLAGE OF RICHMOND
WESTERN DEVELOPMENT LANDS

SUMP PUMP DETAIL

CITY OF OTTAWA

PROJECT No.: 13-654
DATE: MARCH 2013
DRAWN BY: KM / ADF
SCALE: N.T.S.
FIGURE: 6

APPENDIX B

Sump Pump Data Sheet

HYDROMATIC®

FG-100A BATTERY BACK-UP SUMP PUMP

Protects Home
If AC-powered pump fails

Backs Up Residential Sump Pump
Automatically during power outage or pump failure (primary pump and piping not included)

Reliable Operation

Self-Charging System
Continuous

Solid-State Controller
Monitors battery charge and pump operation

Audible Alarm
Sounds when standby pump activates

Controller
Includes connectors for pump, charger, float switch and battery*

Area Light
Illuminates dark, powerless work area

Works with Flooded and Sealed AGM Deep Cycle Lead-Acid Battery
Can use two batteries for extended protection

Resettable Circuit Breaker
Eliminates the need for a fuse

2A 5-Stage Charger
Maintains 90% charge

* Deep cycle marine-type battery (Group 24) recommended – not included with FG-100 system

Catalog Number	Volts	Hz	Cord Length	Aprox. Wt. Lbs.
FG-100A	12	60	8'	14.75

System includes: *Pump, dual check valves, 1-1/4 x 1-1/2 inch adapter tee, battery box and control panel (battery not included).*

Pentair Water™

HYDROMATIC SUPPORTS THE PROFESSIONAL

FG-100A BATTERY BACK-UP SUMP PUMP

Features

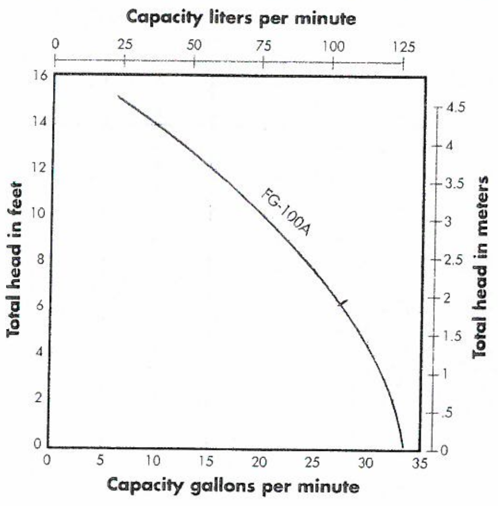
The Hydromatic® FG-100A battery operated back-up sump pump is designed to back up primary residential sump pumps in case of pump or power failure. A must for those installations where an inoperative sump pump cannot be tolerated. Separate float switch and built-in alarm automatically start back-up system and activates warning buzzer to protect against high water damage and warn of primary pump failure.

Details

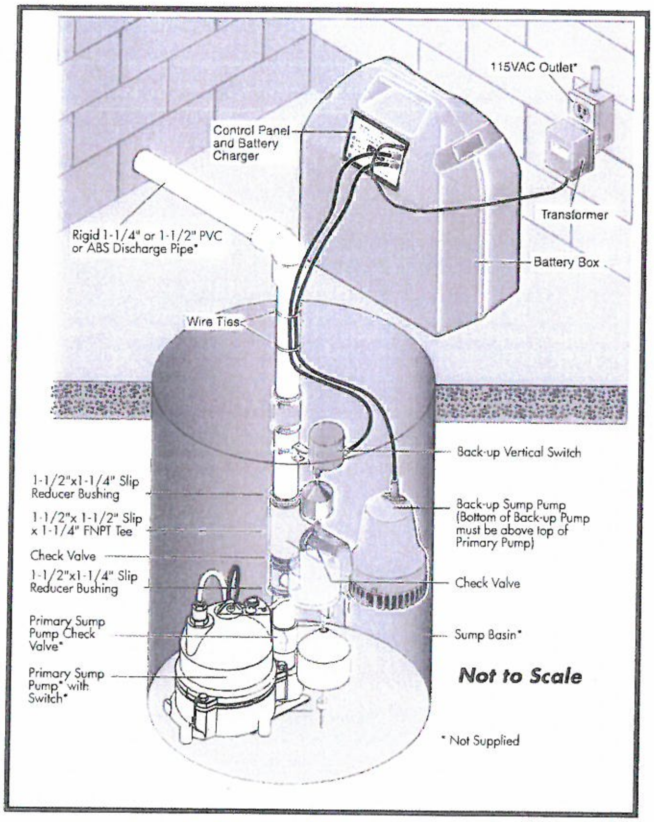
Specifications

- Housing – Thermoplastic
- Intermittent Liquid Temperature – Up to 77°F (25°C)
- Pump Down Range – Variable with float setting
- Pump Discharge – 1.25" NPT (31.75 mm)
- Motor – 12V, DC, 9A at 10 ft. (3M) lift
- Impeller – Thermoplastic
- Power Cord – 8 ft. SJT, 16/2 GA
- Capacities – Up to 30 GPM (114 LPM)
- Heads – To 16 ft. (4.88 m)
- Battery Charger – Input 120V, 60 Hz; Output 12V, 2A; Plug-in type wall-mounted transformer
- Check Valve – Dual check valves
- Battery Requirement* – 12V, deep cycle, marine type (Group 24), minimum 75–120 AH

Performance Data



Typical Installation



* Deep cycle marine-type battery (Group 24) recommended – not included with FG-100 system

HYDROMATIC®
Pentair Water

USA
740 East 9th Street, Ashland, Ohio 44805
Tel: 1-888-957-8677 Fax: 419-281-4087

www.hydromatic.com

CANADA
269 Trillium Drive, Kitchener, Ontario, Canada N2G 4W5
Tel: 519-896-2163 Fax: 519-896-6337

– Your Authorized Local Distributor –

APPENDIX C

DSEL Memorandum Dated October 30, 2013

david schaeffer engineering ltd
SMART SUBDIVISIONS™

120 Iber Road
Unit 203
Stittsville, Ontario K2S 1E9
Tel (613) 836-0856
Fax (613) 836-7183
www.DSEL.ca

October 30, 2013
DSEL File: 10-468

Attention: Note to File

Re: Richmond Village – Western Development Lands – Sump Pump Back-Up Evaluation

---

Through review of development options in the Western Development Lands, in the Village of Richmond, it has been concluded that the provision of sump pumps will be required. The sump pumps and weeping tile system will collect groundwater inflows during those times of the year when the groundwater level rises above the footing level and pump to the storm sewer system. Additional rationale is provided in Golder Associates' ***Technical Memorandum 12-1127-0062(8000).*** The proposed use of sump pumps is similar to the majority of the existing homes in Richmond.

In addition to the typical sump pump configuration, a back-up sump pump option (with standby power) is also proposed. In the event of power outages, the back-up configuration will provide an additional level of protection. After review of various guidelines it is appears that this is not a mandatory requirement of the City of Ottawa.

With the chosen back-up pump model (Hydromatic FG-100A or equivalent), the manufacturer recommends a marine-type battery with a minimum of 75 "amp-hours (AH)" of service. The AH rating is based upon testing over a standard 20 hour period to establish how long it takes the battery to drain. A 75AH battery indicates that a 3.75A load was applied over 20 hours to drain the battery (i.e. 3.75A x 20hr = 75AH). Note: Higher amp loading than the 3.75A tested will equate to less amperage hours available[1]. Assumed to be 10% efficiency loss in this case (i.e. lowers to 90% of original rating).

For a battery with 75AH of service, the pump (9 amp for this model) would have the ability to provide pumping for 7.5 hours (75AH/9A x 0.90 = 7.5 hours). At the pump rating of 100 L/min this would be equivalent to 100 L/min x 7.5hr x 60min = 45,000 L (45 m^3) of outflow. If predicted inflows to the weeping tile system (per the Golder technical memo) are in the order of 2.0m^3/day this equates to approximately 22 days of backup pumping. The back-up power would provide sufficient pumping even in extended periods of power outages.

David Schaeffer Engineering Ltd.

Per: Kevin L. Murphy, P.Eng.

---

[1] Peukert's law expresses the **capacity** of a **lead–acid battery** in terms of the rate at which it is discharged. As the rate increases, the battery's available capacity decreases.

Golder Associates

TECHNICAL MEMORANDUM

DATE October 23, 2013 **PROJECT No.** 12-1127-0062

TO Frank Cairo
Richmond Village (South) Limited

CC Mike Green - Mattamy (Jock River) Limited

FROM Brian Byerley, P. Eng. **EMAIL** Brian_Byerley@golder.com

STORM WATER MANAGEMENT POND 2 DESIGN RECOMMENDATIONS, WESTERN DEVELOPMENT LANDS, VILLAGE OF RICHMOND DEVELOPMENT, OTTAWA, ONTARIO

Richmond Village (South) Limited (RVSL) has requested Golder Associates Ltd. (Golder) to provide preliminary design recommendations concerning the proposed depth of Storm Water Management (SWM) Pond 2 in the Western Development Lands (the site). The current preliminary design of SWM Pond 2 (a shallow wetland pond) has been developed by David Schaeffer Engineering Ltd. (DSEL), based on the required function of the pond and the proposed lay-out of the site infrastructure.

Subsurface conditions in the vicinity of SWM Pond 2 were previously investigated by Golder in 2010 by completion of borehole BH10-06. The subsurface conditions at BH10-6 consist of 0.25 metres of topsoil, over approximately 1.58 metres of sandy silt to silty sand, over approximately 1.24 of glacial till. Dolostone bedrock was encountered directly beneath the glacial till, at approximately 3.07 metres below ground surface (mbgs). Two monitoring wells were installed in BH10-6: MW10-6A (in bedrock) and MW10-6B (in overburden). See the attached borehole record for details regarding subsurface conditions and monitoring well construction.

Groundwater level monitoring between 2010 and 2013 has indicated that the groundwater levels at MW10-6A (in bedrock) ranged from 0.43 mbgs (in spring) to 1.96 mbgs (in summer). Slug testing at MW10-6A determined that the horizontal hydraulic conductivity of the shallow bedrock to be approximately 5x10-6 m/s.

Based on the available information, Golder recommends that SWM Pond 2 be designed to be as shallow as possible, with no or minimal excavation into rock for the following reasons:

- The hydraulic head in the bedrock would prevent the construction of a pond liner, if required;
- Although the bedrock horizontal hydraulic conductivity measured at MW10-6A is similar to the hydraulic conductivity of the overlying silty sand and silty clay deposits, higher hydraulic conductivity has been measured in the bedrock elsewhere on-site. Therefore, the design of the pond should consider the possibility that vertical fractures in the shallow bedrock might permit increased groundwater flow through the bottom of the pond, which could be problematic in terms of the constructability of the pond. The outlet sewer would also need to accommodate additional flow;

- If SWM Pond 2 was excavated into bedrock, possible high rates of groundwater flow into the pond could prevent draining of the pond for regular maintenance; and,
- Lots adjacent to the development lands are serviced by individual water supply wells (typically 30 metres deep), which obtain water from the bedrock. Minimising the amount of rock excavation will also minimise the possibility of impacts to those wells.

We trust that this memorandum is sufficient for your current needs. Please contact the undersigned if you have any questions.

Yours Truly

GOLDER ASSOCIATES LTD.

Mike Cunningham, P.Eng.
Senior Geotechnical Engineer/Associate

Brian Byerley, M.Sc., P.Eng.
Senior Hydrogeologist/Principal

LICENSED PROFESSIONAL ENGINEER
B. T. BYERLEY
23/10/13
PROVINCE OF ONTARIO

MIC/BTB/

Attachments: Important Information and Limitations of this Report
Borehole Record for BH10-6

n:\active\2012\1127 - geosciences\12-1127-0062 richmond village drainage and sump design\reporting\1211270062 - tech memo - swmp 2 - 22oct2013.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, Richmond Village (South) Limited and Mattamy (Jock River) Limited. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

PROJECT: 08-1122-0078

RECORD OF BOREHOLE: 10-6

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: Apr. 30, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m
		GROUND SURFACE		95.67			
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL		0.00			
		Loose to compact grey brown SANDY SILT to SILTY SAND, trace clay		95.42 0.25			
1					1	50 DO	6
2		Dense to very dense grey brown SANDY SILT, some gravel, cobbles and boulders (GLACIAL TILL)		93.84 1.83	2	50 DO	27
					3	50 DO	68
3		Thinly to medium bedded light grey intebedded SANDSTONE and DOLOSTONE BEDROCK		92.60 3.07	4	50 DO	>100
4	Rotary Drill NQ Core				C1	NQ RC	DD
5					C2	NQ RC	DD
		End of Borehole		90.49 5.18			
6							
7							
8							
9							
10							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

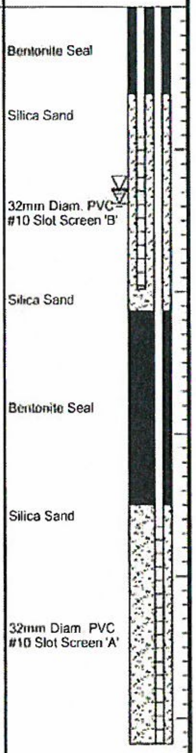
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. in screen 'A' at Elev. 94.29m on Apr. 30, 2010

W.L. in screen 'B' at Elev. 93.38m on Apr. 30, 2010

MIS-BHS 001 0811220078-9500.GPJ GAL-MIS.GDT 6/3/10 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.D.

CHECKED:

PROJECT: 08-1122-0078

RECORD OF DRILLHOLE: 10-6

SHEET 2 OF 2

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Apr. 30, 2010

DRILL RIG: CME 55

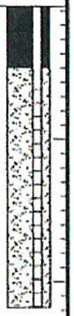
DRILLING CONTRACTOR: Marathon Drilling

DATUM:

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		92.60		
	Rotary Drill NQ Core	Thinly to medium bedded light grey intebedded SANDSTONE and DOLOSTONE BEDROCK		3.07		Bentonite Seal Silica Sand
4					1	32mm Diam. PVC #10 Slot Screen 'A'
5				90.49 5.18		
		End of Borehole				W.L. in screen 'A' at Elev. 94.29m on Apr. 30, 2010
6						W.L. in screen 'B' at Elev. 93.38m on Apr. 30, 2010
7						
8						
9						
10						
11						
12						
13						

Column headings: PENETRATION RATE (m/min) | COLOUR % RETURN | FLUSH | RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.3 (5 10 15 20) | DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION | HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}) | DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.D.

CHECKED:

HYDROMATIC® CSS-3D AND CSS-3V SUMP BASIN PACKAGE

Preplumbed Sump Basin Package

- Basement sumps, dewatering, new or existing installations
- Fully Assembled Check valve included.
- Injection-Molded Structural Foam Provides the highest quality construction for uniform wall thickness and strength.
- Proprietary "Flush Mount" Design for easy shipping and storage. Allows for pallet stacking.

The Hydromatic Preplumbed Sump Basin Package is a rugged, high quality product consisting of our D-A1 or V-A1 .3 HP Cast Iron sump pump, ideal for drain water removal. The 18" x 22" basin is constructed of the highest quality structural foam and has a 20 gallon capacity. The package comes with a single-piece structural foam lid, proprietary "Flush Mount" design for easy shipment and storage. The unit comes field-ready with 1-1/2" NPT threaded discharge flange and a 2" NPT vent flange (adaptable to 3") and includes a 1-1/2" sump check valve.

PENTAIR

WWW.HYDROMATIC.COM

HYDROMATIC® CSS-3D AND CSS-3V
SUMP BASIN PACKAGE

Specifications

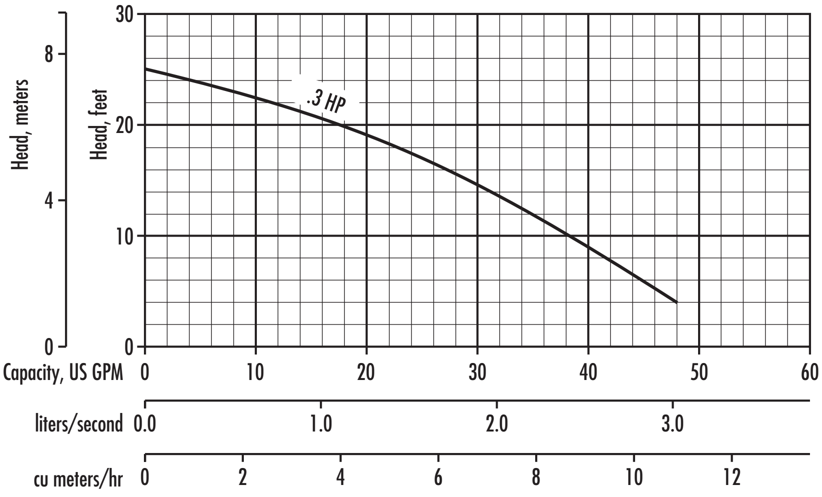
Pump (D-A1 or V-A1)

- Housing – Cast iron.
- Motor – .3 HP, 115 volt, single phase, shaded pole motor, built-in overload protection.
- Maximum Limits – Liquid temperature: 120°F (49°C).

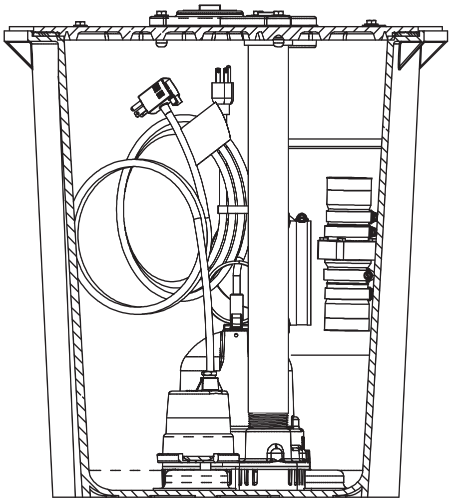
Basin, Lid, and Accessories

- Radon Proof – Radon approved basin and cover for radon prevention.
- Basin – Injection-molded structural foam (polyethylene). 20-gallon capacity. Standard 18" x 22" basin.
- Lid – Injection-molded structural foam (polyethylene). Gas-tight single-piece design with seal.
- Hub Adapter – 4" Snap-in type with stainless steel clamp. Fits 4" DWV SCH. 40 plastic pipe.
- Flanges – 1-1/2" NPT threaded discharge and 2" NPT vent flange (easily converts to 3"). Comes with gas-tight cord grommet.
- Check Valve – Sump pump check valve fits 1-1/2" black, galvanized, and DWV SCH. 40 pipe.

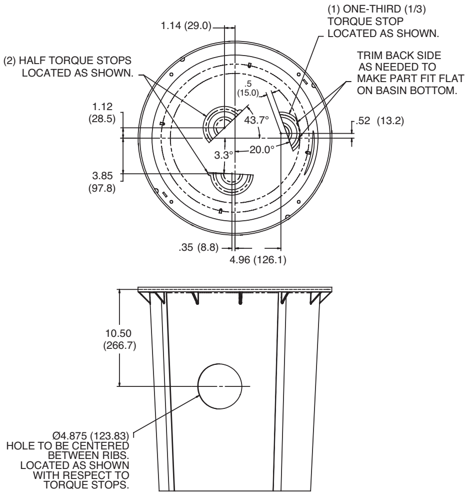
Performance Curve



Sectional View



Outline Dimensions



Ordering Information

Catalog Number	HP	Package Type	Full Load Amps	Cord Length	Mechanical Switch Type	Basin Size
CSS-3D	.3	Preassembled	12	10'	Diaphragm	18" x 22"
CSS-3V	.3	Preassembled	12	10'	Vertical	18" x 22"

PENTAIR

USA
293 WRIGHT STREET, DELAVAN, WI 53115 WWW.HYDROMATIC.COM
PH: 888-957-8677 ORDERS FAX: 800-426-9446

CANADA
269 TRILLIUM DRIVE, KITCHENER, ONTARIO, CANADA N2G 4W5
PH: 519-896-2163 ORDERS FAX: 519-896-6337

HYDROMATIC®

FG-100A BATTERY BACK-UP SUMP PUMP

Protects Home
If AC-powered pump fails

Backs Up Residential Sump Pump
Automatically during power outage or pump failure (primary pump and piping not included)

Reliable Operation

Self-Charging System
Continuous

Solid-State Controller
Monitors battery charge and pump operation

Audible Alarm
Sounds when standby pump activates

Controller
Includes connectors for pump, charger, float switch and battery*

Area Light
Illuminates dark, powerless work area

Works with Flooded and Sealed AGM Deep Cycle Lead-Acid Battery
Can use two batteries for extended protection

Resettable Circuit Breaker
Eliminates the need for a fuse

2A 5-Stage Charger
Maintains 90% charge

* Deep cycle marine-type battery (Group 24) recommended – not included with FG-100 system

Catalog Number	Volts	Hz	Cord Length	Aprox. Wt. Lbs.
FG-100A	12	60	8'	14.75

***System includes:** Pump, dual check valves, 1-1/4 x 1-1/2 inch adapter tee, battery box and control panel (battery not included).*

Pentair Water™

HYDROMATIC SUPPORTS THE PROFESSIONAL

FG-100A BATTERY BACK-UP SUMP PUMP

Features

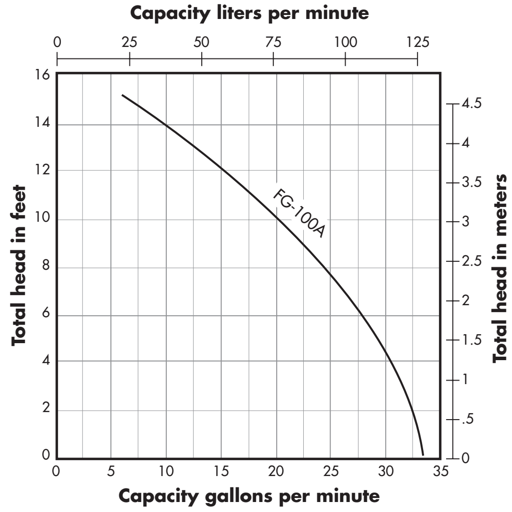
The Hydromatic® FG-100A battery operated back-up sump pump is designed to back up primary residential sump pumps in case of pump or power failure. A must for those installations where an inoperative sump pump cannot be tolerated. Separate float switch and built-in alarm automatically start back-up system and activates warning buzzer to protect against high water damage and warn of primary pump failure.

Details

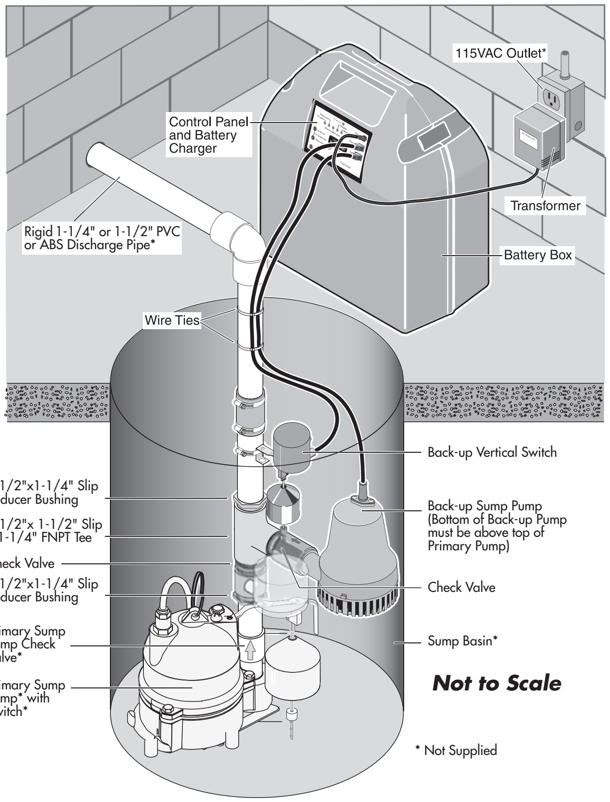
Specifications

- Housing – Thermoplastic
- Intermittent Liquid Temperature – Up to 77°F (25°C)
- Pump Down Range – Variable with float setting
- Pump Discharge – 1.25" NPT (31.75 mm)
- Motor – 12V, DC, 9A at 10 ft. (3M) lift
- Impeller – Thermoplastic
- Power Cord – 8 ft. SJT, 16/2 GA
- Capacities – Up to 30 GPM (114 LPM)
- Heads – To 16 ft. (4.88 m)
- Battery Charger – Input 120V, 60 Hz; Output 12V, 2A; Plug-in type wall-mounted transformer
- Check Valve – Dual check valves
- Battery Requirement* – 12V, deep cycle, marine type (Group 24), minimum 75–120 AH

Performance Data



Typical Installation



* Deep cycle marine-type battery (Group 24) recommended – not included with FG-100 system

HYDROMATIC®
Pentair Water

– Your Authorized Local Distributor –

USA
740 East 9th Street, Ashland, Ohio 44805
Tel: 1-888-957-8677 Fax: 419-281-4087

www.hydromatic.com

CANADA
269 Trillium Drive, Kitchener, Ontario, Canada N2G 4W5
Tel: 519-896-2163 Fax: 519-896-6337

Item #: W-02-7740 7/08

APPENDIX G

Evaluation of Alternatives

Richmond Village (South) Limited
Option 1

Stage 1 Estimate
Cost Summary

			Unit	Approx. Qty.	Unit Rate	Estimated Value
A	**Site Preparation**					
	A.1	Strip and Stockpile Top Soil	m^3	381,830	$ 6.00	$ 2,291,000.00
	A.2	Site Grading - Cut to Fill	m^3	156,660	$ 6.00	$ 940,000.00
	A.3	Import Fill	m^3	1,390,491	$ 18.00	$ 25,029,000.00
	A.4	Re-use top soil as fill	m^3	229,098	$ 6.00	$ 1,375,000.00
	A.5	Engineered fill required	m^3	286,910	$ 25.00	$ 7,173,000.00
	A.6	Estimated Total Surcharge Fill	m^3	720,000	$ 18.00	$ 12,960,000.00
	A.7	Move surcharge fill	m^3	1,080,000	$ 6.00	$ 6,480,000.00
	A.8	Export Surcharge Material	m^3	540,000	$ 12.00	$ 6,480,000.00
					Sub-Total Subsection A	$ 62,728,000.00
B	**Stormwater Conveyance**					
	B.1	Local Storm Sewer	m	8,890.0	$ 445.00	$ 3,956,000.00
	B.2	Trunk Storm Sewers	m	9,225.0	$ 900.00	$ 8,303,000.00
					Sub-Total Subsection B	$ 12,259,000.00
					Estimated Capital Cost	**$ 74,987,000.00**
C	**Estimated Annual Maintenance Cost**					
	C.1	Local Storm Sewer - Flushing	m	8,890.0	$ 5.00	$ 44,000.00
	C.2	Trunk Storm Sewer - Flushing	m	9,225.0	$ 6.00	$ 55,000.00
					Estimated Life Cycle Cost	**$ 99,000.00**

Notes:
1) Import Fill required is net fill required less re-use top soil as fill.
2) Re-use top soil as fill assumes 60% of stripped and stock piled top soil is available for fill material.

Richmond Village (South) Limited
Option 2

Stage 1 Estimate
Cost Summary

			Unit	Approx. Qty.	Unit Rate	Estimated Value
A	**Site Preparation**					
	A.1	Strip and Stockpile Top Soil	m^3	381,830	$ 6.00	$ 2,291,000.00
	A.2	Site Grading - Cut to Fill	m^3	206,305	$ 3.00	$ 619,000.00
	A.3	Import Fill	m^3	184,455	$ 18.00	$ 3,320,000.00
	A.4	Re-use top soil as fill	m^3	229,098	$ 6.00	$ 1,375,000.00
	A.5	Engineered fill required	m^3		$ 25.00	$ -
	A.6	Estimated Total Surcharge Fill	m^3		$ 24.00	$ -
	A.7	Export Surcharge Material	m^3		$ 12.00	$ -
					Sub-Total Subsection A	$ 7,605,000.00
B	**Stormwater Conveyance**					
	B.1	Local Storm Sewer	m	8,890.0	$ 445.00	$ 3,956,000.00
	B.2	Trunk Storm Sewers	m	9,225.0	$ 900.00	$ 8,303,000.00
	B.3	Foundation Collector	m	17,850.0	$ 300.00	$ 5,355,000.00
	B.4	Lift Station	ea	1.0	$ 5,000,000.00	$ 5,000,000.00
					Sub-Total Subsection B	$ 22,614,000.00
					Estimated Capital Cost	**$ 30,219,000.00**
C	**Estimated Annual Maintenance Cost**					
	C.1	Local Storm Sewer - Flushing	m	8,890.0	$ 5.00	$ 44,000.00
	C.2	Trunk Storm Sewer - Flushing	m	9,225.0	$ 6.00	$ 55,000.00
	C.3	Foundation Collector - Flushing	m	17,850.0	$ 5.00	$ 89,000.00
	C.4	Lift Station Operation and Maintenance	m	1.0	$ 100,000.00	$ 100,000.00
					Estimated Life Cycle Cost	**$ 288,000.00**

Notes:

1) Import Fill required is net fill required less re-use top soil as fill.
2) Re-use top soil as fill assumes 60% of stripped and stock piled top soil is available for fill material.

Richmond Village (South) Limited
Option 3

Stage 1 Estimate
Cost Summary

			Unit	Approx. Qty.	Unit Rate	Estimated Value
A	**Site Preparation**					
	A.1	Strip and Stockpile Top Soil	m^3	381,830	$ 6.00	$ 2,291,000.00
	A.2	Site Grading - Cut to Fill	m^3	175,550	$ 3.00	$ 527,000.00
	A.3	Import Fill	m^3	145,395	$ 18.00	$ 2,617,000.00
	A.4	Re-use top soil as fill	m^3	229,098	$ 6.00	$ 1,375,000.00
	A.5	Engineered fill required	m^3	418,398	$ 25.00	$ 10,460,000.00
	A.6	Estimated Total Surcharge Fill	m^3	-	$ 24.00	$ -
	A.7	Export Surcharge Material	m^3	-	$ 12.00	$ -
					Sub-Total Subsection A	$ 17,270,000.00
B	**Stormwater Conveyance**					
	B.1	Local Storm Sewer	m	8,890.0	$ 445.00	$ 3,956,000.00
	B.2	Trunk Storm Sewers	m	9,225.0	$ 900.00	$ 8,303,000.00
					Sub-Total Subsection B	$ 12,259,000.00
					Estimated Capital Cost	**$ 29,529,000.00**
C	**Estimated Annual Maintenance Cost**					
	C.1	Local Storm Sewer - Flushing	m	8,890.0	$ 5.00	$ 44,000.00
	C.2	Trunk Storm Sewer - Flushing	m	9,225.0	$ 6.00	$ 55,000.00
					Estimated Life Cycle Cost	**$ 99,000.00**

Notes:

1) Import Fill required is net fill required less re-use top soil as fill.
2) Re-use top soil as fill assumes 60% of stripped and stock piled top soil is available for fill material.
3) Slab on grade units require engineered fill (Granular materials) when FFE is above existing grade. IE where pregrade is 1.0m
4) Approximately 186m3 of fill required for units with USF's above existing grade

Richmond Village (South) Limited
Option 4

Stage 1 Estimate
Cost Summary

		Unit	Approx. Qty.	Unit Rate	Estimated Value
A	**Site Preparation**				
A.1	Strip and Stockpile Top Soil	m^3	381,830	$ 6.00	$ 2,291,000.00
A.2	Site Grading - Cut to Fill	m^3	198,374	$ 3.00	$ 595,000.00
A.3	Import Fill	m^3	442,159	$ 18.00	$ 7,959,000.00
A.4	Re-use top soil as fill	m^3	229,098	$ 6.00	$ 1,375,000.00
A.5	Engineered fill required	m^3		$ 25.00	$ -
A.6	Estimated Total Surcharge Fill	m^3		$ 24.00	$ -
A.7	Export Surcharge Material	m^3		$ 12.00	$ -
				Sub-Total Subsection A	$ 12,220,000.00
B	**Stormwater Conveyance**				
B.1	Local Storm Sewer	m	8,890.0	$ 445.00	$ 3,956,000.00
B.2	Trunk Storm Sewers	m	9,225.0	$ 900.00	$ 8,303,000.00
B.3	Sump pump assembly	ea	2,300.0	$ 916.00	$ 2,107,000.00
				Sub-Total Subsection B	$ 14,366,000.00
				Estimated Capital Cost	**$ 26,586,000.00**
C	**Estimated Annual Maintenance Cost**				
C.1	Local Storm Sewer - Flushing	m	8,890.0	$ 5.00	$ 44,000.00
C.2	Trunk Storm Sewer - Flushing	m	9,225.0	$ 6.00	$ 55,000.00
				Estimated Life Cycle Cost	**$ 99,000.00**

Notes:

1) Import Fill required is net fill required less re-use top soil as fill.
2) Re-use top soil as fill assumes 60% of stripped and stock piled top soil is available for fill material.
3) Sump pump assembly per list price for the following:
    - $436 - Hydromatic CSS-3D Sump basin package
    - $360 - Hydromatic G-100A Battery backup sump pump
    - $120 - Deep cycle marine-type battery (Group 24)

APPENDIX H

Rational Method Calculation Sheet

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

Manning 0.013 Return Frequency = 5 years

	LOCATION		AREA (Ha)							FLOW					SEWER DATA								
Location	From Node	To Node	R= 0.25	R= 0.60	R= 0.62	R= 0.70	R= 0.75	R= 0.80	R= 0.90	Indiv. 2.78 AC	Accum. 2.78 AC	Time of Conc.	Rainfall Intensity	Peak Flow Q (l/s)	DIA. (mm) (actual)	DIA. (mm) (nominal)	TYPE	SLOPE (%)	LENGTH (m)	CAPACITY (l/s)	VELOCITY (m/s)	TIME OF FLOW (min.)	RATIO Q/Q full
TRUNK 1																							
	101	102			1.43					2.46	2.46	10.00	104.19	257	825	825	CONC	0.10	67.0	454	0.85	1.32	0.57
	102	103								0.00	2.46	11.32	97.72	241	825	825	CONC	0.10	80.0	454	0.85	1.57	0.53
	103	104			1.06					1.83	4.29	12.89	91.08	391	975	975	CONC	0.10	60.5	708	0.95	1.06	0.55
	104	105								0.00	4.29	13.95	87.12	374	975	975	CONC	0.10	23.5	708	0.95	0.41	0.53
	105	106			0.78					1.34	5.64	14.36	85.68	483	1050	1050	CONC	0.10	101.5	863	1.00	1.70	0.56
	106	107			0.59					1.02	6.65	16.06	80.29	534	1050	1050	CONC	0.10	76.5	863	1.00	1.28	0.62
	107	108			1.33					2.29	8.95	17.33	76.70	686	1200	1200	CONC	0.10	74.5	1232	1.09	1.14	0.56
	108	109			0.67					1.15	10.10	18.47	73.79	745	1200	1200	CONC	0.10	26.5	1232	1.09	0.41	0.60
	109	HW								0.00	10.10	18.88	72.81	735	1200	1200	CONC	0.10	31.0	1232	1.09	0.47	0.60
To SWMP											10.10	19.35											
TRUNK 25																							
	251	252			0.86					1.48	1.48	10.00	104.19	154	675	675	CONC	0.10	70.5	266	0.74	1.58	0.58
	252	253			0.42					0.72	2.21	11.58	96.52	213	750	750	CONC	0.10	70.5	352	0.80	1.47	0.61
	253	254			2.80					4.83	7.03	13.06	90.41	636	1050	1050	CONC	0.10	76.5	863	1.00	1.28	0.74
					3.17					5.46	12.50	14.33	85.77	1072									
	254	206			0.64					1.10	13.60	14.33	85.77	1166	1200X1800	1200X1800	CONC	0.10	88.0	2636	1.27	1.16	0.44
To Trunk 2, Pipe 206 - 207											13.60	15.49											
TRUNK 26																							
	261	262			1.06					1.83	1.83	10.00	104.19	190	675	675	CONC	0.10	97.5	266	0.74	2.19	0.72
	262	263			0.99					1.71	3.53	12.19	93.90	332	825	825	CONC	0.10	119.0	454	0.85	2.34	0.73
	263	264			0.68					1.17	4.71	14.52	85.13	401	900	900	CONC	0.10	107.0	572	0.90	1.98	0.70
	264	265			0.60					1.03	5.74	16.51	78.99	453	900	900	CONC	0.10	40.5	572	0.90	0.75	0.79
	265	266			1.81					3.12	8.86	17.26	76.91	681	1050	1050	CONC	0.10	30.0	863	1.00	0.50	0.79
	266	267			2.68					4.62	13.48	17.76	75.59	1019	1200	1200	CONC	0.10	99.0	1232	1.09	1.51	0.83
	267	268			2.86					4.93	18.41	19.27	71.89	1323	1200x1800	1200x1800	CONC	0.10	45.5	2636	1.27	0.60	0.50
	268	211								0.00	18.41	19.87	70.54	1299	1200x1800	1200x1800	CONC	0.10	40.5	2636	1.27	0.53	0.49
To Trunk 2, Pipe 211-212											18.41	20.40											
PERTH STREET														**100 YR**									
External from South: 100-year, 3-hour Chicago Storm (JFSA)														2560									
	1500	1501												2560	1200x1800	1200x1800	CONC	0.10	120.0	2636	1.27	1.58	0.97
	1501	1502												2560	1200x1800	1200x1800	CONC	0.10	116.0	2636	1.27	1.53	0.97
	1502	HW												2560	1200x1800	1200x1800	CONC	0.10	80.0	2636	1.27	1.05	0.97
														2560									
TRUNK 3																							
	301	302			0.67					1.15	1.15	10.00	104.19	120	600	600	CONC	0.10	54.0	194	0.69	1.31	0.62
	302	303								0.00	1.15	11.31	97.75	113	600	600	CONC	0.10	11.0	194	0.69	0.27	0.58
	303	304								0.00	1.15	11.58	96.54	111	600	600	CONC	0.10	67.5	194	0.69	1.64	0.57
	304	305			0.51					0.88	2.03	13.22	89.80	183	675	675	CONC	0.10	73.5	266	0.74	1.65	0.69
	305	306			1.31					2.26	4.29	14.86	84.00	360	900	900	CONC	0.10	76.5	572	0.90	1.42	0.63
	306	307			1.48					2.55	6.84	16.28	79.63	545	900x1800	900x1800	CONC	0.10	79.5	1724	1.12	1.18	0.32
	307	308			1.99					3.43	10.27	17.47	76.36	784	900x1800	900x1800	CONC	0.10	77.0	1724	1.12	1.15	0.45
	308	407								0.00	10.27	18.61	73.45	755	900x1800	900x1800	CONC	0.10	94.5	1724	1.12	1.41	0.44
To ALN. 04 , Pipe 407 - 408											10.27	20.02											

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Notes:
1) Ottawa Rainfall-Intensity Curve
2) Min. Velocity = 0.76 m/sec

Designed: K.M.
Checked: Z.L.
Dwg. Reference: Storm Drainage Plan

PROJECT: VILLAGE OF RICHMOND
SWM - SUBMISSION 4
LOCATION: City of Ottawa
File Ref: 11-468
Date: November, 2013
Sheet No. 1 of 5

468_SWM_Stm_Designsheet_4-values.xls

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

Ottawa

Manning 0.013 Return Frequency = 5 years

	LOCATION		AREA (Ha)							FLOW					SEWER DATA								
			R=	R=	R=	R=	R=	R=	R=	Indiv.	Accum.	Time of	Rainfall	Peak Flow	DIA. (mm)	DIA. (mm)	TYPE	SLOPE	LENGTH	CAPACITY	VELOCITY	TIME OF	RATIO
Location	From Node	To Node	0.25	0.6	0.62	0.70	0.75	0.80	0.9	2.78 AC	2.78 AC	Conc.	Intensity	Q (l/s)	(actual)	(nominal)		(%)	(m)	(l/s)	(m/s)	FLOW (min.)	Q/Q full
TRUNK 2																							
	202	203			1.06					1.83	1.83	10.00	104.19	190	675	675	CONC	0.10	120.0	266	0.74	2.69	0.72
	203	204			1.10					1.90	3.72	12.69	91.84	342	825	825	CONC	0.10	77.0	454	0.85	1.51	0.75
	204	205			0.54					0.93	4.65	14.20	86.22	401	900	900	CONC	0.10	72.0	572	0.90	1.33	0.70
	205	206			0.52					0.90	5.55	15.54	81.86	454	975	975	CONC	0.10	73.0	708	0.95	1.28	0.64
Contribution From ALN. 25, Pipe 254 - 206										13.60	19.15	15.49											
					0.07					0.12	19.27												
	206	207			0.75					1.29	20.56	16.82	78.11	1606	1200x1800	1200x1800	CONC	0.10	34.0	2636	1.27	0.45	0.61
	207	208			0.37					0.64	21.20	17.27	76.89	1630	1200x1800	1200x1800	CONC	0.10	55.5	2636	1.27	0.73	0.62
	208	209								0.00	21.20	18.00	74.98	1590	1200x1800	1200x1800	CONC	0.10	39.5	2636	1.27	0.52	0.60
	209	210			2.01					3.46	24.66	18.52	73.69	1817	1200x1800	1200x1800	CONC	0.10	76.5	2636	1.27	1.01	0.69
	210	211			1.11					1.91	26.58	19.52	71.32	1896	1200x1800	1200x1800	CONC	0.10	76.5	2636	1.27	1.01	0.72
Contribution From ALN. 26, Pipe 268 - 211										18.41	44.99	20.40											
	211	212			0.60					1.03	46.02	20.53	69.11	3181	1200x2400	1200x2400	CONC	0.12	82.0	4158	1.48	0.92	0.77
	212	213			0.60					1.03	47.05	21.45	67.22	3163	1200x2400	1200x2400	CONC	0.12	65.5	4158	1.48	0.74	0.76
	213	214								0.00	47.05	22.18	65.80	3096	1200x2400	1200x2400	CONC	0.12	31.5	4158	1.48	0.35	0.74
	214	215								0.00	47.05	22.54	65.13	3065	1200x2400	1200x2400	CONC	0.12	34.5	4158	1.48	0.39	0.74
	215	2015								0.00	47.05	22.92	64.42	3031	1200x2400	1200x2400	CONC	0.12	42.5	4158	1.48	0.48	0.73
To SWMP											47.05	23.40											
TRUNK 4																							
	401	402			1.17					2.02	2.02	10.00	104.19	210	675	675	CONC	0.11	26.5	279	0.78	0.57	0.75
	402	403								0.00	2.02	10.57	101.29	204	750	750	CONC	0.10	11.0	352	0.80	0.23	0.58
	403	404								0.00	2.02	10.80	100.16	202	750	750	CONC	0.10	80.0	352	0.80	1.67	0.57
	404	405			0.91					1.57	3.59	12.47	92.73	332	825	825	CONC	0.10	106.5	454	0.85	2.09	0.73
	405	406			1.42					2.45	6.03	14.56	85.00	513	900	900	CONC	0.13	68.0	652	1.03	1.10	0.79
	406	407								0.00	6.03	15.67	81.47	491	900	900	CONC	0.13	66.5	652	1.03	1.08	0.75
Contribution From ALN. 03, Pipe 308 - 407										10.27	16.31	20.02											
	407	408								0.00	16.31	20.02	70.21	1145	900x1800	900x1800	CONC	0.10	17.5	1724	1.12	0.26	0.66
	408	4008								0.00	16.31	20.28	69.65	1136	900x1800	900x1800	CONC	0.10	16.5	1724	1.12	0.25	0.66
To SWMP											16.31	20.52											
TRUNK 5																							
	500	501			1.18					2.03	2.03	10.00	104.19	212	750	750	CONC	0.10	81.0	352	0.80	1.69	0.60
	501	502								0.00	2.03	11.69	96.02	195	750	750	CONC	0.10	71.5	352	0.80	1.50	0.56
	502	503			0.55					0.95	2.98	13.19	89.90	268	825	825	CONC	0.10	11.0	454	0.85	0.22	0.59
	503	504								0.00	2.98	13.41	89.09	266	825	825	CONC	0.10	61.0	454	0.85	1.20	0.59
	504	505								0.00	2.98	14.60	84.86	253	825	825	CONC	0.10	11.0	454	0.85	0.22	0.56
	505	506								0.00	2.98	14.82	84.15	251	825	825	CONC	0.10	36.0	454	0.85	0.71	0.55
	506	507			0.96					1.65	4.64	15.53	81.90	380	900	900	CONC	0.10	76.0	572	0.90	1.41	0.66
	507	508			1.72					2.96	7.60	16.93	77.79	591	1050	1050	CONC	0.10	79.5	863	1.00	1.33	0.69
	508	509			0.12					0.21	7.81	18.26	74.31	580	1050	1050	CONC	0.10	30.0	863	1.00	0.50	0.67
	509	510			1.20					2.07	9.88	18.76	73.09	722	1200	1200	CONC	0.10	72.5	1232	1.09	1.11	0.59
	510	511			0.70					1.21	11.08	19.87	70.53	782	1200	1200	CONC	0.10	86.5	1232	1.09	1.32	0.63
	511	611			0.88					1.52	12.60	21.19	67.74	853	1200	1200	CONC	0.10	106.5	1232	1.09	1.63	0.69
To ALN. 06 , Pipe 611 - 612											12.60	22.82											

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Notes:
1) Ottawa Rainfall-Intensity Curve
2) Min. Velocity = 0.76 m/sec

Designed: K.M.
Checked: Z.L.
Dwg. Reference: Storm Drainage Plan

PROJECT: **VILLAGE OF RICHMOND**
SWM - SUBMISSION 4
LOCATION: **City of Ottawa**
File Ref: 11-468
Date: November, 2013
Sheet No. 2 of 5

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

Ottawa

Manning 0.013 Return Frequency = 5 years

	LOCATION		AREA (Ha)							FLOW					SEWER DATA								
Location	From Node	To Node	R= 0.25	R= 0.60	R= 0.62	R= 0.70	R= 0.75	R= 0.80	R= 0.9	Indiv. 2.78 AC	Accum. 2.78 AC	Time of Conc.	Rainfall Intensity	Peak Flow Q (l/s)	DIA. (mm) (actual)	DIA. (mm) (nominal)	TYPE	SLOPE (%)	LENGTH (m)	CAPACITY (l/s)	VELOCITY (m/s)	TIME OF FLOW (min.)	RATIO Q/Q full
TRUNK 6																							
	601	602			0.84					1.45	1.45	10.00	104.19	151	600	600	CONC	0.10	68.5	194	0.69	1.66	0.78
	602	603								0.00	1.45	11.66	96.16	139	600	600	CONC	0.10	11.0	194	0.69	0.27	0.72
	603	604			0.43					0.74	2.19	11.93	95.00	208	750	750	CONC	0.10	68.5	352	0.80	1.43	0.59
	604	605			1.01					1.74	3.93	13.36	89.25	351	900	900	CONC	0.10	76.5	572	0.90	1.42	0.61
	605	606			0.98					1.69	5.62	14.78	84.28	474	975	975	CONC	0.10	79.5	708	0.95	1.40	0.67
	606	607			1.14					1.96	7.58	16.17	79.94	606	1050	1050	CONC	0.10	79.5	863	1.00	1.33	0.70
	607	608			0.47					0.81	8.39	17.50	76.25	640	1050	1050	CONC	0.10	69.0	863	1.00	1.15	0.74
	608	609								0.00	8.39	18.66	73.34	616	1050	1050	CONC	0.10	11.0	863	1.00	0.18	0.71
	609	610			0.71					1.22	9.62	18.84	72.90	701	1200	1200	CONC	0.10	34.5	1232	1.09	0.53	0.57
	610	611			1.13					1.95	11.57	19.37	71.67	829	1200	1200	CONC	0.10	25.0	1232	1.09	0.38	0.67
Contribution From ALN. 05, Pipe 511 - 611										12.60	24.16	22.82											
	611	612								0.00	24.16	22.82	64.61	1561	1800x1200	1800x1200	CONC	0.10	22.0	2636	1.27	0.29	0.59
	612	613								0.00	24.16	23.11	64.09	1549	1800x1200	1800x1200	CONC	0.10	18.0	2636	1.27	0.24	0.59
To SWMP											24.16	23.35											
TRUNK 8																							
	806	807			0.25					0.43	0.43	12.50	92.61	40	675	675	CONC	0.10	21.5	266	0.74	0.48	0.15
	807	808			1.13					1.95	2.38	12.98	90.70	216	675	675	CONC	0.10	35.5	266	0.74	0.80	0.81
	808	809			0.59					1.02	3.40	13.78	87.72	298	750	750	CONC	0.10	11.0	352	0.80	0.23	0.85
	809	710								0.00	3.40	14.01	86.90	295	750	750	CONC	0.10	18.5	352	0.80	0.39	0.84
To Trunk 7, Pipe 710 - 711											3.40	14.40											
TRUNK 75																							
	751	752			0.7					1.21	1.21	10.00	104.19	126	600	600	CONC	0.10	94.0	194	0.69	2.28	0.65
	752	753			0.48					0.83	2.03	12.28	93.51	190	750	750	CONC	0.10	82.0	352	0.80	1.72	0.54
	753	754			2.38					4.10	6.14	14.00	86.95	534	975	975	CONC	0.10	86.0	708	0.95	1.51	0.75
	754	705			0.76					1.31	7.45	15.51	81.96	610	1050	1050	CONC	0.10	116.0	863	1.00	1.94	0.71
To Trunk 7, Pipe 705 - 706											7.45	17.45											
TRUNK 7																							
	7000	700			2.59					4.46	4.46	10.00	104.19	465	900	900	CONC	0.10	71.5	572	0.90	1.32	0.81
	700	701			0.53					0.91	5.38	11.32	97.68	525	975	975	CONC	0.10	74.0	708	0.95	1.30	0.74
	701	702			0.66					1.14	6.52	12.62	92.11	600	1050	1050	CONC	0.10	40.0	863	1.00	0.67	0.70
	702	703			4.17					7.19	13.70	13.29	89.51	1227	1200X1800	1200X1800	CONC	0.10	71.5	2636	1.27	0.94	0.47
	703	704			2.52					4.34	18.05	14.23	86.12	1554	1200X1800	1200X1800	CONC	0.10	105.5	2636	1.27	1.39	0.59
	704	705			4.21					7.26	25.30	15.62	81.61	2065	1200X1800	1200X1800	CONC	0.10	71.5	2636	1.27	0.94	0.78
Contribution From Trunk 75, Pipe 754 - 705										7.45	32.75	17.45											
	705	706			2.74					4.72	37.47	17.45	76.41	2863	1200X2400	1200X2400	CONC	0.10	67.0	3795	1.36	0.82	0.75
	706	707			0.53					0.91	38.38	18.27	74.29	2852	1200X2400	1200X2400	CONC	0.10	83.5	3795	1.36	1.03	0.75
	707	708			2.71					4.67	43.06	19.30	71.84	3093	1200X2400	1200X2400	CONC	0.10	70.0	3795	1.36	0.86	0.81
	708	709			0.93					1.60	44.66	20.16	69.91	3122	1200X2400	1200X2400	CONC	0.10	66.0	3795	1.36	0.81	0.82
	709	710								0.00	44.66	20.97	68.20	3046	1200X2400	1200X2400	CONC	0.10	70.5	3795	1.36	0.87	0.80
Contribution From Trunk 80, Pipe 809 - 710										3.40	48.05	14.40											
	710	711								0.00	48.05	21.83	66.47	3194	1200X2400	1200X2400	CONC	0.10	131.0	3795	1.36	1.61	0.84
	711	712								0.00	48.05	23.45	63.50	3051	1200X2400	1200X2400	CONC	0.10	21.5	3795	1.36	0.26	0.80
	712	7012								0.00	48.05	23.71	63.04	3029	1200X2400	1200X2400	CONC	0.10	8.5	3795	1.36	0.10	0.80
	TO SWMP										48.05	23.81											

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Notes:
1) Ottawa Rainfall-Intensity Curve
2) Min. Velocity = 0.76 m/sec

Designed: K.M.
Checked: Z.L.
Dwg. Reference: Storm Drainage Plan

PROJECT: **VILLAGE OF RICHMOND**
SWM - SUBMISSION 4
LOCATION: **City of Ottawa**
File Ref: 11-468
Date: November, 2013
Sheet No. 3 of 5

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

Manning 0.013 Return Frequency = 5 years

	LOCATION		AREA (Ha)							FLOW					SEWER DATA								
			R=	R=	R=	R=	R=	R=	R=	Indiv.	Accum.	Time of	Rainfall	Peak Flow	DIA. (mm)	DIA. (mm)	TYPE	SLOPE	LENGTH	CAPACITY	VELOCITY	TIME OF	RATIO
Location	From Node	To Node	0.25	0.60	0.62	0.70	0.75	0.80	0.9	2.78 AC	2.78 AC	Conc.	Intensity	Q (l/s)	(actual)	(nominal)		(%)	(m)	(l/s)	(m/s)	FLOW (min.)	Q/Q full
TRUNK 9																							
	901	902			0.60					1.03	1.03	10.00	104.19	108	600	600	CONC	0.10	65.0	194	0.69	1.58	0.56
	902	903			0.44					0.76	1.79	11.58	96.54	173	675	675	CONC	0.10	70.0	266	0.74	1.57	0.65
	903	904			0.34					0.59	2.38	13.15	90.06	214	750	750	CONC	0.10	69.5	352	0.80	1.45	0.61
	904	905			1.34					2.31	4.69	14.60	84.87	398	900	900	CONC	0.10	116.0	572	0.90	2.15	0.70
	905	1007								0.00	4.69	16.75	78.30	367	900	900	CONC	0.10	16.0	572	0.90	0.30	0.64
To Trunk 10, Pipe 1008 - 1009											4.69	17.05											
OTTAWA STREET - TRUNK 12														**100 Yr**									
External from South: 100-year, 3-hour Chicago Storm Flow (JFSA)														1600									
	1600	1601												1600	1219	1200	CONC	0.16	107.5	1625	1.39	1.29	0.98
	1601	1602												1600	1219	1200	CONC	0.16	95.0	1625	1.39	1.14	0.98
	1602	1603												1600	1219	1200	CONC	0.16	95.0	1625	1.39	1.14	0.98
	1603	1604												1600	1219	1200	CONC	0.16	95.0	1625	1.39	1.14	0.98
	1604	1605												1600	1219	1200	CONC	0.16	86.5	1625	1.39	1.03	0.98
	1605	1008												1600	1219	1200	CONC	0.16	72.0	1625	1.39	0.86	0.98
To MH 1008 - HW														1600									
TRUNK 10																							
	1001	1002			0.53					0.91	0.91	10.00	104.19	95	525	525	CONC	0.10	72.5	136	0.63	1.92	0.70
	1002	1003			0.31					0.53	1.45	11.92	95.02	138	600	600	CONC	0.10	77.0	194	0.69	1.87	0.71
	1003	1004			0.84					1.45	2.90	13.79	87.67	254	750	750	CONC	0.10	71.5	352	0.80	1.50	0.72
	1004	1005			0.83					1.43	4.33	15.29	82.64	358	900	900	CONC	0.10	74.0	572	0.90	1.37	0.62
	1005	1006			0.57					0.98	5.31	16.66	78.56	417	975	975	CONC	0.10	71.0	708	0.95	1.25	0.59
	1006	1007			3.15					5.43	10.74	17.90	75.21	808	1200	1200	CONC	0.10	83.5	1232	1.09	1.28	0.66
Contribution From Trunk 9, Pipe 905 - 1007										4.69	15.43	17.05											
	1007	1008			0.17					0.29	15.72	19.18	72.10	1133	1200	1200	CONC	0.20	29.0	1743	1.54	0.31	0.65
Contribution From Trunk 16, Pipe 1605 - 1008														1600									
	1008	HW								0.00	15.72	19.49	71.38	2722	2400x1200	2400x1200	CONC	0.10	23.0	3795	1.36	0.28	0.72
To SWMP											15.72	19.78											
TRUNK 12																							
	1201	1202			1.71					2.95	2.95	10.00	104.19	307	825	825	CONC	0.10	51.5	454	0.85	1.01	0.68
	1202	1203								0.00	2.95	11.01	99.14	292	825	825	CONC	0.10	11.0	454	0.85	0.22	0.64
	1203	1204			0.85					1.47	4.41	11.23	98.13	433	975	975	CONC	0.10	67.5	708	0.95	1.19	0.61
	1204	1205								0.00	4.41	12.41	92.97	410	975	975	CONC	0.11	67.5	743	1.00	1.13	0.55
	1205	1206			0.58					1.00	5.41	13.54	88.58	479	975	975	CONC	0.10	11.0	708	0.95	0.19	0.68
	1206	1207								0.00	5.41	13.74	87.88	476	975	975	CONC	0.10	56.0	708	0.95	0.98	0.67
	1207	1208								0.00	5.41	14.72	84.48	457	975	975	CONC	0.10	11.0	708	0.95	0.19	0.65
	1208	1209								0.00	5.41	14.91	83.84	454	975	975	CONC	0.10	36.0	708	0.95	0.63	0.64
	1209	1210			0.64					1.10	6.52	15.54	81.84	533	1050	1050	CONC	0.10	7.5	863	1.00	0.13	0.62
	1210	1211								0.00	6.52	15.67	81.46	531	1050	1050	CONC	0.10	29.0	863	1.00	0.48	0.61
	1211	1212								0.00	6.52	16.15	80.01	521	1050	1050	CONC	0.10	7.5	863	1.00	0.13	0.60
	1212	1213								0.00	6.52	16.28	79.64	519	1050	1050	CONC	0.10	64.0	863	1.00	1.07	0.60
	1213	1214			1.71					2.95	9.46	17.35	76.66	725	1200	1200	CONC	0.10	71.5	1232	1.09	1.09	0.59
	1214	1215			0.94					1.62	11.08	18.44	73.87	819	1200	1200	CONC	0.10	71.5	1232	1.09	1.09	0.66
	1215	1216			2.13					3.67	14.75	19.53	71.29	1052	1350	1350	CONC	0.10	71.5	1687	1.18	1.01	0.62
	1216	1217			3.05					5.26	20.01	20.55	69.08	1382	1500	1500	CONC	0.10	84.5	2234	1.26	1.11	0.62
	1217	1218			1.33					2.29	22.30	21.66	66.81	1490	1500	1500	CONC	0.10	83.0	2234	1.26	1.09	0.67
	1218	1219			0.39					0.67	22.98	22.75	64.74	1487	1500	1500	CONC	0.10	46.5	2234	1.26	0.61	0.67
	1219	1108			2.49					4.29	27.27	23.37	63.64	1735	1500	1500	CONC	0.10	74.0	2234	1.26	0.97	0.78
To Trunk 11, Pipe 1108 - 1109											27.27	24.34											

Definitions:
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A = Areas in hectares (ha)
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R = Runoff Coefficient

Notes:
1) Ottawa Rainfall-Intensity Curve
2) Min. Velocity = 0.76 m/sec

Designed: K.M.
Checked: Z.L.
Dwg. Reference: Storm Drainage Plan

PROJECT: **VILLAGE OF RICHMOND**
SWM - SUBMISSION 4
LOCATION: **City of Ottawa**
File Ref: 11-468
Date: November, 2013
Sheet No. 4 of 5

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

Manning 0.013 Return Frequency = 5 years

	LOCATION		AREA (Ha)							FLOW					SEWER DATA								
Location	From Node	To Node	R= 0.25	R= 0.60	R= 0	R= 0.70	R= 0.75	R= 0.80	R= 0.9	Indiv. 2.78 AC	Accum. 2.78 AC	Time of Conc.	Rainfall Intensity	Peak Flow Q (l/s)	DIA. (mm) (actual)	DIA. (mm) (nominal)	TYPE	SLOPE (%)	LENGTH (m)	CAPACITY (l/s)	VELOCITY (m/s)	TIME OF FLOW (min.)	RATIO Q/Q full
TRUNK 11																							
	1101	1102			1.21					2.09	2.09	10.00	104.19	217	750	750	CONC	0.10	119.0	352	0.80	2.49	0.62
	1102	1103								0.00	2.09	12.49	92.66	193	750	750	CONC	0.10	8.0	352	0.80	0.17	0.55
	1103	1104			0.89					1.53	3.62	12.66	91.98	333	900	900	CONC	0.10	65.5	572	0.90	1.21	0.58
	1104	1105			0.43					0.74	4.36	13.87	87.40	381	900	900	CONC	0.10	71.5	572	0.90	1.32	0.67
	1105	1106								0.00	4.36	15.19	82.94	362	900	900	CONC	0.10	54.5	572	0.90	1.01	0.63
	1106	1107			2.03					3.50	7.86	16.20	79.86	628	1050	1050	CONC	0.10	7.0	863	1.00	0.12	0.73
	1107	1108								0.00	7.86	16.32	79.52	625	1050	1050	CONC	0.40	113.5	1726	1.99	0.95	0.36
Contribution From Trunk 12, Pipe 1219 - 1108										27.27	35.13	24.34											
	1108	1109			1.39					2.40	37.52	24.34	61.97	2325	1500	1500	CONC	0.20	73.5	3160	1.79	0.68	0.74
	1109	1110								0.00	37.52	25.02	60.86	2283	1500x2400	1500x2400	CONC	0.10	67.5	5250	1.49	0.75	0.43
	1110	HW								0.00	37.52	25.78	59.68	2239	1500x2400	1500x2400	CONC	0.10	22.5	5250	1.49	0.25	0.43
To SWMP											37.52	26.03											

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Notes:
1) Ottawa Rainfall-Intensity Curve
2) Min. Velocity = 0.76 m/sec

Designed: K.M.	PROJECT: **VILLAGE OF RICHMOND SWM - SUBMISSION 4**		
Checked: Z.L.	LOCATION: **City of Ottawa**		
Dwg. Reference: Storm Drainage Plan	File Ref: 11-468	Date: November, 2013	Sheet No. 5 of 5

SANITARY SEWER CALCULATION SHEET

Manning's n=0.013

LOCATION			RESIDENTIAL AREA AND POPULATION									COMM		INDUST		INSTIT		C+I+I	INFILTRATION				PIPE					
STREET	FROM M.H.	TO M.H.	AREA (ha)	UNITS	UNITS Towns	UNITS* (KWR)	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA (mm)	SLOPE (%)	CAP. (FULL) (l/s)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
TRUNK 2																												
	201A	202A	0.9				54	0.90	54	4.00	0.88							0.00	0.90	0.90	0.252	1.13	53.5	200	0.65	26.44	0.83	0.41
	202A	203A					0	0.90	54	4.00	0.88							0.00	0.00	0.90	0.252	1.13	14.5	200	0.40	20.74	0.67	0.35
	203A	204A	0.9				54	1.80	108	4.00	1.75							0.00	0.90	1.80	0.504	2.25	59.5	200	0.40	20.74	0.67	0.44
	204A	205A					0	1.80	108	4.00	1.75							0.00	0.00	1.80	0.504	2.25	79.0	200	0.40	20.74	0.67	0.44
	205A	206A	0.5				30	2.30	138	4.00	2.24							0.00	0.50	2.30	0.644	2.88	14.0	200	0.40	20.74	0.67	0.47
	206A	207A					0	2.30	138	4.00	2.24							0.00	0.00	2.30	0.644	2.88	56.0	250	0.24	29.13	0.59	0.37
	207A	208A					0	2.30	138	4.00	2.24							0.00	0.00	2.30	0.644	2.88	7.5	250	0.24	29.13	0.59	0.37
	208A	209A					0	2.30	138	4.00	2.24							0.00	0.00	2.30	0.644	2.88	36.5	250	0.24	29.13	0.59	0.37
	209A	210A	0.6				36	2.90	174	4.00	2.82							0.00	0.60	2.90	0.812	3.63	10.0	250	0.24	29.13	0.59	0.40
	210A	211A					0	2.90	174	4.00	2.82							0.00	0.00	2.90	0.812	3.63	28.5	250	0.24	29.13	0.59	0.40
	211A	212A					0	2.90	174	4.00	2.82							0.00	0.00	2.90	0.812	3.63	4.5	250	0.24	29.13	0.59	0.40
	212A	213A					0	2.90	174	4.00	2.82							0.00	0.00	2.90	0.812	3.63	62.0	250	0.19	25.92	0.53	0.37
	213A	214A	1.8				108	4.70	282	4.00	4.57							0.00	1.80	4.70	1.316	5.89	71.5	250	0.24	29.13	0.59	0.46
	214A	215A	1.0				60	5.70	342	4.00	5.54							0.00	1.00	5.70	1.596	7.14	75.5	250	0.24	29.13	0.59	0.49
	215A	216A	1.5				90	7.20	432	4.00	7.00							0.00	1.50	7.20	2.016	9.02	75.5	250	0.24	29.13	0.59	0.52
	216A	217A	2.8				168	10.00	600	3.93	9.55							0.00	2.80	10.00	2.800	12.35	74.5	250	0.24	29.13	0.59	0.56
			0.8				0	10.80	600									0.00	0.80	10.80								
	217A	218A	0.4				24	11.20	624	3.92	9.91							0.00	0.40	11.20	3.136	13.05	89.0	250	0.24	29.13	0.59	0.57
	218A	219A	8.3				498	19.50	1122	3.77	17.14							0.00	8.30	19.50	5.460	22.60	50.5	300	0.19	42.15	0.59	0.60
	219A	220A	3.5				210	23.00	1332	3.72	20.07							0.00	3.50	23.00	6.440	26.51	91.5	300	0.19	42.15	0.59	0.62
	220A	221A	2.3				138	25.30	1470	3.69	21.97							0.00	2.30	25.30	7.084	29.05	71.5	300	0.19	42.15	0.59	0.64
	221A	222A	2.8				168	28.10	1638	3.65	24.22							0.00	2.80	28.10	7.868	32.09	58.5	300	0.19	42.15	0.59	0.65
	222A	223A	2.8				168	30.90	1806	3.62	26.48							0.00	2.80	30.90	8.652	35.13	55.5	375	0.14	65.60	0.60	0.61
	223A	224A	1.7				102	32.60	1908	3.60	27.83							0.00	1.70	32.60	9.128	36.96	76.5	375	0.14	65.60	0.60	0.62
			3.3				0	35.90	1908									0.00	3.30	35.90								
	224A	225A	4.4				264	40.30	2172	3.56	31.32							0.00	4.40	40.30	11.284	42.60	71.5	375	0.14	65.60	0.600	0.64
	225A	226A	1.9				114	42.20	2286	3.54	32.78							0.00	1.90	42.20	11.816	44.60	33.5	375	0.14	65.60	0.60	0.64
	226A	227A	0.0				0	42.20	2286	3.54	32.78					2.40	2.40	2.16	2.40	44.60	12.488	47.43	71.5	375	0.14	65.60	0.60	0.65
	227A	228A	1.3				78	43.50	2364	3.53	33.80						2.40	2.16	1.30	45.90	12.852	48.81	71.5	375	0.14	65.60	0.60	0.66
	228A	229A	6.6				396	50.10	2760	3.47	38.80						2.40	2.16	6.60	52.50	14.700	55.66	73.5	450	0.12	98.76	0.62	0.64
	229A	230A	0.5				85	50.60	2845	3.46	39.88						2.40	2.16	0.50	53.00	14.840	56.88	81.0	450	0.12	98.76	0.62	0.64
			0.7				119	51.30	2964								2.40	2.16	0.70	53.70								
	230A	231A	2.5				150	53.80	3114	3.43	43.27						2.40	2.16	2.50	56.20	15.736	61.17	72.5	450	0.12	98.76	0.62	0.65
			0.3				51	54.10	3165								2.40	2.16	0.30	56.50								
	231A	232A	0.8				48	54.90	3213	3.42	44.51						2.40	2.16	0.80	57.30	16.044	62.71	55.5	450	0.12	98.76	0.62	0.66
	232A	233A					0	54.90	3213	3.42	44.51						2.40	2.16	0.00	57.30	16.044	62.71	77.0	450	0.12	98.76	0.62	0.66
			0.2				12	55.10	3225								2.40	2.16	0.20	57.50								
			0.4				68	55.50	3293								2.40	2.16	0.40	57.90								
	233A	121A	1.0				60	56.50	3353	3.40	46.18						2.40	2.16	1.00	58.90	16.492	64.83	114.0	450	0.12	98.76	0.62	0.66
To Trunk 1 , Pipe 121A - 122A								56.50	3353								2.40											

Average Daily Flow =	350	l/p/day	350 l/p/day	Industrial Peak Factor = as per MOE Graph		
Comm/Inst Flow =	50000	L/ha/da	50000 L/ha/da	Extraneous Flow =	0.280	L/s/ha
Industrial Flow =	35000	L/ha/da	35000 L/ha/da	Minimum Velocity =	0.760	m/s
Max Res. Peak Factor =	4.00		4	Manning's n =	0.013	
Commercial/Inst peak Factor =	1.50		2	Townhouse coeff=	2.7	
Institutional	0.60	l/s/Ha		Single house coeff=	3.4	

K.M.

Checked:
Z.L.

Dwg. Reference:
Sanitary Drainage Plan

VILLAGE OF RICHMOND
SWM - SUBMISSION 4

LOCATION:
City of Ottawa

File Ref: 11-468

Date: November, 2013

Sheet No. 1 of 2

SANITARY SEWER CALCULATION SHEET

Manning's n=0.013

LOCATION				RESIDENTIAL AREA AND POPULATION									COMM		INDUST		INSTIT		C+I+I	INFILTRATION				PIPE					
STREET		FROM M.H.	TO M.H.	AREA (ha)	UNITS	UNITS Towns	UNITS* (KWR)	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA (mm)	SLOPE (%)	CAP. (FULL) (l/s)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
TRUNK 26																													
		401A	402A	0.9				54	0.90	54	4.00	0.88							0.00	0.90	0.90	0.252	1.13	94.5	200	0.65	26.44	0.83	0.41
		402A	403A	0.6				36	1.50	90	4.00	1.46							0.00	0.60	1.50	0.420	1.88	120.0	200	0.35	19.40	0.60	0.38
		403A	404A	0.4				24	1.90	114	4.00	1.85							0.00	0.40	1.90	0.532	2.38	106.5	200	0.35	19.40	0.60	0.41
		404A	405A	0.6				36	2.50	150	4.00	2.43							0.00	0.60	2.50	0.700	3.13	40.5	250	0.30	32.57	0.67	0.42
		405A	406A	2.8				168	5.30	318	4.00	5.15							0.00	2.80	5.30	1.484	6.63	30.0	250	0.30	32.57	0.67	0.52
		406A	407A	1.7				102	7.00	420	4.00	6.81							0.00	1.70	7.00	1.960	8.77	90.0	250	0.30	32.57	0.67	0.56
		407A	408A	2.6				156	9.60	576	3.94	9.19							0.00	2.60	9.60	2.688	11.88	54.5	250	0.30	32.57	0.67	0.62
		408A	111A					0	9.60	576	3.94	9.19							0.00	0.00	9.60	2.688	11.88	35.0	250	0.30	32.57	0.67	0.62
To Trunk 1 , Pipe 111A - 112A									9.60	576																			
Trunk 3																													
	301A	302A		0.000				0	0.00	0									0.00	0.00	0.00	0.000	0.00	80.5	300	0.70	80.91	1.14	0.06
	302A	303A		0.000				0	0.00	0									0.00	0.00	0.00	0.000	0.00	8.5	300	0.70	80.91	1.14	0.06
				0.600				36	0.60	36									0.00	0.60	0.60	0.168							
	303A	304A		0.100				6	0.70	42	2.00	0.34							0.00	0.10	0.70	0.196	0.54	78.5	300	0.40	61.16	0.87	0.26
	304A	305A		0.000				0	0.70	42	2.00	0.34							0.00	0.00	0.70	0.196	0.54	80.0	300	0.40	61.16	0.87	0.26
	305A	123A		0.000				0	0.70	42	2.00	0.34							0.00	0.00	0.70	0.196	0.54	96.0	300	0.40	61.16	0.87	0.26
To Trunk 1 , Pipe 123A - 124A									0.70	42																			
TRUNK 1																													
		102A	103A	1.0				60	1.00	60	4.00	0.97							0.00	1.00	1.00	0.280	1.25	120.0	200	0.65	26.44	0.83	0.42
		103A	104A	1.0				60	2.00	120	4.00	1.94							0.00	1.00	2.00	0.560	2.50	74.0	250	0.25	29.73	0.61	0.37
		104A	105A	0.9				54	2.90	174	4.00	2.82							0.00	0.90	2.90	0.812	3.63	66.0	250	0.25	29.73	0.61	0.41
		105A	106A					0	2.90	174	4.00	2.82							0.00	0.00	2.90	0.812	3.63	82.0	250	0.25	29.73	0.61	0.41
		106A	107A	3.2				192	6.10	366	4.00	5.93							0.00	3.20	6.10	1.708	7.64	38.5	250	0.25	29.73	0.61	0.51
		107A	108A					0	6.10	366	4.00	5.93							0.00	0.00	6.10	1.708	7.64	56.5	250	0.25	29.73	0.61	0.51
		108A	109A	2.1				126	8.20	492	3.98	7.93							0.00	2.10	8.20	2.296	10.23	38.0	250	0.25	29.73	0.61	0.55
		109A	110A					0	8.20	492	3.98	7.93							0.00	0.00	8.20	2.296	10.23	76.5	250	0.25	29.73	0.61	0.55
		110A	111A	1.6				96	9.80	588	3.94	9.38							0.00	1.60	9.80	2.744	12.12	76.0	250	0.25	29.73	0.61	0.58
Contribution From Trunk 26, Pipe 408A - 111A									9.60	576										9.60	19.40								
		111A	112A	3.7				222	23.10	1386	3.70	20.77							0.00	3.70	23.10	6.468	27.24	120.0	375	0.15	67.91	0.62	0.59
				2.8				168	25.90	1554	3.67	23.10								2.80	25.90								
		112A	113A	0.8				48	26.70	1434	3.69	21.44							0.00	0.80	26.70	7.476	28.92	119.5	375	0.15	67.91	0.62	0.59
		113A	114A	0.9				54	27.60	1488	3.68	22.18							0.00	0.90	27.60	7.728	29.91	112.5	375	0.15	67.91	0.62	0.60
		114A	115A	4.5				270	32.10	1758	3.63	25.85							0.00	4.50	32.10	8.988	34.84	79.0	375	0.15	67.91	0.62	0.62
		115A	116A					0	32.10	1758	3.63	25.85							0.00	0.00	32.10	8.988	34.84	94.5	375	0.15	67.91	0.62	0.62
		116A	117A	2.8				168	34.90	1926	3.60	28.09							0.00	2.80	34.90	9.772	37.86	14.5	375	0.15	67.91	0.62	0.64
		117A	118A					0	34.90	1926	3.60	28.09							0.00	0.00	34.90	9.772	37.86	65.0	375	0.15	67.91	0.62	0.64
		118A	119A	1.0				60	35.90	1986	3.59	28.88							0.00	1.00	35.90	10.052	38.93	90.5	375	0.15	67.91	0.62	0.64
		119A	120A	6.7				402	42.60	2388	3.53	34.15							0.00	6.70	42.60	11.928	46.08	117.0	375	0.15	67.91	0.62	0.66
		120A	121A	6.2				372	48.80	2760	3.47	38.80							0.00	6.20	48.80	13.664	52.46	24.5	450	0.12	98.76	0.62	0.63
Contribution From Trunk 2, Pipe 233A - 121A									56.50	3353								2.40		58.90	107.70								
		121A	122A					0	105.30	6113	3.16	78.25						2.40	2.16	0.00	107.70	30.156	110.57	74.5	525	0.12	148.98	0.69	0.76
		122A	123A					0	105.30	6113	3.16	78.25						2.40	2.16	0.00	107.70	30.156	110.57	60.5	525	0.12	148.98	0.69	0.76
Contribution From Trunk 3, Pipe 305A - 123A									0.70	42										0.70	108.40								
		123A	124A					0	106.00	6155	3.16	78.79						2.40	2.16	0.00	108.40	30.352	111.30	113.5	525	0.12	148.98	0.69	0.76
		124A	125A					0	106.00	6155	3.16	78.79						2.40	2.16	0.00	108.40	30.352	111.30	95.5	525	0.12	148.98	0.69	0.76
		125A	126A					0	106.00	6155	3.16	78.79						2.40	2.16	0.00	108.40	30.352	111.30	64.0	525	0.12	148.98	0.69	0.76

DESIGN PARAMETERS

Average Daily Flow =	350	l/p/day	350 l/p/day	Industrial Peak Factor = as per MOE Graph		
Comm/Inst Flow =	50000	L/ha/da	50000 L/ha/da	Extraneous Flow =	0.280	L/s/ha
Industrial Flow =	35000	L/ha/da	35000 L/ha/da	Minimum Velocity =	0.760	m/s
Max Res. Peak Factor =	4.00		4	Manning's n =	0.013	
Commercial/Inst peak Factor =	1.50		2	Townhouse coeff=	2.7	
Institutional	0.60			Single house coeff=	3.4	

Designed: K.M.	PROJECT: **VILLAGE OF RICHMOND** **SWM - SUBMISSION 4**		
Checked: Z.L.	LOCATION: **City of Ottawa**		
Dwg. Reference: Sanitary Drainage Plan	File Ref: 11-468	Date: November, 2013	Sheet No. 2 of 2

APPENDIX I

JFSA Hydrologic / Hydraulic Modeling

J.F. Sabourin and Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS

52 Springbrook Drive
Ottawa (Stittsville), ON K2S 1B9
TEL: (613) 836-3884
FAX: (613) 836-0332
WEB: www.jfsa.com

October 31, 2013

David Schaeffer Engineering Ltd.
120 Iber Road, Unit 203
Ottawa, Ontario K2S 1E9

Attention: **Kevin Murphy, P.Eng.**

Subject: **Richmond Village (South) Limited Subdivision / Preliminary Stormwater Management Plan** *our file: 922-11*

As requested by your office, we have evaluated, based on the provided information as described below; (i) the adequacy of the proposed minor system to convey the 5- and 100-year storm flows from within the development to the stormwater management (SWM) facilities; (ii) the capacity of the proposed major system to safely convey the excess 100-year flows to the SWM facilities; (iii) the operation of the proposed SWM facilities based on quality, erosion and quantity control requirements; and (iv) the hydraulic impact of the proposed subdivision and SWM facilities on the Van Gaal Drain.

The proposed Richmond Village (South) development consists of a 126.81 ha drainage area to be treated by two Stormwater Management (SWM) facilities; SWM Facility 1 (91.82 ha at 51% imperviousness) discharging to Van Gaal Drain, and SWM Facility 2 (34.99 ha at 51% imperviousness) discharging to the Jock River. It should be noted that excess major system flows from 30.79 ha and 21.75 ha of the SWM Facility 1 drainage area will discharge directly to the Van Gaal Drain and Moore Drain Tributary, respectively. Additionally, of the undeveloped lands south of the proposed subdivision, 97.50 ha will drain through SWM Facility 1, 71.8 ha will drain through SWM Facility 2, and 94.2 ha will be conveyed through the subdivision by a tributary of the Moore Drain. Refer to Figures 1A and 1B of Attachment 1 for the existing and proposed subject site drainage areas, respectively.

SWM Facility 1 is a wet pond discharging to the Van Gaal Drain and requires quality, erosion and quantity control, and SWM Facility 2 is a wetland discharging to the Jock River and requires quality control only. Quality control will be provided for SWM Facilities 1 and 2 by permanent pools and 40 m^3/ha of active storage volume (released over 24 hours) in accordance with Ministry of the Environment enhanced protection requirements. Erosion control for SWM Facility 1 will be provided by controlling the 2-year release rate to 330 L/s or less, where 330 L/s is the erosion threshold for the Van Gaal Drain identifed by Parish Geomorphic in the *Natural Environment & Impact Assessment Study for the Mattamy Richmond Lands* (March 2009). Furthermore, the October 26, 2012 *Van Gaal Drain Erosion Assessment* memo by JTB Environmental Systems Inc. indicates that the 2-year outflows from SWM Facility 1 should discharge to the Van Gaal Drain at a velocity of 0.225 m/s or less. This may be achieved by a plunge pool or other velocity reduction measures at the SWM Facility 1 extended detention outlet pipe to the Van Gaal Drain. Quantity control will be provided for SWM Facility 1 to limit the 2- to 100-year release rates to pre-development levels.

The SWMHYMO program was used to simulate the major system flows and minor system inflows for the drainage area to the SWM facilities, to estimate the quantity control target release rates for SWM Facility 1 based on existing conditions, and to simulate proposed conditions flows on the Van Gaal Drain and Jock River. The XPSWMM program was used to model the conveyance of the minor system flows and the operation of the SWM facilities. The HEC-RAS program was used to simulate water levels on the Van Gaal Drain and Jock River. Refer to Attachment 2 for a schematic of the XPSWMM model; digital SWMHYMO, XPSWMM and HEC-RAS models are attached.

PROPOSED MINOR AND MAJOR SYSTEM DRAINAGE

The proposed minor and major system drainage routes are shown in plan view in Figure 1B of Attachment 1. The catchments shown in Figure 1B were divided into front yard / non-residential areas and rear yard areas for the SWMHYMO model, where rear yard areas are equal to approximately 37.5% of residential catchments based on a typical subdivision layout.

In accordance with City of Ottawa standards, the minor system has been designed to accommodate the 5-year post-development flows from within the site. For modelling purposes, minor system captures rates on front yard / non-residential areas were limited to 112% of the 5-year flows simulated in SWMHYMO, in order to account for additional flows captured by standard inlet control devices and catchbasins during the 100-year storm. In accordance with the potential design approach suggested in the January 2012 *City of Ottawa Technical Bulletin ISTB-2012-1*, 100% of the 100-year flows simulated in the rear yard areas are to be captured to the minor system. Additionally, 100% of the 100-year flows on Ottawa Street and Perth Street are to be captured to the minor system to prevent flows from crossing these roads.

The street segments within the proposed development are to be designed using a ‘saw tooth’ or ‘sagged’ road profile. The runoff from within front yard / non-residential areas will be conveyed to catchbasins located at low points on the street. Flows in excess of the minor system capture rate are temporarily stored within approximately 30 m^3/ha of surface storage and released slowly to the storm sewers and then, when that storage is surpassed, conveyed overland to the next downstream catchment. A 0.5% longitudinal slope, 3% road cross-slope and 3.5% shoulder cross-slope were assumed for the purposes of modelling the routing of major system flows along the main streets in the development. Road widths are as provided by DSEL. Refer to Table 1A of Attachment 2 for a summary of the subdivision drainage areas as modelled in SWMHYMO.

The proposed storm sewers will convey 100% of the 100-year flows generated on 97.5 ha of undeveloped lands south of the subject site to SWM Facility 1, and on 71.8 ha of undeveloped lands south of the subject site to SWM Facility 2. Total drainage areas through the subject site under existing and proposed conditions, including undeveloped lands and the proposed subdivision drainage areas, are summarized in Tables 1B and 1C of Attachment 2.

We understand that homes in the proposed subdivision are to be serviced by sump pumps. It is estimated that sump pumps will contribute approximately 0.23 L/s/ha of flow to the proposed storm sewer based on a average development density of approximately 27.8 lots/ha, where 50% of sump pumps are on at any given time, and a flow contribution of 1.44 m^3/day/lot per the October 3, 2012 *Updated Assessment of Subsurface Drainage and Analysis of 100 Year Flood Event - Proposed Village of Richmond Development* memo by Golder Associates Limited. Sump pump flows have been accounted for in evaluating the operations of the proposed minor system.

In addition to the typical summer design storms, the performances of the SWM facilities were also assessed for the 100-year 10-day spring snowmelt plus rainfall event. This is in keeping with the previous floodplain mapping studies for the Van Gaal Drain and the Jock River, where the spring snowmelt plus rainfall events resulted in the highest flows and water levels on the watercourses, both at and downstream of the SWM facility outfalls.

Several modifications were made to the drainage area characteristics in the spring SWMHYMO models in order to best represent spring conditions. An SCS curve number (CN) of 95 was selected for undeveloped lands to model the limited infiltration capacity of the frozen soils. Similarly, the Horton’s minimum and maximum infiltration parameters for the sudivision drainage areas were set to 2.4 mm/hour; the lowest infiltration rate in the *SWMHYMO User’s Manual* (May 2000, JFSA). Furthermore, it was assumed, according to generally accepted practice, that half of the volume in the snowmelt plus rainfall event may be attributed to snowmelt, and half to rainfall. As it is expected that most of the snow on impervious areas like roads, driveways and roofs would have melted prior to such an event, half of the impervious area was removed from each developed drainage area such that only the runoff resulting from rainfall, not snowmelt, is simulated for impervious areas.

The SWMHYMO and XPSWMM analyses, discussed in the next sections, demonstrate that it is possible for the proposed drainage systems for the development to control the excess flow during a 100-year storm and safely capture and convey the minor system flows to the SWM facility.

ASSUMPTIONS AND SOURCES OF DATA USED

The following parameters and assumptions used in the analysis are based on City of Ottawa standards and generally accepted stormwater management design guidelines.

- Stormwater Management Model: *SWMHYMO (version 5.02), XPSWMM (version 10), HEC-RAS (version 4.1.0)*
- Minor System Design: *1:5 year*
- Major System Design: *1:100 year*
- Max. Allowable Flow Depth: *30 cm above gutter.*
- Extent of Major System: *Must be contained within the municipal right-of-way.*
- SWMHYMO Model Parameters: *Fo = 76.2 mm/hr, Fc = 13.2 mm/hr, DCAY = 4.14/hr, D.Stor.Imp. = 1.57 mm, D.Stor.Per. = 4.67 mm (as per 2004 City of Ottawa Guidelines).*
- Undeveloped Area Characteristics: *As per "Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond" (November 2009, JFSA).*
- Imperviousness: *SWM Facilities: based on SWM block layout.*
  Front Yard / Non-Residential: based on runoff coefficient (C) where Percent Imperviousness = (C - 0.2) / 0.7 x 100%.
  Rear Yard: equal to half of the front yard percent imperviousness. Half of that rear yard impervious area is assumed to be indirectly connected.
- Design Storms: *Chicago 3-hour and SCS Type II 24-hour design storms based on 2004 City of Ottawa Sewer Design Guidelines; maximum intensity averaged over 10 minutes. 10-day snowmelt plus rainfall events based on AES Ottawa CDA snowmelt plus rainfall IDF curves; maximum intensity averaged over 1 hour.*
- Historical Events: *July 1st, 1979 event per 2004 City of Ottawa Sewer Design Guidelines.*
- Climate Change Street Test: *20% increase in the 100-year, 3-hour Chicago and 100-year, 24-hour SCS storms, as per January 2012 City of Ottawa Technical Bulletin ISTB-2012-1.*
- Manning's Roughness Coeff.: *0.013 for concrete pipes (free flow).*
- Minor System Losses: *Refer to Attachment 2 for manhole loss coefficients.*
- Sump Pump Flows: *0.23 L/s/ha based on 27.8 lot/ha, 1.44* m^3*/day/lot (per October 3, 2012 "Updated Assessment of Subsurface Drainage and Analysis of 100 Year Flood Event - Proposed Village of Richmond Development" memo by Golder Associates Limited), and 50% of sump pumps on.*
- Downstream HGL: *Jock River water levels at SWM Facility 2 outlet as per "Jock River Flood Risk Mapping (within the City of Ottawa) Hydraulics Report" (November 2004, PSR Group Ltd. and JFSA).*
  Van Gaal Drain water levels at SWM Facility 1 outlet as per HEC-RAS models from "Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond" (November 2009, JFSA) with proposed Fortune Street culvert improvements in place.

MAJOR SYSTEM CONVEYANCE

As per City standards, the total 100-year depth of water (static and dynamic) on the street must be retained within the right-of-way and should not exceed 30 cm. Although static ponding depths are unknown at this stage of the design, the dynamic flow depths at a typical low point were estimated in SWMHYMO based on the excess major system flows in a front yard / non-residential catchment and an assumed longitudinal street slope of 0.15% from high point to high point. The results of this analysis are presented in Table 2A of Attachment 3 for the 100-year 3-hour Chicago storm. As may be seen in Attachment 3, the dynamic flow depths on all catchments are below 30 cm, allowing for some static ponding depth to be incorporated into the detailed design without exceeding City standards for total depth of water. In general, it may be concluded that it is possible to provide a total 100-year depth of water that is less than 30 cm and retained within the right-of-way, in accordance with City standards.

Table 2B of Attachment 3 presents the simulated dynamic flow depths for the development based on a 20% increase in the 100-year 3-hour Chicago storm, in accordance with the climate change stress test prescribed in the January 2012 *City of Ottawa Technical Bulletin ISTB-2012-1*. As shown in Table 2B, the maximum dynamic flow depth at a typical low point was estimated as approximately 28.5 cm under these conditions.

STORMWATER MANAGEMENT FACILITIES

As previously noted, 99.82 ha of the subdivision at 51% imperviousness are serviced by SWM Facility 1; and 34.99 ha at 51% imperviousness are serviced by SWM Facility 2. Additionally, 97.5 ha of undeveloped lands to the south of the proposed subdivision will drain to SWM Facility 1, and 71.8 ha of undeveloped lands to the south of the proposed subdivision will drain to SWM Facility 2. Refer to Tables 3A and 3B of Attachment 4 for a summary of the proposed operating conditions for SWM Facilities 1 and 2, respectively.

The permanent pool volumes of SWM Facilities 1 and 2 are sufficient to provide an enhanced protection level (80% long-term suspended solids removal) according to Ministry of the Environment standards for wet ponds and wetlands, respectively. Active storage volumes of 40 m^3/ha minimum were also provided for quality control in SWM Facilities 1 and 2 and detained for approximately 24 hours. Drawdown time calculations for SWM Facilities 1 and 2 are presented in Tables 4A and 4B of Attachment 4.

Erosion control for SWM Facility 1 will be provided by controlling the 2-year release rate to 330 L/s or less, where 330 L/s is the erosion threshold for the Van Gaal Drain identifed by Parish Geomorphic in the *Natural Environment & Impact Assessment Study for the Mattamy Richmond Lands* (March 2009). Furthermore, the October 26, 2012 *Van Gaal Drain Erosion Assessment* memo by JTB Environmental Systems Inc. indicates that the 2-year outflows from SWM Facility 1 should discharge to the Van Gaal Drain at a velocity of 0.225 m/s or less. This may be achieved by a plunge pool or other velocity reduction measures at the SWM Facility 1 extended detention outlet pipe to the Van Gaal Drain. We understand that erosion control is not required for SWM Facility 2, which discharges to the Jock River.

Quantity control for SWM Facility 1 is to be provided by controlling post-development outflows to pre-development levels for the 2- to 100-year 24-hour SCS design storms, taking into account the uncontrolled major system flows from 30.79 ha and 21.75 ha of the SWM Facility 1 drainage area that discharge directly to the Van Gaal Drain and Moore Drain Tributary, respectively. Pre-development flows from the site were estimated in SWMHYMO based on the undeveloped drainage area characteristics presented in Figure 1A of Attachment 1 and Table 1B of Attachment 2. We understand that quantity control is not required for SWM Facility 2, which discharges to the Jock River; however, the 100-year release rate from SWM Facility 2 has been limited to a maximum of 2.235 m^3/s based on the capacity of the 1500 mm diameter (at 0.1% slope) outlet pipe to the Jock River. Refer to Tables 5A and 5B of Attachment 4 for the outlet control design and stage-storage-discharge relationships for SWM Facilities 1 and 2, respectively.

The performances of the SWM facilities were analyzed in XPSWMM based on both free outfall and restrictive downstream conditions. Restrictive downstream conditions for the outlet of SWM Facility 2 to the Jock River are based on the *Jock River Flood Risk Mapping (within the City of Ottawa) Hydraulics Report* (November 2004, PSR Group Ltd. and JFSA). During the 100-year 10-day spring snowmelt plus rainfall event, the water level on the Jock River at the outlet of SWM Facility 2 (Jock River Lower Reach 2 cross-section 19353) is 94.18 m. During the 100-year 24-hour SCS design storm, the water level on the Jock River at this location is below the permanent pool elevation of SWM Facility 2 and therefore does not affect the operation of the SWM facility.

Restrictive downstream conditions for the outlet of SWM Facility 1 to the Van Gaal Drain are based on the existing conditions HEC-RAS models from *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* (November 2009, JFSA) with proposed Fortune Street culvert improvements in place. It is proposed that the width of the Fortune Street culvert on the Van Gaal Drain be increased from 4.2 m to 6.3 m. Under these conditions, the water level on the Van Gaal Drain at the outlet of SWM Facility 1 (Van Gaal Drain Reach 2 cross-section 961) is 93.68 m for the 100-year 24-hour SCS design storm and 94.11 m for the 100-year 10-day spring snowmelt plus rainfall event.

HYDRAULIC GRADELINE ANALYSIS

The minor system and hydraulic gradeline analysis was completed for the proposed systems discharging to SWM Facility 1 and 2 using the XPSWMM program based on the 100-year 3-hour Chicago and 100-year 24-hour SCS design storms, the 100-year 10-day spring snowmelt plus rainfall event and for the July 1st 1979 historical event. Attachment 5 summarizes the hydraulic simulation results for the proposed systems under restrictive downstream conditions. Note that the flowing full pipe velocities are not less than 0.8 m/s and no greater than 6.0 m/s for all proposed pipes. Also note that all manholes where the 100-year hydraulic gradeline is less than 0.5 m below the ground elevation are located within the pond block, and do not have storm sewer connections to buildings; therefore a high 100-year hydraulic gradeline at these locations will not have any negative impacts.

Attachment 5 also presents the hydraulic simulation results for the climate change stress test based on a 20% increase in the 100-year 3-hour Chicago and 100-year 24-hour SCS design storms, as per the January 2012 *City of Ottawa Technical Bulletin ISTB-2012-1*.

DOWNSTREAM HYDRAULIC IMPACTS

The impact of the proposed subdivision and SWM facility designs on the flows and flood levels on the downstream Van Gaal Drain was evaluated using the existing conditions SWMHYMO and HEC-RAS models from *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* (November 2009, JFSA).

The SWMHYMO models from the November 2009 *Floodplain Mapping Report* were modified to include the proposed subdivision and SWM facility storage-discharge curves, including the . The resultant proposed conditions flows entered into the HEC-RAS models. The HEC-RAS models were also modified to include the proposed Fortune Street culvert improvements; as noted above, it is proposed that the width of the Fortune Street culvert on the Van Gaal Drain be increased from 4.2 m to 6.3 m.

As previously noted, 94.2 ha of undeveloped lands to the southwest of the proposed subdivision will be conveyed through the subject site by a tributary of the Moore Drain. The Moore Drain tributary is to be reconstructed in accordance with the channel profile and proposed culverts provided by DSEL. Furthermore, the Van Gaal Drain north of Perth Street, currently crossing through the proposed development lands, is to be realigned to follow the boundary of the subject site in accordance with the November 5, 2012 *Richmond Village Development / Proposed Realignment of Van Gaal Drain* memo by JFSA. The November 2009 SWMHYMO and HEC-RAS models have been modified to reflect these proposed conditions.

Attachment 6 presents a comparison of the existing and proposed conditions 100-year flows and water levels on the Van Gaal Drain. Three different scenarios are considered, as per the November 2009 report; (1) When the Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River; (2) When the Van Gaal Drain 100-year spring snowmelt plus rainfall peak flow reaches the Jock River; and (4) When the Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain. Existing and proposed conditions flows and water levels on the Van Gaal Drain during the 2-, 5-, 10- and 25-year events are summarized in Attachment 7.

Note that 100-year flows and water levels on the Van Gaal Drain generally decrease between existing and proposed conditions. This result is expected, as the provided release rates from SWM Facility 1 to the Van Gaal Drain are well below pre-development levels owing to the real-world limitations of the outlet controls. That is, the 45 m long quantity control weir for SWM Facility 1 is set 1.33 m above the permanent pool elevation at an invert of 93.68 m to match the 100-year 24-hour SCS water level on the Van Gaal Drain (per existing conditions with the Fortune Street culvert improvements in place). This results in minimal head over the quantity control weir and consequently lower release rates.

As may be seen from Attachment 6, proposed conditions 100-year peak flows on the Van Gaal Drain, Moore Drain and Joy’s Road Tributary are equal to or less than existing peak flows, and proposed conditions 100-year water levels are equal to or less than existing levels, except for two locations with minor 1 cm increases in water level (Van Gaal Drain cross-section 647 and Moore Drain cross-section 298). Note that proposed conditions 100-year water levels on the Moore Drain tributary though the subject site are higher than existing levels; this is not a concern since only the subject site is affected by this increase in water levels.

Yours truly,
J.F. Sabourin and Associates Inc.

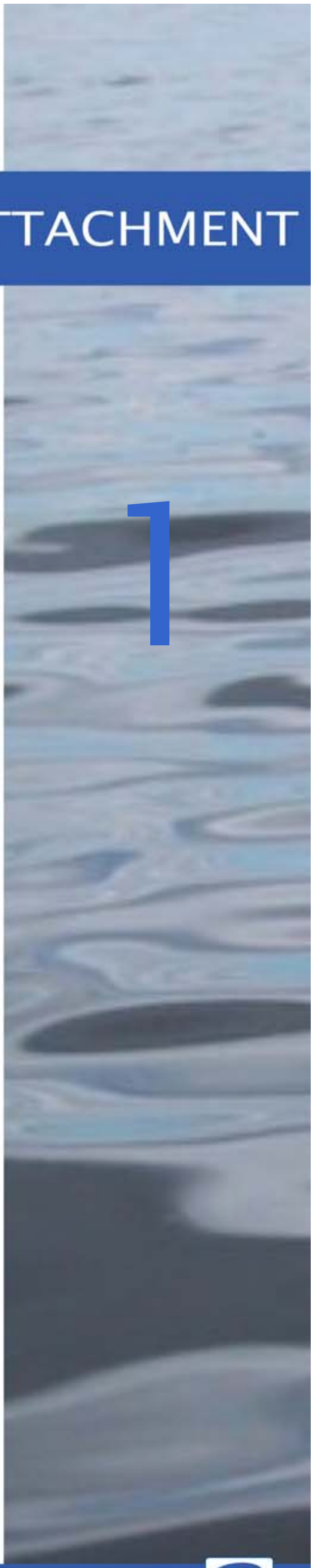
Laura Pipkins, P.Eng.

cc: J.F. Sabourin, M.Eng, P.Eng.
Director of Water Resources Projects

Attachment 1: Drainage Area to SWM Facilities
Attachment 2: Summary of Drainage Area to SWM Facilities; XPSWMM Model Schematic; and Minor System Loss Coefficients
Attachment 3: Major System Results
Attachment 4: SWM Facility Operating Conditions
Attachment 5: Pipe Data and Hydraulic Simulation Results
Attachment 6: 100-Year Flows and Water Levels on the Van Gaal Drain Under Existing and Proposed Conditions
Attachment 7: 2 to 25-Year Flows and Water Levels on the Van Gaal Drain Under Existing and Proposed Conditions

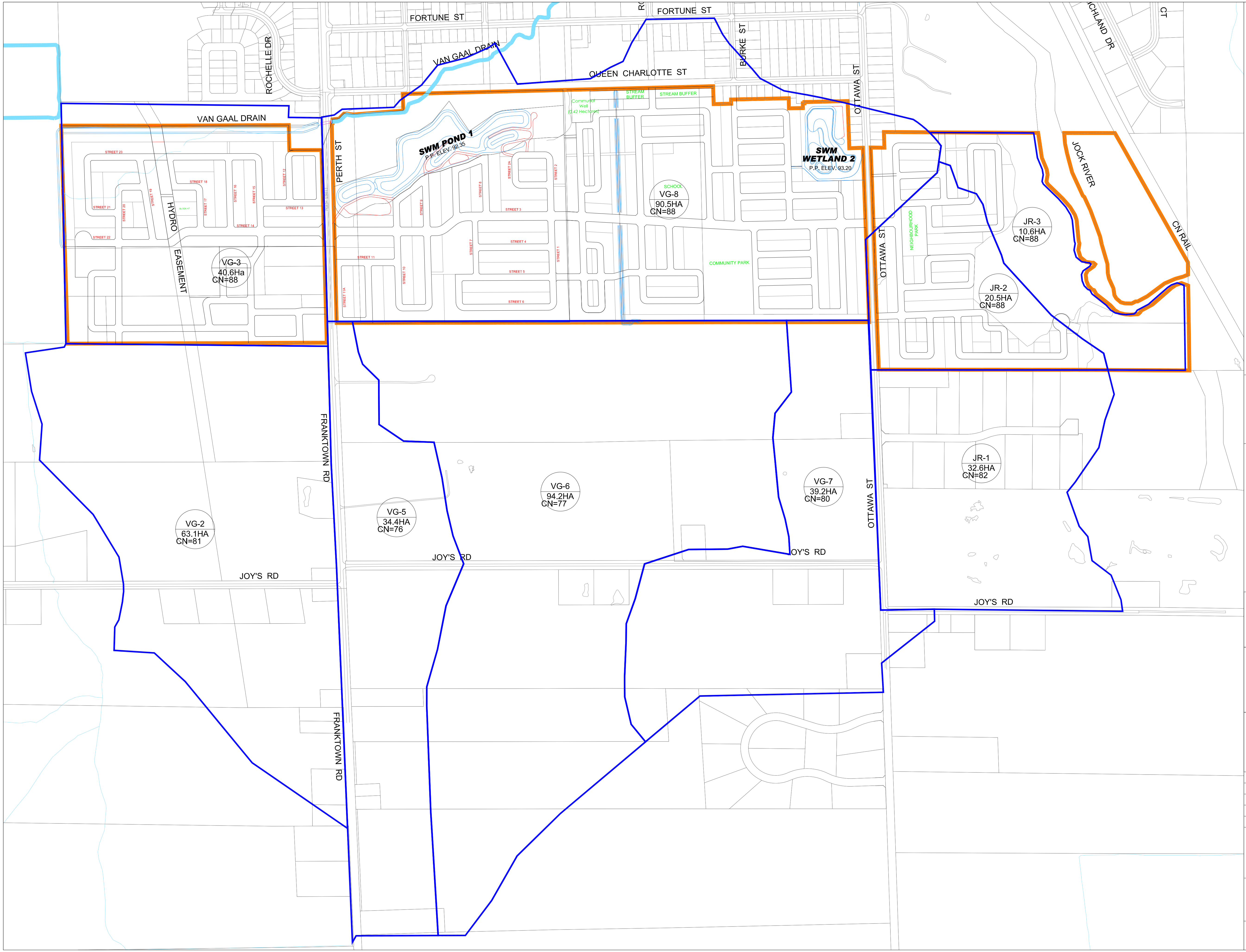
ATTACHMENT

1



DRAINAGE AREA TO SWM FACILITIES

JFSA Water Resources and Environmental Consultants



LEGEND :

SUBCATCHMENT BOUNDARY (PER DEC 2009 RICHMOND FLOODPLAIN STUDY)

JR-3 / 10.6HA / CN=88
- SUBCATCHMENT ID
- SUBCATCHMENT AREA
- CURVE NUMBER (CN)

SCALE :

0 100 200 300 400 500m

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WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS
OTTAWA (613) 836–3884
GATINEAU (819) 243–6858

CLIENT :

DSEL
david schaeffer engineering ltd
600 ALDEN ROAD., SUITE 500
MARKHAM, ONTARIO., L3R 0E7
(905) 475–3080

PROJECT :

RICHMOND VILLAGE (SOUTH) LIMITED SUBDIVISION

	BY	DATE	DESCRIPTION	BY

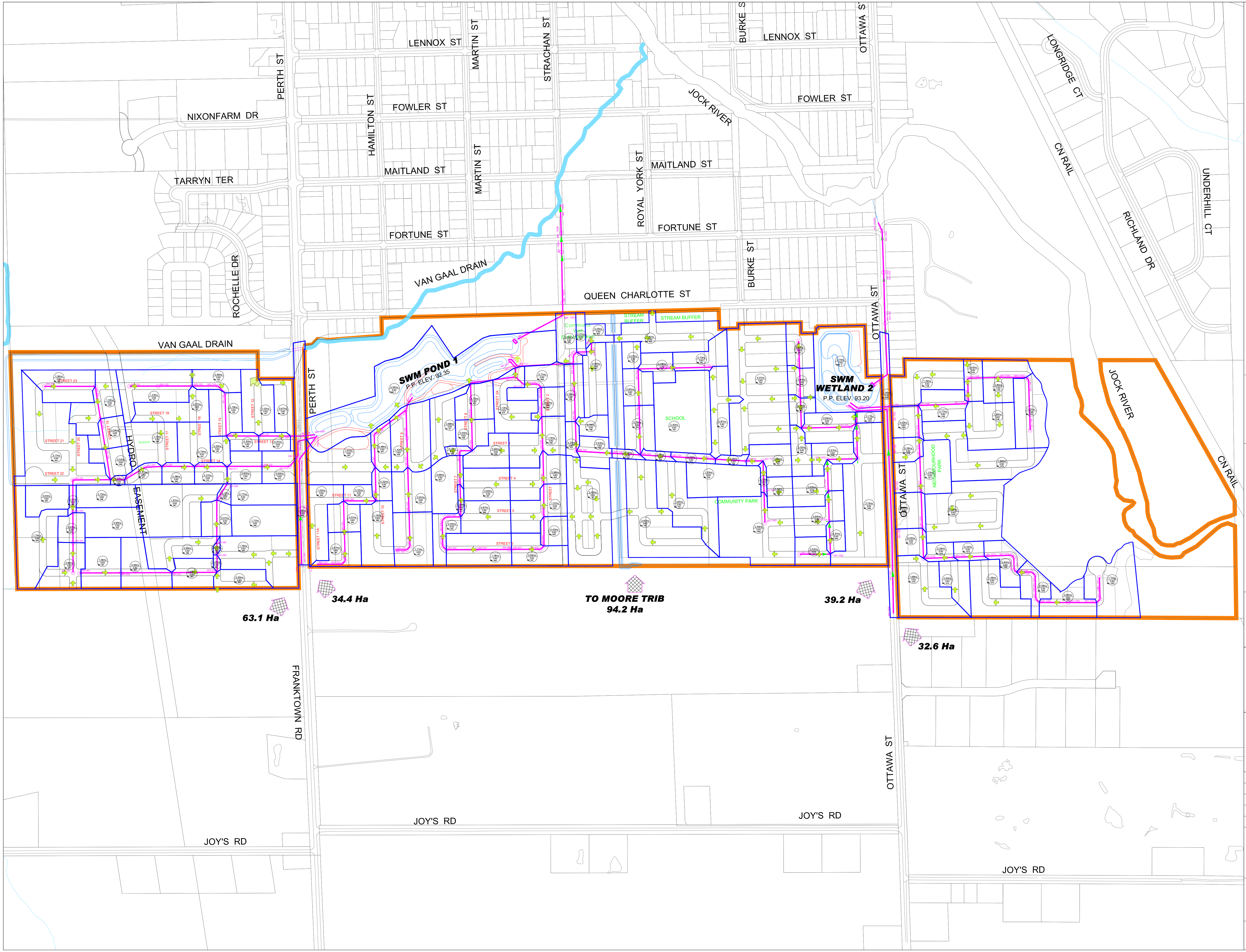
EXISTING DRAINAGE AREAS THROUGH SUBJECT SITE

FIGURE 1A

DESIGNED:
DRAWN: LP
VERIFIED: JFS
APPROVED: JFS

DRAWING REF.
922–11\201310 FSR\Design\CAD\JFSA Figures.dwg

DATE	PROJECT No.
Oct/13	922–11



LEGEND :

▬	SUBCATCHMENT BOUNDARY (PER DSEL)
1.35Ha 0.62 MH 1600	SUB-CATCHMENT AREA RUNOFF COEFFICIENT UPSTREAM MANHOLE
⇨	MAJOR SYSTEM FLOW DIRECTION
63.1 Ha ▨	UNDEVELOPED DRAINAGE AREA

SCALE :

0 100 200 300 400 500m

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PROJECT :

RICHMOND VILLAGE (SOUTH) LIMITED SUBDIVISION

	BY	DATE	DESCRIPTION	BY

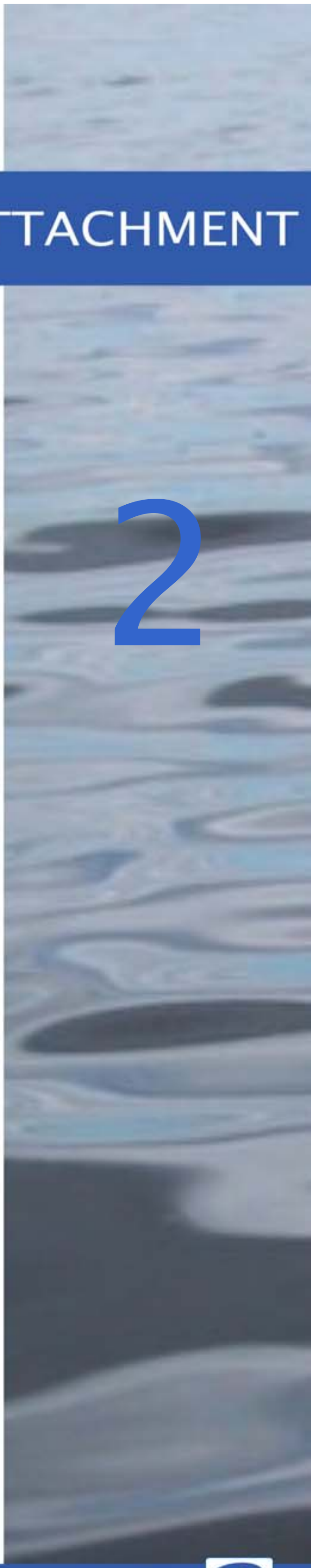
PROPOSED DRAINAGE AREA TO SWM FACILITIES

FIGURE 1B

DESIGNED:
DRAWN: LP
VERIFIED: JFS
APPROVED: JFS

DRAWING REF.
922-11\201310 FSR\Design\CAD\
JFSA Figures.dwg

DATE	PROJECT No.
Oct/13	922-11



ATTACHMENT

2

SUMMARY OF DRAINAGE AREA TO SWM FACILITIES

XPSWMM MODEL SCHEMATIC

MINOR SYSTEM LOSS COEFFICIENTS

JFSA Water Resources and Environmental Consultants

Table 1A: Proposed Subdivision Drainage Area to SWM Facilities 1 and 2

MH	SWMHYMO ID	Total Area (ha)	Front Yard / Non-Resid. (ha)	Rear Yard (1) (ha)	Runoff Coefficient	Front Yard / Non-Residential Area Parameters							Sump Pump Flow (2) (m^3/s)
						Imperviousness (%)	Storage (m^3)	5-Year Flow (m^3/s)	Minor Capture (m^3/s)	Road Width (m)	Length (m)	Downstream Segment	
101	101a	1.43	0.894	0.536	0.62	60	26.82	0.155	0.174	8.5	219	VGaal	0.00033
103	103a	0.56	0.455	0.105	0.62	60	13.65	0.081	0.091	8.5	87	105a	0.00013
103	103b	0.97	0.606	0.364	0.62	60	18.18	0.107	0.120	8.5	85	VGaal	0.00022
105	105a	0.78	0.487	0.293	0.62	60	14.61	0.086	0.096	8.5	101	106b	0.00018
106	106a	0.29	0.181	0.109	0.62	60	5.43	0.032	0.036	8.5	71	106b	0.00007
106	106b	0.30	0.244	0.056	0.62	60	7.32	0.043	0.048	8.5	77	107b	0.00007
107	107a	0.31	0.252	0.058	0.62	60	7.56	0.045	0.050	8.5	75	1500a	0.00007
107	107b	1.02	0.637	0.383	0.62	60	19.11	0.112	0.125	8.5	128	VGaal	0.00023
108	108a	0.67	0.544	0.126	0.62	60	16.32	0.096	0.108	8.5	186	1500a	0.00015
202	202a	1.06	0.662	0.398	0.62	60	19.86	0.116	0.130	8.5	136	203a	0.00024
203	203a	1.10	0.687	0.413	0.62	60	20.61	0.120	0.134	8.5	77	204a	0.00025
204	204a	0.54	0.337	0.203	0.62	60	10.11	0.060	0.067	8.5	72	205a	0.00012
205	205a	0.52	0.325	0.195	0.62	60	9.75	0.058	0.065	8.5	72	206a	0.00012
206	206a	0.07	0.070	0.000	0.62	60	2.10	0.013	0.015	8.5	34	207a	0.00002
206	206b	0.75	0.469	0.281	0.62	60	14.07	0.083	0.093	8.5	53	207a	0.00017
207	207a	0.37	0.301	0.069	0.62	60	9.03	0.054	0.060	8.5	95	209a	0.00009
209	209a	0.38	0.309	0.071	0.62	60	9.27	0.055	0.062	8.5	77	210a	0.00009
209	209b	1.15	0.719	0.431	0.62	60	21.57	0.126	0.141	8.5	104	209a	0.00026
210	210a	0.37	0.301	0.069	0.62	60	9.03	0.054	0.060	8.5	77	106a	0.00009
210	210b	0.74	0.462	0.278	0.62	60	13.86	0.082	0.092	8.5	106	210a	0.00017
211	211a	0.60	0.375	0.225	0.62	60	11.25	0.067	0.075	8.5	82	106a	0.00014
212	212a	0.60	0.375	0.225	0.62	60	11.25	0.067	0.075	8.5	78	1500a	0.00014
251	251a	0.86	0.537	0.323	0.62	60	16.11	0.095	0.106	8.5	137	252a	0.00020
252	252a	0.42	0.262	0.158	0.62	60	7.86	0.047	0.053	8.5	70	253a	0.00010
253	253a	0.11	0.089	0.021	0.62	60	2.67	0.016	0.018	8.5	64	206a	0.00003
253	253b	0.95	0.594	0.356	0.62	60	17.82	0.105	0.118	8.5	70	253a	0.00022
253	253c	1.74	1.087	0.653	0.62	60	32.61	0.187	0.209	8.5	194	253a	0.00040
254	254a	0.64	0.400	0.240	0.62	60	12.00	0.071	0.080	8.5	56	206a	0.00015
254	254b	3.17	1.981	1.189	0.62	60	59.43	0.333	0.373	8.5	375	206a	0.00073
261	261a	1.06	0.662	0.398	0.62	60	19.86	0.116	0.130	8.5	94	262a	0.00024
262	262a	0.99	0.619	0.371	0.62	60	18.57	0.109	0.122	8.5	119	263a	0.00023
263	263a	0.68	0.425	0.255	0.62	60	12.75	0.075	0.084	8.5	106	264a	0.00016
264	264a	0.05	0.050	0.000	0.62	60	1.50	0.009	0.010	8.5	41	265a	0.00001
264	264b	0.55	0.447	0.103	0.62	60	13.41	0.079	0.088	8.5	151	264a	0.00013
265	265a	0.05	0.050	0.000	0.62	60	1.50	0.009	0.010	8.5	31	266a	0.00001
265	265b	1.76	1.100	0.660	0.62	60	33.00	0.189	0.212	8.5	191	265a	0.00040
266	266a	0.57	0.356	0.214	0.62	60	10.68	0.063	0.071	8.5	85	267a	0.00013
266	266b	0.99	0.619	0.371	0.62	60	18.57	0.109	0.122	8.5	182	266a	0.00023
266	266c	1.11	0.694	0.416	0.62	60	20.82	0.122	0.137	8.5	73	267a	0.00026
267	267a	0.12	0.120	0.000	0.62	60	3.60	0.021	0.024	8.5	45	106a	0.00003

Table 1A: Proposed Subdivision Drainage Area to SWM Facilities 1 and 2

MH	SWMHYMO ID	Total Area (ha)	Front Yard / Non-Resid. (ha)	Rear Yard [1] (ha)	Runoff Coefficient	Front Yard / Non-Residential Area Parameters							Sump Pump Flow [2] (m^3/s)
						Imperviousness (%)	Storage (m^3)	5-Year Flow (m^3/s)	Minor Capture (m^3/s)	Road Width (m)	Length (m)	Downstream Segment	
267	267b	0.77	0.481	0.289	0.62	60	14.43	0.085	0.095	8.5	50	1500a	0.00018
267	267c	1.97	1.231	0.739	0.62	60	36.93	0.211	0.236	8.5	158	267a	0.00045
301	301a	0.67	0.419	0.251	0.62	60	12.57	0.074	0.083	8.5	146	304a	0.00015
304	304a	0.51	0.319	0.191	0.62	60	9.57	0.057	0.064	8.5	74	305a	0.00012
305	305a	0.32	0.260	0.060	0.62	60	7.80	0.046	0.052	8.5	76	306a	0.00007
305	305b	1.00	0.812	0.188	0.62	60	24.36	0.141	0.158	8.5	213	305a	0.00023
306	306a	0.32	0.260	0.060	0.62	60	7.80	0.046	0.052	8.5	80	307a	0.00007
306	306b	1.16	0.725	0.435	0.62	60	21.75	0.127	0.142	8.5	148	306a	0.00027
307	307a	0.48	0.390	0.090	0.62	60	11.70	0.069	0.077	8.5	124	Pond1a	0.00011
307	307b	1.51	0.944	0.566	0.62	60	28.32	0.163	0.183	8.5	160	307a	0.00035
401	401a	1.17	0.731	0.439	0.62	60	21.93	0.128	0.143	8.5	121	404a	0.00027
404	404a	0.91	0.569	0.341	0.62	60	17.07	0.100	0.112	8.5	106	405a	0.00021
405	405a	1.42	0.887	0.533	0.62	60	26.61	0.154	0.172	8.5	146	Pond1a	0.00033
500	500a	1.18	0.737	0.443	0.62	60	22.11	0.129	0.144	8.5	153	502a	0.00027
502	502a	0.55	0.447	0.103	0.62	60	13.41	0.079	0.088	8.5	118	506a	0.00013
506	506a	0.09	0.090	0.000	0.62	60	2.70	0.016	0.018	8.5	76	507b	0.00002
506	506b	0.87	0.544	0.326	0.62	60	16.32	0.096	0.108	8.5	117	506a	0.00020
507	507a	0.83	0.519	0.311	0.62	60	15.57	0.092	0.103	8.5	109	507b	0.00019
507	507b	0.89	0.723	0.167	0.62	60	21.69	0.127	0.142	8.5	69	509a	0.00020
508	508a	0.12	0.120	0.000	0.62	60	3.60	0.021	0.024	8.5	61	509a	0.00003
509	509a	0.54	0.337	0.203	0.62	60	10.11	0.060	0.067	8.5	73	510a	0.00012
509	509b	0.66	0.412	0.248	0.62	60	12.36	0.073	0.082	8.5	86	509a	0.00015
510	510a	0.70	0.437	0.263	0.62	60	13.11	0.077	0.086	8.5	84	Pond1a	0.00016
511	511a	0.88	0.550	0.330	0.62	60	16.50	0.097	0.109	8.5	223	Pond1a	0.00020
601	601a	0.84	0.525	0.315	0.62	60	15.75	0.093	0.104	8.5	79	603a	0.00019
603	603a	0.43	0.430	0.000	0.62	60	12.90	0.076	0.085	8.5	69	604a	0.00010
604	604a	1.01	0.631	0.379	0.62	60	18.93	0.111	0.124	8.5	76	605a	0.00023
605	605a	0.98	0.612	0.368	0.62	60	18.36	0.108	0.121	8.5	79	606a	0.00023
606	606a	1.14	0.712	0.428	0.62	60	21.36	0.125	0.140	8.5	78	607a	0.00026
607	607a	0.47	0.382	0.088	0.62	60	11.46	0.068	0.076	8.5	68	609a	0.00011
609	609a	0.71	0.444	0.266	0.62	60	13.32	0.079	0.088	8.5	42	Pond1a	0.00016
610	610a	1.13	0.706	0.424	0.62	60	21.18	0.124	0.139	8.5	179	Pond1a	0.00026
700	700a	0.10	0.100	0.000	0.62	60	3.00	0.018	0.020	8.5	74	702a	0.00002
700	700b	0.43	0.269	0.161	0.62	60	8.07	0.048	0.054	8.5	61	700a	0.00010
701	701a	0.66	0.412	0.248	0.62	60	12.36	0.073	0.082	8.5	108	702a	0.00015
702	702a	0.16	0.160	0.000	0.62	60	4.80	0.029	0.032	8.5	71	703a	0.00004
702	702b	4.01	2.506	1.504	0.62	60	75.18	0.410	0.459	8.5	377	702a	0.00092
703	703a	0.22	0.220	0.000	0.62	60	6.60	0.039	0.044	8.5	105	704a	0.00005
703	703b	0.97	0.788	0.182	0.62	60	23.64	0.137	0.153	8.5	215	703a	0.00022
703	703c	1.33	1.330	0.000	0.62	60	39.90	0.228	0.255	8.5	125	703a	0.00031

Table 1A: Proposed Subdivision Drainage Area to SWM Facilities 1 and 2

MH	SWMHYMO ID	Total Area (ha)	Front Yard / Non-Resid. (ha)	Rear Yard (1) (ha)	Runoff Coefficient	Front Yard / Non-Residential Area Parameters							Sump Pump Flow (2) (m^3/s)
						Imperviousness (%)	Storage (m^3)	5-Year Flow (m^3/s)	Minor Capture (m^3/s)	Road Width (m)	Length (m)	Downstream Segment	
704	704a	0.16	0.160	0.000	0.62	60	4.80	0.029	0.032	8.5	72	705a	0.00004
704	704b	1.65	1.031	0.619	0.62	60	30.93	0.178	0.199	8.5	260	704a	0.00038
704	704c	2.40	2.400	0.000	0.62	60	72.00	0.394	0.441	8.5	163	704a	0.00055
705	705a	0.15	0.150	0.000	0.62	60	4.50	0.027	0.030	8.5	67	MTrib	0.00003
705	705b	2.60	1.625	0.975	0.62	60	48.75	0.275	0.308	8.5	332	705a	0.00060
706	706a	0.53	0.331	0.199	0.62	60	9.93	0.059	0.066	8.5	85	606a	0.00012
707	707a	0.25	0.203	0.047	0.62	60	6.09	0.036	0.040	8.5	69	606a	0.00006
707	707b	2.46	1.537	0.923	0.62	60	46.11	0.261	0.292	8.5	237	604a	0.00057
708	708a	0.34	0.276	0.064	0.62	60	8.28	0.049	0.055	8.5	54	708b	0.00008
708	708b	0.59	0.369	0.221	0.62	60	11.07	0.065	0.073	8.5	135	Pond1a	0.00014
751	751a	0.70	0.569	0.131	0.62	60	17.07	0.100	0.112	8.5	156	752a	0.00016
752	752a	0.48	0.300	0.180	0.62	60	9.00	0.053	0.059	8.5	83	753a	0.00011
753	753a	0.27	0.219	0.051	0.62	60	6.57	0.039	0.044	8.5	63	MTrib	0.00006
753	753b	0.60	0.375	0.225	0.62	60	11.25	0.067	0.075	8.5	88	753a	0.00014
753	753c	1.51	0.944	0.566	0.62	60	28.32	0.163	0.183	8.5	231	MTrib	0.00035
754	754a	0.76	0.475	0.285	0.62	60	14.25	0.084	0.094	8.5	115	753b	0.00017
806	806a	0.25	0.203	0.047	0.62	60	6.09	0.036	0.040	8.5	59	807a	0.00006
807	807a	0.21	0.171	0.039	0.62	60	5.13	0.031	0.035	8.5	23	808a	0.00005
807	807b	0.92	0.575	0.345	0.62	60	17.25	0.101	0.113	8.5	150	807a	0.00021
808	808a	0.59	0.369	0.221	0.62	60	11.07	0.065	0.073	8.5	32	Pond1a	0.00014
901	901a	0.60	0.375	0.225	0.62	60	11.25	0.067	0.075	8.5	96	902a	0.00014
902	902a	0.44	0.275	0.165	0.62	60	8.25	0.049	0.055	8.5	70	Pond2a	0.00010
903	903a	0.34	0.212	0.128	0.62	60	6.36	0.038	0.043	8.5	71	Pond2a	0.00008
904	904a	0.50	0.312	0.188	0.62	60	9.36	0.055	0.062	8.5	99	903a	0.00012
904	904b	0.84	0.525	0.315	0.62	60	15.75	0.093	0.104	8.5	127	Pond2a	0.00019
1001	1001a	0.53	0.331	0.199	0.62	60	9.93	0.059	0.066	8.5	83	1002a	0.00012
1002	1002a	0.31	0.252	0.058	0.62	60	7.56	0.045	0.050	8.5	77	1003a	0.00007
1003	1003a	0.84	0.525	0.315	0.62	60	15.75	0.093	0.104	8.5	71	1004a	0.00019
1004	1004a	0.83	0.519	0.311	0.62	60	15.57	0.092	0.103	8.5	74	1005a	0.00019
1005	1005a	0.57	0.463	0.107	0.62	60	13.89	0.082	0.092	8.5	71	Pond2a	0.00013
1006	1006a	3.15	1.969	1.181	0.62	60	59.07	0.331	0.371	8.5	375	Pond2a	0.00072
1007	1007a	0.17	0.138	0.032	0.62	60	4.14	0.025	0.028	8.5	26	Pond2a	0.00004
1101	1101a	1.21	0.756	0.454	0.62	60	22.68	0.132	0.148	8.5	169	1103a	0.00028
1103	1103a	0.89	0.556	0.334	0.62	60	16.68	0.098	0.110	8.5	133	1106b	0.00020
1104	1104a	0.43	0.349	0.081	0.62	60	10.47	0.062	0.069	8.5	99	1106a	0.00010
1106	1106a	0.77	0.481	0.289	0.62	60	14.43	0.085	0.095	8.5	112	1600a	0.00018
1106	1106b	1.26	0.787	0.473	0.62	60	23.61	0.137	0.153	8.5	209	1219a	0.00029
1108	1108a	1.39	0.869	0.521	0.62	60	26.07	0.151	0.169	8.5	179	1600a	0.00032
1201	1201a	1.71	1.069	0.641	0.62	60	32.07	0.184	0.206	8.5	112	1203a	0.00039
1203	1203a	0.85	0.531	0.319	0.62	60	15.93	0.094	0.105	8.5	135	1205a	0.00020

Table 1A: Proposed Subdivision Drainage Area to SWM Facilities 1 and 2

MH	SWMHYMO ID	Total Area (ha)	Front Yard / Non-Resid. (ha)	Rear Yard (1) (ha)	Runoff Coefficient	Front Yard / Non-Residential Area Parameters							Sump Pump Flow (2) (m^3/s)
						Imperviousness (%)	Storage (m^3)	5-Year Flow (m^3/s)	Minor Capture (m^3/s)	Road Width (m)	Length (m)	Downstream Segment	
1205	1205a	0.58	0.362	0.218	0.62	60	10.86	0.064	0.072	8.5	117	1209a	0.00013
1209	1209a	0.64	0.400	0.240	0.62	60	12.00	0.071	0.080	8.5	108	1213a	0.00015
1213	1213a	0.32	0.260	0.060	0.62	60	7.80	0.046	0.052	8.5	71	1214a	0.00007
1213	1213b	1.39	0.869	0.521	0.62	60	26.07	0.151	0.169	8.5	202	1213a	0.00032
1214	1214a	0.29	0.236	0.054	0.62	60	7.08	0.042	0.047	8.5	71	1215a	0.00007
1214	1214b	0.65	0.406	0.244	0.62	60	12.18	0.072	0.081	8.5	93	1214a	0.00015
1215	1215a	0.45	0.366	0.084	0.62	60	10.98	0.065	0.073	8.5	71	1216a	0.00010
1215	1215b	1.68	1.050	0.630	0.62	60	31.50	0.181	0.203	8.5	169	1215a	0.00039
1216	1216a	0.61	0.496	0.114	0.62	60	14.88	0.088	0.099	8.5	85	1217a	0.00014
1216	1216b	2.44	1.525	0.915	0.62	60	45.75	0.259	0.290	8.5	227	1216a	0.00056
1217	1217a	1.33	1.081	0.249	0.62	60	32.43	0.186	0.208	8.5	82	1218a	0.00031
1218	1218a	0.39	0.317	0.073	0.62	60	9.51	0.056	0.063	8.5	47	1219a	0.00009
1219	1219a	0.53	0.431	0.099	0.62	60	12.93	0.076	0.085	8.5	74	1600a	0.00012
1219	1219b	1.96	1.225	0.735	0.62	60	36.75	0.210	0.235	8.5	350	1219a	0.00045
1500 (3)	1500a	1.36	1.360	0.000	0.62	60	40.80	0.232	100% Capt.	8.5	287	N/A	0.00031
1600 (3)	1600a	1.35	1.350	0.000	0.62	60	40.50	0.231	100% Capt.	8.5	489	N/A	0.00031
7000	7000a	2.59	2.104	0.486	0.62	60	63.12	0.352	0.394	8.5	213	700a	0.00060
Pond1	Pond1a	5.94	5.940	0.000	0.62	60	N/A	0.906	N/A	N/A	N/A	N/A	0.00000
Pond2	Pond2a	2.75	2.750	0.000	0.62	60	N/A	0.447	N/A	N/A	N/A	N/A	0.00000
		126.810	**89.059**	**37.751**									

(1) Rear yard imperviousness is equal to half of the front yard imperviousness, and half of that rear yard impervious area is assumed to be indirectly connected. 100% of the 100-year flows generated on the rear yards are captured to the minor system.

(2) 0.23 L/s/ha based on approximately 1.44 m^3/day/lot (per October 3, 2012 "*Updated Assessment of Subsurface Drainage and Analysis of 100 Year Flood Event - Proposed Village of Richmond Development* " by Golder Associates Limited), 27.8 lots/ha, and 50% of sump pumps on at any given time.

(3) 100% capture of the 100-year flows on Ottawa Street and Perth Street to prevent flow across roads.

Table 1B: Summary of Study Area under Existing Conditions (Drains to SWM Facilities 1 and 2 in Future)

Area ID	VG-2	VG-3 (1)	VG-5	VG-7	VG-8 (1)	VG-8 (1)	JR-1	JR-2 (1)	JR-3 (1)
In future drains to:	Pond 1	Pond 1	Pond 1	Pond 2	Pond 1	Pond 2	Pond 2	Pond 2	Pond 2
A (ha)	63.1	33.327	34.4	39.2	58.393	15.510	32.6	15.747	3.833
CN (-)	81	88	76	80	88	88	82	88	88
Ia	2.8	2.5	3.0	3.5	2.6	2.6	3.5	2.5	2.5
L (m)	1220	630	1540	1520	1160	420	790	330	150
S (%)	0.4	0.2	0.4	0.2	0.2	0.2	0.2	0.1	0.6
Tc (min)	147.6	96.5	207.1	256.9	157.3	69.8	142.8	81.3	17.7
Tp (hrs)	1.6	1.1	2.3	2.9	1.7	0.8	1.6	0.9	0.2

(1) Proposed subdivision drainage area.

(2) Characteristics of undeveloped lands as per *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* dated November 2009 by JFSA.

Table 1C: Summary of Study Area under Proposed Conditions (Drains to SWM Facilities 1 and 2)

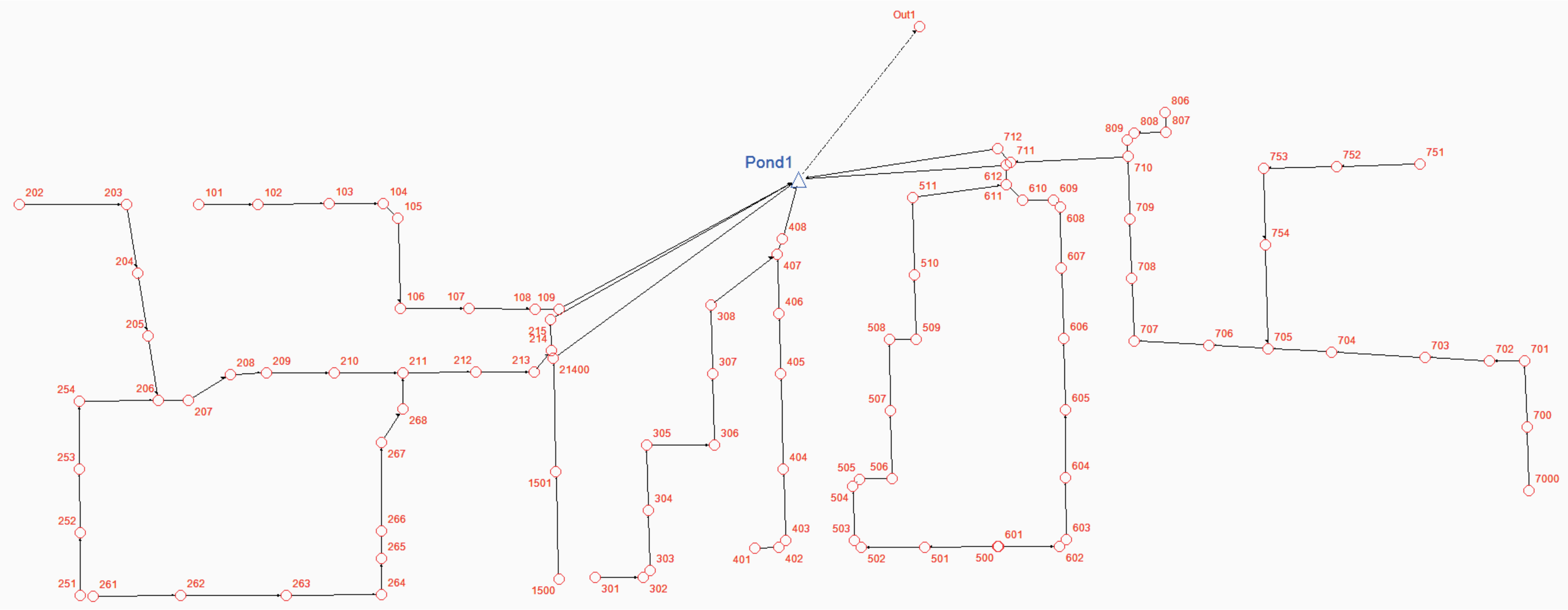
Area ID	VG-2	VG-5	VG-7	JR-1	Pond 1 (1)	Pond 2 (1)
Drains to:	Pond 1	Pond 1	Pond 2	Pond 2	Pond 1	Pond 2
A (ha)	63.1	34.4	39.2	32.6	91.82	34.99
CN (-)	81	76	80	82	N/A	N/A
Ia	2.8	3.0	3.5	3.5	N/A	N/A
L (m)	1220	1540	1520	790	N/A	N/A
S (%)	0.4	0.4	0.2	0.2	0.5	0.5
Tc (min)	147.6	207.1	256.9	142.8	N/A	N/A
Tp (hrs)	1.6	2.3	2.9	1.6	N/A	N/A
TIMP	N/A	N/A	N/A	N/A	51.1	50.9
XIMP	N/A	N/A	N/A	N/A	46.7	46.4

(1) Proposed subdivision drainage area; major system flows from 30.79 ha of the Pond 1 subdivision area discharge directly to the Van Gaal Drain, and from 21.75 ha to the Moore Drain Tributary.

(2) Characteristics of undeveloped lands as per *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* dated November 2009 by JFSA.

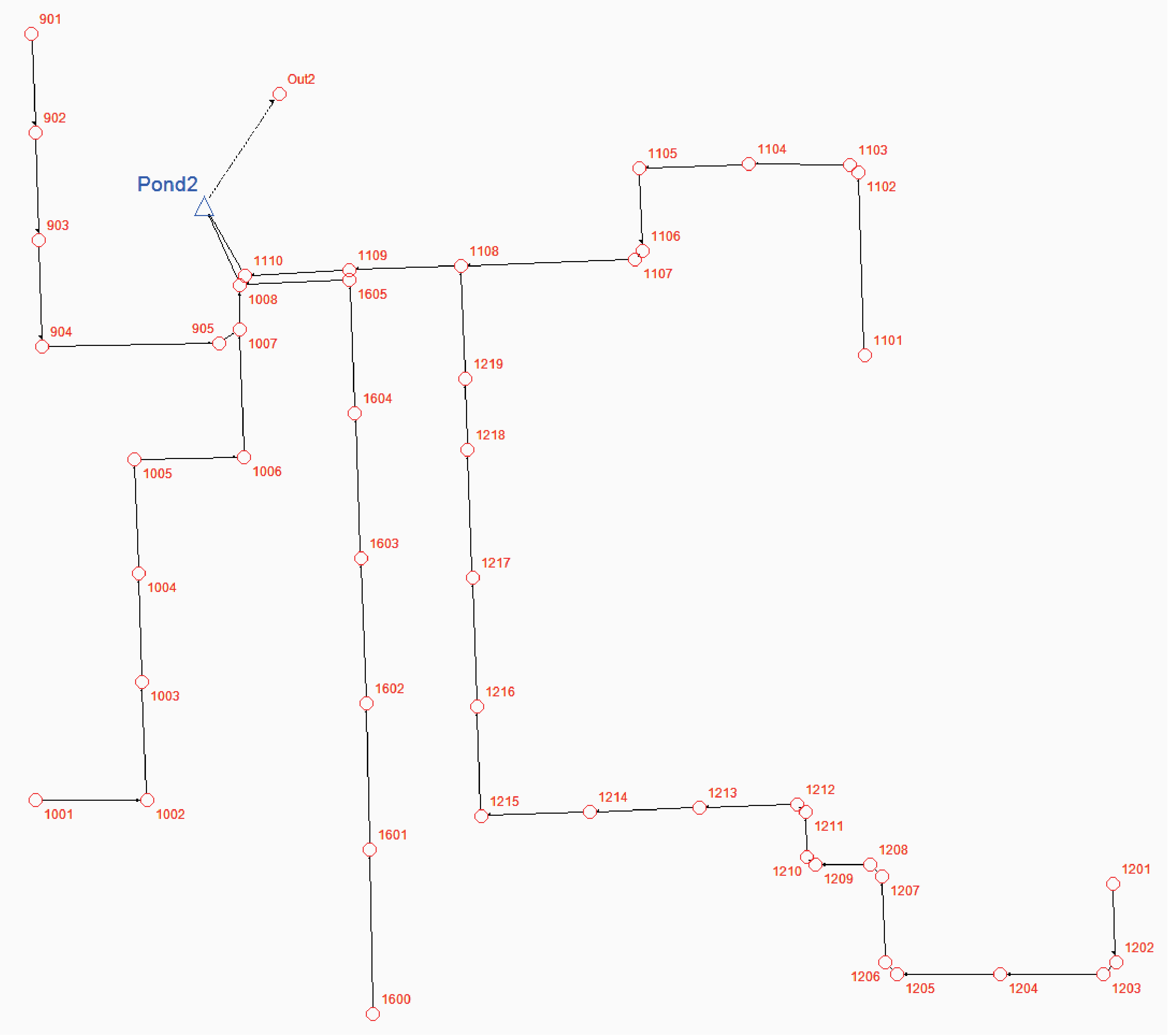
XPSWMM MODEL SCHEMATIC - SWMF 1

JFSA Water Resources and Environmental Consultants



XPSWMM MODEL SCHEMATIC - SWMF 2

JFSA Water Resources and Environmental Consultants



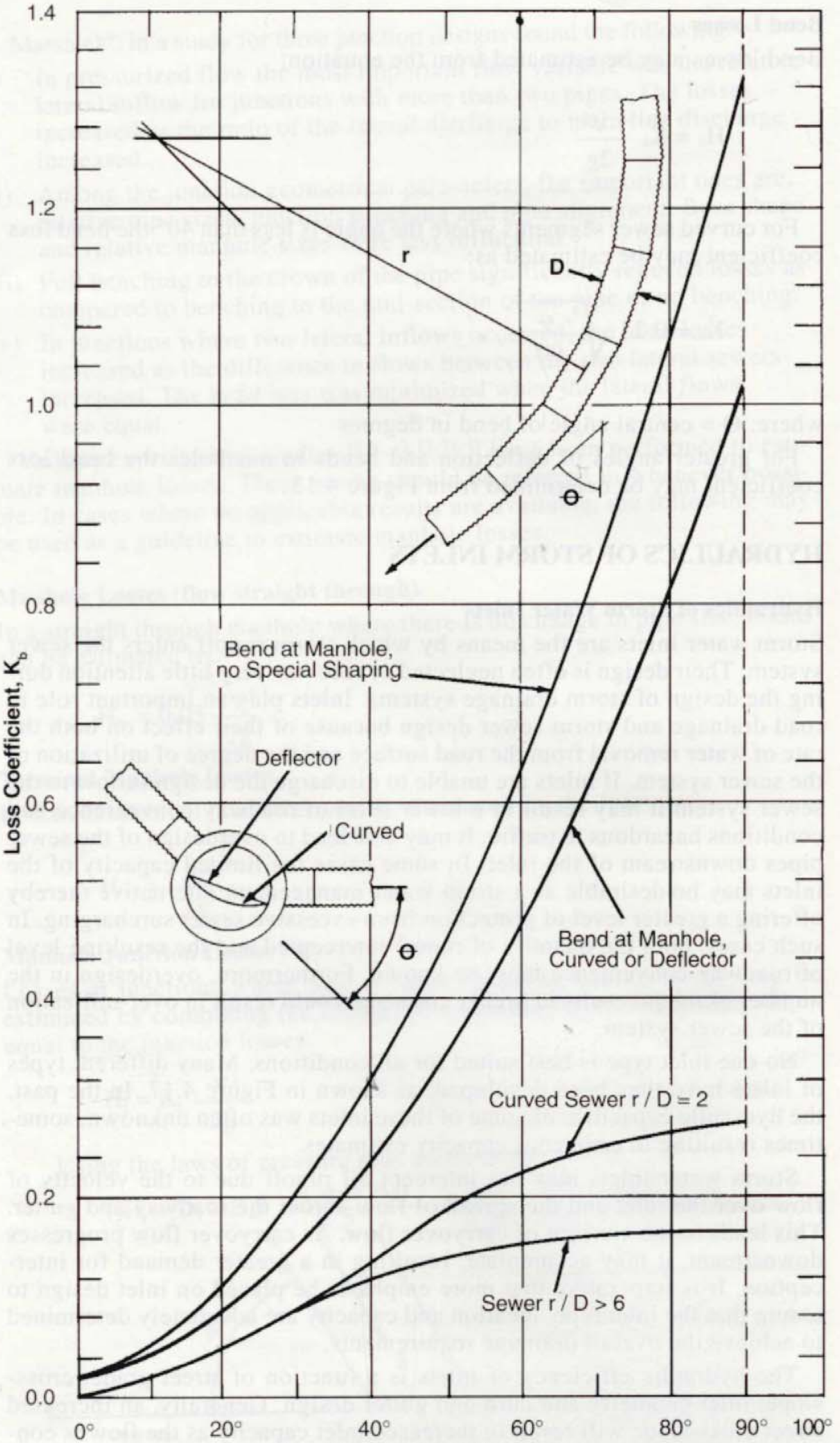


Figure 4.13 Sewer bend loss coefficient[16]

Manhole Loss Coefficient as per Nomograph

Angle (^{o})	MH Loss Coef
0	0.02
5	0.035
10	0.055
15	0.08
20	0.11
25	0.16
30	0.21
35	0.26
40	0.32
45	0.39
50	0.47
55	0.54
60	0.635
65	0.73
70	0.84
75	0.95
80	1.07
85	1.19
90	1.33

ATTACHMENT

3

MAJOR SYSTEM RESULTS

JFSA Water Resources and Environmental Consultants

Table 2A: Major System Results for the 100-Year, 3-Hour Chicago Storm

MH	SWMHYMO ID	Flow (m^3/s)	Flow Depth (m)	Velocity (m/s)	V*D (m^2/s)
101	101a	0.295	0.133	0.500	0.067
103	103a	0.152	0.104	0.414	0.043
103	103b	0.202	0.115	0.443	0.051
105	105a	0.162	0.107	0.422	0.045
106	106a	0.639	0.174	0.666	0.116
106	106b	0.640	0.173	0.663	0.115
107	107a	0.086	0.084	0.355	0.030
107	107b	0.684	0.178	0.682	0.121
108	108a	0.238	0.122	0.463	0.056
202	202a	0.220	0.119	0.453	0.054
203	203a	0.325	0.137	0.514	0.070
204	204a	0.233	0.121	0.462	0.056
205	205a	0.206	0.115	0.443	0.051
206	206a	0.679	0.178	0.680	0.121
206	206b	0.156	0.106	0.419	0.044
207	207a	0.673	0.177	0.678	0.120
209	209a	0.565	0.165	0.636	0.105
209	209b	0.239	0.122	0.464	0.057
210	210a	0.499	0.159	0.616	0.098
210	210b	0.154	0.105	0.418	0.044
211	211a	0.208	0.116	0.447	0.052
212	212a	0.127	0.097	0.391	0.038
251	251a	0.179	0.110	0.430	0.047
252	252a	0.160	0.106	0.420	0.045
253	253a	0.354	0.140	0.529	0.074
253	253b	0.198	0.114	0.441	0.050
253	253c	0.359	0.141	0.532	0.075
254	254a	0.136	0.100	0.399	0.040
254	254b	0.634	0.172	0.660	0.114
261	261a	0.220	0.119	0.453	0.054
262	262a	0.303	0.134	0.503	0.067
263	263a	0.245	0.124	0.469	0.058
264	264a	0.165	0.107	0.423	0.045
264	264b	0.149	0.104	0.413	0.043
265	265a	0.281	0.130	0.489	0.064
265	265b	0.363	0.142	0.534	0.076
266	266a	0.415	0.148	0.561	0.083
266	266b	0.206	0.116	0.446	0.052
266	266c	0.230	0.121	0.460	0.056
267	267a	0.443	0.151	0.578	0.087
267	267b	0.160	0.107	0.421	0.045
267	267c	0.405	0.147	0.559	0.082
301	301a	0.142	0.101	0.404	0.041
304	304a	0.164	0.107	0.423	0.045
305	305a	0.270	0.129	0.485	0.063
305	305b	0.270	0.128	0.483	0.062
306	306a	0.287	0.132	0.495	0.065
306	306b	0.241	0.122	0.465	0.057
307	307a	0.373	0.143	0.540	0.077
307	307b	0.313	0.135	0.508	0.069
401	401a	0.243	0.123	0.466	0.057
404	404a	0.297	0.133	0.500	0.067
405	405a	0.389	0.145	0.550	0.080

Table 2A: Major System Results for the 100-Year, 3-Hour Chicago Storm

MH	SWMHYMO ID	Flow (m^3/s)	Flow Depth (m)	Velocity (m/s)	V*D (m^2/s)
500	500a	0.245	0.123	0.467	0.057
502	502a	0.256	0.126	0.474	0.060
506	506a	0.185	0.111	0.433	0.048
506	506b	0.181	0.111	0.432	0.048
507	507a	0.173	0.109	0.427	0.047
507	507b	0.408	0.147	0.557	0.082
508	508a	0.041	0.062	0.287	0.018
509	509a	0.361	0.142	0.534	0.076
509	509b	0.140	0.100	0.402	0.040
510	510a	0.343	0.139	0.523	0.073
511	511a	0.183	0.111	0.433	0.048
601	601a	0.175	0.110	0.428	0.047
603	603a	0.212	0.117	0.449	0.053
604	604a	0.533	0.162	0.625	0.101
605	605a	0.422	0.149	0.566	0.084
606	606a	0.458	0.153	0.587	0.090
607	607a	0.360	0.141	0.532	0.075
609	609a	0.326	0.137	0.515	0.071
610	610a	0.234	0.122	0.462	0.056
700	700a	0.366	0.141	0.533	0.075
700	700b	0.091	0.085	0.360	0.031
701	701a	0.140	0.100	0.402	0.040
702	702a	0.616	0.170	0.654	0.111
702	702b	0.796	0.188	0.715	0.134
703	703a	0.667	0.176	0.672	0.118
703	703b	0.262	0.127	0.478	0.061
703	703c	0.438	0.151	0.576	0.087
704	704a	0.760	0.186	0.709	0.132
704	704b	0.341	0.139	0.523	0.073
704	704c	0.763	0.185	0.708	0.131
705	705a	0.856	0.193	0.728	0.141
705	705b	0.523	0.161	0.623	0.100
706	706a	0.112	0.092	0.378	0.035
707	707a	0.488	0.158	0.612	0.097
707	707b	0.496	0.158	0.614	0.097
708	708a	0.094	0.086	0.362	0.031
708	708b	0.161	0.106	0.420	0.045
751	751a	0.189	0.113	0.436	0.049
752	752a	0.177	0.110	0.429	0.047
753	753a	0.231	0.121	0.460	0.056
753	753b	0.188	0.112	0.435	0.049
753	753c	0.313	0.135	0.508	0.069
754	754a	0.158	0.106	0.420	0.045
806	806a	0.069	0.078	0.340	0.027
807	807a	0.159	0.106	0.420	0.045
807	807b	0.191	0.113	0.437	0.049
808	808a	0.243	0.123	0.467	0.057
901	901a	0.127	0.097	0.391	0.038
902	902a	0.142	0.102	0.406	0.041
903	903a	0.113	0.092	0.378	0.035
904	904a	0.106	0.090	0.372	0.033
904	904b	0.375	0.143	0.538	0.077
1001	1001a	0.112	0.092	0.378	0.035

Table 2A: Major System Results for the 100-Year, 3-Hour Chicago Storm

MH	SWMHYMO ID	Flow (m^3/s)	Flow Depth (m)	Velocity (m/s)	V*D (m^2/s)
1002	1002a	0.129	0.097	0.392	0.038
1003	1003a	0.228	0.120	0.456	0.055
1004	1004a	0.265	0.128	0.481	0.062
1005	1005a	0.272	0.129	0.486	0.063
1006	1006a	0.630	0.172	0.659	0.113
1007	1007a	0.047	0.065	0.297	0.019
1101	1101a	0.251	0.124	0.471	0.058
1103	1103a	0.296	0.133	0.500	0.067
1104	1104a	0.118	0.094	0.383	0.036
1106	1106a	0.206	0.115	0.444	0.051
1106	1106b	0.350	0.140	0.527	0.074
1108	1108a	0.288	0.132	0.495	0.065
1201	1201a	0.353	0.140	0.529	0.074
1203	1203a	0.335	0.138	0.520	0.072
1205	1205a	0.231	0.121	0.459	0.056
1209	1209a	0.217	0.117	0.450	0.053
1213	1213a	0.282	0.131	0.493	0.065
1213	1213b	0.288	0.132	0.495	0.065
1214	1214a	0.266	0.128	0.482	0.062
1214	1214b	0.137	0.100	0.401	0.040
1215	1215a	0.377	0.144	0.542	0.078
1215	1215b	0.347	0.140	0.526	0.074
1216	1216a	0.596	0.168	0.646	0.109
1216	1216b	0.493	0.158	0.612	0.097
1217	1217a	0.601	0.169	0.649	0.110
1218	1218a	0.372	0.143	0.539	0.077
1219	1219a	0.577	0.167	0.643	0.107
1219	1219b	0.403	0.147	0.558	0.082
7000	7000a	0.672	0.176	0.674	0.119

Table 2B: Major System Results for the 100-Year, 3-Hour Chicago Storm +20%

MH	SWMHYMO ID	Flow (m^3/s)	Flow Depth (m)	Velocity (m/s)	V*D (m^2/s)
101	101a	0.370	0.143	0.539	0.077
103	103a	0.194	0.113	0.439	0.050
103	103b	0.258	0.126	0.475	0.060
105	105a	0.207	0.116	0.445	0.052
106	106a	2.105	0.279	0.898	0.251
106	106b	2.156	0.282	0.902	0.254
107	107a	0.108	0.091	0.374	0.034
107	107b	2.209	0.285	0.907	0.258
108	108a	0.288	0.132	0.495	0.065
202	202a	0.282	0.130	0.491	0.064
203	203a	0.438	0.151	0.576	0.087
204	204a	0.407	0.147	0.557	0.082
205	205a	0.405	0.147	0.557	0.082
206	206a	1.338	0.233	0.821	0.191
206	206b	0.200	0.115	0.442	0.051
207	207a	1.425	0.239	0.833	0.199
209	209a	1.421	0.239	0.833	0.199
209	209b	0.304	0.134	0.505	0.068
210	210a	1.400	0.236	0.828	0.195
210	210b	0.197	0.114	0.440	0.050
211	211a	0.242	0.123	0.465	0.057
212	212a	0.159	0.107	0.421	0.045
251	251a	0.229	0.120	0.457	0.055
252	252a	0.229	0.120	0.458	0.055
253	253a	0.562	0.165	0.637	0.105
253	253b	0.253	0.125	0.472	0.059
253	253c	0.450	0.152	0.583	0.089
254	254a	0.170	0.109	0.426	0.046
254	254b	0.805	0.189	0.717	0.136
261	261a	0.282	0.130	0.491	0.064
262	262a	0.409	0.147	0.558	0.082
263	263a	0.413	0.148	0.564	0.083
264	264a	0.375	0.142	0.538	0.076
264	264b	0.190	0.113	0.437	0.049
265	265a	0.559	0.165	0.636	0.105
265	265b	0.455	0.153	0.587	0.090
266	266a	0.801	0.189	0.717	0.136
266	266b	0.263	0.127	0.479	0.061
266	266c	0.294	0.133	0.500	0.067
267	267a	1.016	0.208	0.768	0.160
267	267b	0.205	0.115	0.444	0.051
267	267c	0.508	0.159	0.618	0.098
301	301a	0.178	0.110	0.430	0.047
304	304a	0.228	0.120	0.457	0.055
305	305a	0.430	0.150	0.570	0.086
305	305b	0.343	0.139	0.525	0.073
306	306a	0.581	0.167	0.644	0.108
306	306b	0.307	0.135	0.506	0.068
307	307a	0.803	0.189	0.716	0.135
307	307b	0.391	0.145	0.548	0.079
401	401a	0.310	0.135	0.507	0.068
404	404a	0.401	0.147	0.557	0.082
405	405a	0.602	0.169	0.651	0.110

Table 2B: Major System Results for the 100-Year, 3-Hour Chicago Storm +20%

MH	SWMHYMO ID	Flow (m^3/s)	Flow Depth (m)	Velocity (m/s)	V*D (m^2/s)
500	500a	0.312	0.135	0.508	0.069
502	502a	0.351	0.140	0.528	0.074
506	506a	0.370	0.142	0.536	0.076
506	506b	0.232	0.121	0.459	0.056
507	507a	0.221	0.118	0.453	0.053
507	507b	0.707	0.179	0.687	0.123
508	508a	0.051	0.068	0.305	0.021
509	509a	0.710	0.181	0.692	0.125
509	509b	0.175	0.110	0.429	0.047
510	510a	0.715	0.180	0.691	0.124
511	511a	0.234	0.121	0.461	0.056
601	601a	0.223	0.119	0.454	0.054
603	603a	0.297	0.133	0.500	0.067
604	604a	0.796	0.189	0.717	0.136
605	605a	0.855	0.194	0.729	0.141
606	606a	0.962	0.203	0.755	0.153
607	607a	0.832	0.191	0.722	0.138
609	609a	0.826	0.191	0.723	0.138
610	610a	0.299	0.134	0.502	0.067
700	700a	0.569	0.166	0.638	0.106
700	700b	0.115	0.093	0.380	0.035
701	701a	0.175	0.110	0.429	0.047
702	702a	1.178	0.220	0.793	0.174
702	702b	0.998	0.205	0.760	0.156
703	703a	1.459	0.241	0.837	0.202
703	703b	0.334	0.138	0.519	0.072
703	703c	0.548	0.163	0.631	0.103
704	704a	1.903	0.269	0.883	0.238
704	704b	0.427	0.149	0.569	0.085
704	704c	0.971	0.203	0.753	0.153
705	705a	2.039	0.276	0.893	0.246
705	705b	0.667	0.176	0.672	0.118
706	706a	0.142	0.101	0.404	0.041
707	707a	0.507	0.160	0.619	0.099
707	707b	0.632	0.172	0.660	0.114
708	708a	0.118	0.094	0.383	0.036
708	708b	0.219	0.118	0.452	0.053
751	751a	0.242	0.123	0.465	0.057
752	752a	0.252	0.125	0.473	0.059
753	753a	0.408	0.147	0.557	0.082
753	753b	0.264	0.127	0.479	0.061
753	753c	0.391	0.145	0.548	0.079
754	754a	0.202	0.115	0.442	0.051
806	806a	0.087	0.084	0.356	0.030
807	807a	0.246	0.123	0.467	0.057
807	807b	0.245	0.123	0.467	0.057
808	808a	0.359	0.141	0.530	0.075
901	901a	0.159	0.107	0.421	0.045
902	902a	0.199	0.114	0.442	0.050
903	903a	0.160	0.106	0.420	0.045
904	904a	0.133	0.099	0.397	0.039
904	904b	0.424	0.149	0.566	0.084
1001	1001a	0.142	0.101	0.404	0.041

Table 2B: Major System Results for the 100-Year, 3-Hour Chicago Storm +20%

MH	SWMHYMO ID	Flow (m^3/s)	Flow Depth (m)	Velocity (m/s)	V*D (m^2/s)
1002	1002a	0.181	0.111	0.431	0.048
1003	1003a	0.338	0.138	0.519	0.072
1004	1004a	0.418	0.149	0.565	0.084
1005	1005a	0.467	0.154	0.594	0.091
1006	1006a	0.801	0.188	0.716	0.135
1007	1007a	0.059	0.072	0.319	0.023
1101	1101a	0.321	0.136	0.512	0.070
1103	1103a	0.401	0.147	0.557	0.082
1104	1104a	0.149	0.103	0.411	0.042
1106	1106a	0.284	0.131	0.492	0.064
1106	1106b	0.556	0.165	0.636	0.105
1108	1108a	0.360	0.141	0.533	0.075
1201	1201a	0.443	0.151	0.579	0.087
1203	1203a	0.464	0.154	0.595	0.092
1205	1205a	0.413	0.147	0.560	0.082
1209	1209a	0.389	0.144	0.546	0.079
1213	1213a	0.475	0.156	0.602	0.094
1213	1213b	0.360	0.141	0.533	0.075
1214	1214a	0.533	0.162	0.626	0.101
1214	1214b	0.173	0.109	0.428	0.047
1215	1215a	0.751	0.184	0.705	0.130
1215	1215b	0.435	0.150	0.574	0.086
1216	1216a	1.068	0.211	0.776	0.164
1216	1216b	0.627	0.171	0.658	0.113
1217	1217a	1.236	0.225	0.804	0.181
1218	1218a	1.024	0.209	0.770	0.161
1219	1219a	1.365	0.233	0.821	0.191
1219	1219b	0.506	0.159	0.617	0.098
7000	7000a	0.855	0.193	0.727	0.140

ATTACHMENT

4

SWM FACILITY OPERATING CONDITIONS

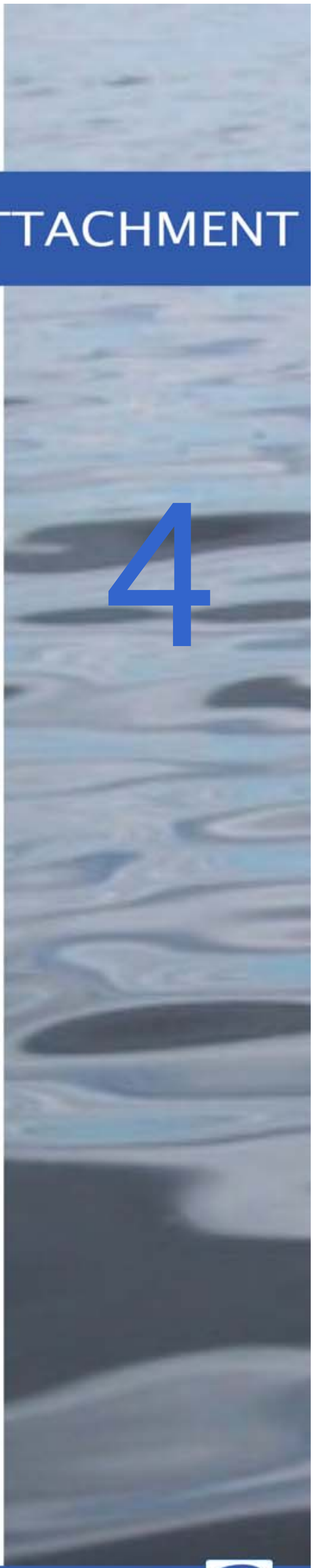


Table 3A: Summary of SWM Facility 1 Operating Characteristics (1)

Pond Component	Pre-Development Outflow (m^3/s)	Major System Outflow (2) (m^3/s)	Pond Level (m)	Pond Outflow (m^3/s)	Volume Used (3) (m^3)
Permanent Pool	N/A	N/A	92.35	N/A	27226
Quality Control	N/A	N/A	92.50	0.038	4443
2-Year, 24-Hour SCS	2.767	0.000	93.15	0.295	26914
5-Year, 24-Hour SCS	4.290	0.000	93.52	0.377	41388
10-Year, 24-Hour SCS	5.348	0.000	93.71	0.753	49336
25-Year, 24-Hour SCS	6.694	0.000	93.76	2.017	51535
50-Year, 24-Hour SCS	7.749	1.870	93.78	2.832	52536
100-Year, 24-Hour SCS	8.894	4.039	93.80	3.579	53454
100-Year, 24-Hour SCS (4)	8.894	4.039	93.81	3.666	53946
100-Year, 10-Year Spring (4)	N/A	0.000	94.22	2.915	72725

(1) Based on 24 hour detention of 40 m^3/ha quality control volume, erosion control of 2-year release rate to 330 L/s (as per Parish Geomorphic threshold) and quantity control of 5- to 100-year release rates to pre-development levels.

(2) Major system flows from 30.79 ha of the Pond 1 subdivision area discharge directly to the Van Gaal Drain, and from 21.75 ha to the Moore Drain Tributary, as modelled in SWMHYMO Rtar_F.*.

(3) Volumes used are active storage only for all pond components except the permanent pool.

(4) Restrictive downstream conditions; Summer = 93.68 m, Spring = 94.11 m. All other results based on free outfall conditions.

Table 3B: Summary of SWM Facility 2 Operating Characteristics (1)

Pond Component	Pre-Development Outflow (m^3/s)	Pond Level (m)	Pond Outflow (m^3/s)	Volume Used (2) (m^3)
Permanent Pool	N/A	93.20	N/A	2990
Quality Control	N/A	93.35	0.038	1837
2-Year, 24-Hour SCS	0.705	93.68	0.591	6618
5-Year, 24-Hour SCS	1.094	93.82	0.947	8987
10-Year, 24-Hour SCS	1.364	93.91	1.195	10526
25-Year, 24-Hour SCS	1.708	94.02	1.514	12430
50-Year, 24-Hour SCS	1.976	94.11	1.765	13893
100-Year, 24-Hour SCS	2.267	94.20	2.042	15471
100-Year, 10-Year Spring (3)	N/A	94.84	1.455	27454

(1) Based on 24 hour detention of 40 m^3/ha quality control volume and quantity control of 100-year release rate to capacity of 1500 mm outlet pipe at 0.1% slope (2.235 m^3/s). No erosion control is provided.

(2) Volumes used are active storage only for all pond components except the permanent pool.

(3) Restrictive downstream conditions; Spring = 94.18 m. All other results based on free outfall conditions.

Table 4A: Extended Detention Parameters for SWM Facility 1

Permanent Pool Parameters			Quality Orifice Parameters		
Area (C3)	27226.15	m^2	Diameter	0.300	m
Volume	39222.65	m^3			
PP Elev	92.35	m	Area	0.07069	m^2
QC Elev	92.50	m	Invert	92.35	m
h (m)	0.15	m	C_o	0.620	

Notes:
C3 is the intercept from the area-depth linear regression
PP Elev indicates the elevation of the permanent pool
QC Elev indicates the elevation of the storage volume required by MOE for quality control
h is the maximum water elevation above the orifice (m)

Table 4B: Extended Detention Drawdown Time for SWM Facility 1

Elev.	Active Storage			C2	Drawdown Time	Drawdown Time	Flow	Demarkation
(m)	V (m^3)	A (m^2)	depth (m)	(m²/m)	(h)	(days)	(m^3/s)	Point
92.35	**0.00**	**27226.15**	**0.00**				**0.000**	**PP Elev**
92.40	1436.02	28902.23	0.05	33522	17.75	0.74	0.013	
92.45	2929.23	29951.41	0.10	27253	25.42	1.06	0.025	
92.50	**4443.44**	**30505.36**	**0.15**	**21861**	**31.33**	**1.31**	**0.038**	**QC Elev**
92.55	5991.84	31236.19	0.20	20050	36.49	1.52	0.050	

Notes:
C2 is the slope coefficient from the area-depth linear regression
QC Elev indicates the elevation of the quality control volume required by MOE

Table 4C: Extended Detention Parameters for SWM Facility 2

Permanent Pool Parameters			Quality Orifice Parameters		
Area (C3)	11075.10	m^2	Diameter	0.300	m
Volume	2990.03	m^3			
PP Elev	93.20	m	Area	0.07069	m^2
QC Elev	93.35	m	Invert	93.20	m
h (m)	0.15	m	C_o	0.620	

Notes:
C3 is the intercept from the area-depth linear regression
PP Elev indicates the elevation of the permanent pool
QC Elev indicates the elevation of the storage volume required by MOE for quality control
h is the maximum water elevation above the orifice (m)

Table 4D: Extended Detention Drawdown Time for SWM Facility 2

Elev.	Active Storage			C2	Drawdown Time	Drawdown Time	Flow	Demarkation
(m)	V (m^3)	A (m^2)	depth (m)	(m²/m)	(h)	(days)	(m^3/s)	Point
93.20	**0.00**	**11075.10**	**0.00**				**0.000**	**PP Elev**
93.25	573.33	11836.91	0.90	846	30.71	1.28	0.013	
93.30	1196.62	12676.33	0.95	1685	32.32	1.35	0.025	
93.35	**1836.54**	**12867.85**	**1.00**	**1793**	**33.34**	**1.39**	**0.038**	**QC Elev**
93.40	2504.49	13398.14	1.05	2212	34.68	1.44	0.050	

Notes:
C2 is the slope coefficient from the area-depth linear regression
QC Elev indicates the elevation of the quality control volume required by MOE

Table 5A: Stage-Storage-Outflow Curve for SWM Facility 1

			Quality Control 1		Erosion Control 1		Quantity Control 1			
			Vertical Orifice		Vertical Orifice		Rectangular Weir			
			Dia (m)	0.300	Dia (m)	0.300	L (m)	45.000		
			Area (m^2)	0.071	Area (m^2)	0.071				
			Invert (m)	92.35	Invert (m)	92.50	C_w	1.700		
			C_o	0.62	C_o	0.62	Invert (m)	93.68		
			Q @ D	0.075	Q @ D	0.075	n contr.	2		
Elevation (m)	Active Sto. (m^3)	Demarkation Points	Head (m)	Outflow (m^3/s)	Head (m)	Outflow (m^3/s)	Head (m)	Outflow (m^3/s)	Outflow (m^3/s)	Storage (ha·m)
92.35	**0**	**PP Elev**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**
92.40	1436		0.050	0.013	0.000	0.000	0.000	0.000	0.013	0.144
92.45	2929		0.100	0.025	0.000	0.000	0.000	0.000	0.025	0.293
92.50	**4443**	**QC Elev**	**0.150**	**0.038**	**0.000**	**0.000**	**0.000**	**0.000**	**0.038**	**0.444**
92.55	5992		0.200	0.050	0.050	0.013	0.000	0.000	0.063	0.599
92.60	7560		0.250	0.063	0.100	0.025	0.000	0.000	0.088	0.756
92.65	9168		0.300	0.075	0.150	0.038	0.000	0.000	0.113	0.917
92.70	10802		0.350	0.087	0.200	0.050	0.000	0.000	0.137	1.080
92.75	12459		0.400	0.097	0.250	0.063	0.000	0.000	0.160	1.246
92.80	14148		0.450	0.106	0.300	0.075	0.000	0.000	0.182	1.415
92.85	15867		0.500	0.115	0.350	0.087	0.000	0.000	0.202	1.587
92.90	17620		0.550	0.123	0.400	0.097	0.000	0.000	0.220	1.762
92.95	19390		0.600	0.130	0.450	0.106	0.000	0.000	0.237	1.939
93.00	21197		0.650	0.137	0.500	0.115	0.000	0.000	0.252	2.120
93.05	23028		0.700	0.144	0.550	0.123	0.000	0.000	0.267	2.303
93.10	24894		0.750	0.150	0.600	0.130	0.000	0.000	0.281	2.489
93.15	26779		0.800	0.157	0.650	0.137	0.000	0.000	0.294	2.678
93.20	28700		0.850	0.162	0.700	0.144	0.000	0.000	0.306	2.870
93.25	30634		0.900	0.168	0.750	0.150	0.000	0.000	0.318	3.063
93.30	32587		0.950	0.174	0.800	0.157	0.000	0.000	0.330	3.259
93.35	34548		1.000	0.179	0.850	0.162	0.000	0.000	0.341	3.455
93.40	36547		1.050	0.184	0.900	0.168	0.000	0.000	0.352	3.655
93.45	38559		1.100	0.189	0.950	0.174	0.000	0.000	0.363	3.856
93.50	40641		1.150	0.194	1.000	0.179	0.000	0.000	0.373	4.064
93.55	42717		1.200	0.199	1.050	0.184	0.000	0.000	0.383	4.272
93.60	44822		1.250	0.204	1.100	0.189	0.000	0.000	0.393	4.482
93.65	46961		1.300	0.208	1.150	0.194	0.000	0.000	0.402	4.696
93.68	**48248**	**Ext. Det.**	**1.330**	**0.211**	**1.180**	**0.197**	**0.000**	**0.000**	**0.408**	**4.825**
93.70	49106		1.350	0.213	1.200	0.199	0.020	0.216	0.628	4.911
93.75	51313		1.400	0.217	1.250	0.204	0.070	1.416	1.837	5.131
93.80	53489		1.450	0.221	1.300	0.208	0.120	3.178	3.608	5.349
93.85	55848		1.500	0.226	1.350	0.213	0.170	5.358	5.796	5.585
93.90	58767		1.550	0.230	1.400	0.217	0.220	7.886	8.333	5.877
93.95	60124		1.600	0.234	1.450	0.221	0.270	10.720	11.175	6.012
94.00	**62403**	**Top of Berm**	**1.650**	**0.238**	**1.500**	**0.226**	**0.320**	**13.828**	**14.292**	**6.240**
94.05	64689		1.700	0.242	1.550	0.230	0.370	17.189	17.660	6.469
94.10	67024		1.750	0.246	1.600	0.234	0.420	20.784	21.263	6.702
94.11	**67493**	**100-Yr Spr**	**1.760**	**0.246**	**1.610**	**0.235**	**0.430**	**21.529**	**22.010**	**6.749**
94.15	69372		1.800	0.249	1.650	0.238	0.470	24.598	25.085	6.937
94.20	71747		1.850	0.253	1.700	0.242	0.520	28.619	29.114	7.175
94.25	74132		1.900	0.257	1.750	0.246	0.570	32.838	33.340	7.413
94.30	76504		1.950	0.260	1.800	0.249	0.620	37.244	37.753	7.650
94.35	78940		2.000	0.264	1.850	0.253	0.670	41.829	42.346	7.894
94.40	81361		2.050	0.268	1.900	0.257	0.720	46.587	47.112	8.136
94.45	83758		2.100	0.271	1.950	0.260	0.770	51.512	52.044	8.376
94.50	86225		2.150	0.275	2.000	0.264	0.820	56.597	57.136	8.622

Notes :
- PP Elev indicates the elevation of the permanent pool.
- QC Elev indicates the elevation of the storage volume provided for quality control according to MOE requirements.
- Ext. Det. indicates the elevation of extended detention provided, and of the Van Gaal Drain floodplain during the 100-year summer (SCS) event.
- 100-Yr Spr indicates the elevation of the Van Gaal Drain floodplain during the 100-year spring (snow+rain) event.
- Top of Berm indicates the elevation at the top of the berm.

Table 5B: Stage-Storage-Outflow Curve for SWM Facility 2

			Quality Control 1		Quantity Control 1			
			Vertical Orifice		Rectangular Weir			
			Dia (m)	0.300	L (m)	1.500		
			Area (m^2)	0.071				
			Invert (m)	93.20	C_w	1.800		
			C_o	0.62	Invert (m)	93.35		
			Q @ D	0.075	n contr.	2		
Elevation	Active Sto.	Demarkation Points	Head	Outflow	Head	Outflow	Outflow	Storage
(m)	(m^3)		(m)	(m^3/s)	(m)	(m^3/s)	(m^3/s)	(ha·m)
93.20	**0**	**PP Elev**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**
93.25	573		0.050	0.013	0.000	0.000	0.013	0.057
93.30	1197		0.100	0.025	0.000	0.000	0.025	0.120
93.35	**1837**	**QC Elev**	**0.150**	**0.038**	**0.000**	**0.000**	**0.038**	**0.184**
93.40	2504		0.200	0.050	0.050	0.030	0.080	0.250
93.45	3183		0.250	0.063	0.100	0.084	0.147	0.318
93.50	3889		0.300	0.075	0.150	0.154	0.229	0.389
93.55	4649		0.350	0.087	0.200	0.235	0.322	0.465
93.60	5428		0.400	0.097	0.250	0.326	0.423	0.543
93.65	6210		0.450	0.106	0.300	0.426	0.532	0.621
93.70	7016		0.500	0.115	0.350	0.533	0.648	0.702
93.75	7835		0.550	0.123	0.400	0.647	0.769	0.783
93.80	8666		0.600	0.130	0.450	0.766	0.896	0.867
93.85	9502		0.650	0.137	0.500	0.891	1.028	0.950
93.90	10346		0.700	0.144	0.550	1.021	1.165	1.035
93.95	11191		0.750	0.150	0.600	1.154	1.305	1.119
94.00	12047		0.800	0.157	0.650	1.292	1.449	1.205
94.05	12914		0.850	0.162	0.700	1.434	1.596	1.291
94.10	13785		0.900	0.168	0.750	1.578	1.746	1.378
94.15	14663		0.950	0.174	0.800	1.726	1.900	1.466
94.18	**15193**	**100-Yr Spr**	**0.980**	**0.177**	**0.830**	**1.816**	**1.993**	**1.519**
94.20	15547		1.000	0.179	0.850	1.876	2.055	1.555
94.25	16438		1.050	0.184	0.900	2.029	2.213	1.644
94.30	17336		1.100	0.189	0.950	2.183	2.373	1.734
94.35	18242		1.150	0.194	1.000	2.340	2.534	1.824
94.40	19153		1.200	0.199	1.050	2.498	2.697	1.915
94.45	20072		1.250	0.204	1.100	2.658	2.862	2.007
94.50	20997		1.300	0.208	1.150	2.819	3.027	2.100
94.55	21930		1.350	0.213	1.200	2.981	3.194	2.193
94.60	22870		1.400	0.217	1.250	3.144	3.362	2.287
94.65	23817		1.450	0.221	1.300	3.308	3.530	2.382
94.70	24770		1.500	0.226	1.350	3.473	3.698	2.477
94.75	25732		1.550	0.230	1.400	3.638	3.867	2.573
94.81	26699		1.609	0.234	1.459	3.833	4.067	2.670
94.85	27675		1.650	0.238	1.500	3.968	4.206	2.767
94.90	28654		1.700	0.242	1.550	4.133	4.375	2.865
94.95	29644		1.750	0.246	1.600	4.299	4.544	2.964
95.00	30645		1.800	0.249	1.650	4.464	4.713	3.064
95.05	31649		1.850	0.253	1.700	4.628	4.881	3.165
95.10	32661		1.900	0.257	1.750	4.792	5.049	3.266
95.15	33688		1.950	0.260	1.800	4.955	5.216	3.369
95.20	34713		2.000	0.264	1.850	5.118	5.382	3.471
95.25	35748		2.050	0.268	1.900	5.280	5.547	3.575
95.30	36756		2.100	0.271	1.950	5.441	5.712	3.676
95.35	37820		2.150	0.275	2.000	5.600	5.875	3.782
95.40	38883		2.200	0.278	2.050	5.759	6.037	3.888
95.45	39936		2.250	0.281	2.100	5.916	6.197	3.994
95.50	40980		2.300	0.285	2.150	6.072	6.356	4.098
95.55	42077		2.350	0.288	2.200	6.226	6.514	4.208
95.60	43165		2.400	0.291	2.250	6.379	6.670	4.317
95.65	44262		2.450	0.294	2.300	6.530	6.824	4.426

Table 5B: Stage-Storage-Outflow Curve for SWM Facility 2

			Quality Control 1		Quantity Control 1			
			Vertical Orifice		Rectangular Weir			
			Dia (m)	0.300	L (m)	1.500		
			Area (m^2)	0.071				
			Invert (m)	93.20	C_w	1.800		
			C_o	0.62	Invert (m)	93.35		
			Q @ D	0.075	n contr.	2		
Elevation	Active Sto.	Demarkation Points	Head	Outflow	Head	Outflow	Outflow	Storage
(m)	(m^3)		(m)	(m^3/s)	(m)	(m^3/s)	(m^3/s)	(ha·m)
95.70	45394		2.500	0.298	2.350	6.679	6.977	4.539
95.75	46502		2.550	0.301	2.400	6.826	7.127	4.650
95.80	47623		2.600	0.304	2.450	6.972	7.276	4.762
95.85	48764		2.650	0.307	2.500	7.115	7.422	4.876
95.90	51022		2.700	0.310	2.550	7.256	7.566	5.102
95.95	50837		2.750	0.313	2.600	7.395	7.708	5.084
96.00	**51987**	**Top of Berm**	**2.800**	**0.316**	**2.650**	**7.532**	**7.848**	**5.199**

Notes :
- PP Elev indicates the elevation of the permanent pool.
- QC Elev indicates the elevation of the storage volume provided for quality control according to MOE requirements.
- 100-Yr Spr indicates the elevation of the Jock River floodplain during the 100-year spring (snow+rain) event.
- Top of Berm indicates the elevation at the top of the berm.

ATTACHMENT

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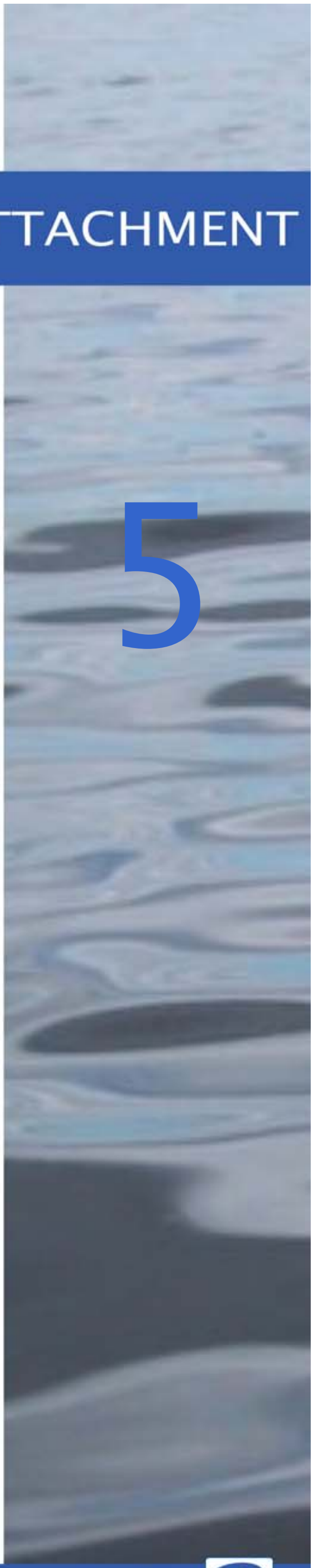


PIPE DATA AND HYDRAULIC SIMULATION RESULTS

Table 6: Pipe Data and Hydraulic Simulation Results [(1)]

U/S MH	D/S MH	U/S Invert (m)	D/S Invert (m)	Pipe Diameter / Height (m)	Pipe Width (m)	Pipe Length (m)	Pipe Slope (%)	n	U/S MH Cover Elev. (m)	D/S MH Cover Elev. (m)	Design Velocity (m/s)	Design Flow (m^3/s)	100-year, 3-hour Chicago Storm							100-year, 12-hour SCS Type II Storm							100-year, 10-day Spring Snowmelt + Rainfall Event						
													Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [(2)] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [(2)] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [(2)] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)
101	102	93.228	93.161	0.825	N/A	67.0	0.1	0.013	95.598	95.587	0.85	0.45	0.29	0.64	-0.28	1.1	93.78	93.78	1.82	0.29	0.64	-0.24	12.0	93.82	93.81	1.78	0.02	0.04	0.17	109.0	94.22	94.22	1.38
102	103	93.141	93.061	0.825	N/A	80.0	0.1	0.013	95.587	95.470	0.85	0.45	0.29	0.64	-0.19	1.1	93.78	93.78	1.81	0.28	0.62	-0.15	12.1	93.81	93.81	1.77	0.02	0.04	0.26	109.0	94.22	94.22	1.37
103	104	92.911	92.850	0.975	N/A	60.5	0.1	0.013	95.470	95.373	0.95	0.71	0.60	0.85	-0.11	1.1	93.78	93.77	1.69	0.58	0.82	-0.07	12.1	93.81	93.81	1.66	0.04	0.06	0.34	109.0	94.22	94.22	1.25
104	105	92.820	92.796	0.975	N/A	23.5	0.1	0.013	95.373	95.331	0.95	0.71	0.60	0.85	-0.02	1.1	93.77	93.77	1.60	0.58	0.82	0.02	12.1	93.81	93.81	1.56	0.04	0.06	0.43	109.0	94.22	94.22	1.15
105	106	92.721	92.619	1.050	N/A	101.5	0.1	0.013	95.331	95.180	1.00	0.86	0.76	0.88	0.00	1.1	93.77	93.77	1.56	0.72	0.83	0.04	12.1	93.81	93.81	1.52	0.05	0.06	0.45	109.0	94.22	94.22	1.11
106	107	92.559	92.483	1.050	N/A	76.5	0.1	0.013	95.180	95.070	1.00	0.86	0.87	1.01	0.16	1.1	93.77	93.77	1.41	0.82	0.95	0.20	12.1	93.81	93.81	1.37	0.05	0.06	0.61	109.0	94.22	94.22	0.96
107	108	92.333	92.258	1.200	N/A	74.5	0.1	0.013	95.070	95.057	1.09	1.23	1.14	0.92	0.24	1.1	93.77	93.77	1.30	1.06	0.86	0.28	12.1	93.81	93.81	1.26	0.07	0.06	0.69	109.0	94.22	94.22	0.85
108	109	92.238	92.211	1.200	N/A	26.5	0.1	0.013	95.057	95.487	1.09	1.23	1.26	1.02	0.33	1.1	93.77	93.77	1.29	1.17	0.95	0.37	12.1	93.81	93.81	1.25	0.08	0.06	0.78	109.0	94.22	94.22	0.84
109	Pond1	92.181	92.150	1.200	N/A	31.0	0.1	0.013	95.487	94.500	1.09	1.23	1.26	1.02	0.39	1.1	93.77	93.77	1.72	1.16	0.94	0.43	12.1	93.81	93.81	1.68	0.08	0.1	0.84	109.0	94.22	94.22	1.27
202	203	93.826	93.706	0.675	N/A	120.0	0.1	0.013	96.212	96.030	0.74	0.27	0.21	0.79	-0.16	1.1	94.34	94.24	1.87	0.21	0.79	0.02	12.0	94.52	94.46	1.69	0.01	0.04	-0.27	109.1	94.23	94.23	1.99
203	204	93.556	93.479	0.825	N/A	77.0	0.1	0.013	96.030	95.920	0.85	0.45	0.43	0.95	-0.14	1.1	94.24	94.19	1.79	0.42	0.93	0.08	12.0	94.46	94.40	1.57	0.03	0.07	-0.16	109.1	94.23	94.23	1.81
204	205	93.404	93.332	0.900	N/A	72.0	0.1	0.013	95.920	95.812	0.90	0.57	0.53	0.93	-0.12	1.1	94.19	94.14	1.73	0.52	0.91	0.10	12.2	94.40	94.35	1.52	0.03	0.05	-0.08	109.1	94.23	94.23	1.70
205	206	93.257	93.184	0.975	N/A	73.0	0.1	0.013	95.812	95.700	0.95	0.71	0.62	0.87	-0.10	1.1	94.14	94.05	1.68	0.61	0.86	0.11	12.2	94.35	94.26	1.47	0.04	0.06	-0.01	109.1	94.23	94.22	1.59
206	207	92.959	92.925	1.200	1.800	34.0	0.1	0.013	95.700	95.898	1.23	2.66	2.27	0.85	-0.11	1.1	94.05	94.02	1.65	2.20	0.83	0.10	12.2	94.26	94.23	1.44	0.15	0.06	0.06	109.0	94.22	94.22	1.48
207	208	92.875	92.819	1.200	1.800	55.5	0.1	0.013	95.898	95.684	1.23	2.66	2.31	0.87	-0.06	1.1	94.02	93.96	1.88	2.27	0.85	0.15	12.2	94.23	94.18	1.67	0.15	0.06	0.15	109.1	94.22	94.22	1.67
208	209	92.769	92.729	1.200	1.800	39.5	0.1	0.013	95.684	95.520	1.23	2.66	2.32	0.87	-0.01	1.3	93.96	93.93	1.72	2.27	0.85	0.21	12.2	94.18	94.15	1.51	0.15	0.06	0.25	109.1	94.22	94.22	1.46
209	210	92.679	92.603	1.200	1.800	76.5	0.1	0.013	95.520	95.410	1.23	2.66	2.59	0.97	0.05	1.3	93.93	93.86	1.59	2.49	0.94	0.27	12.2	94.15	94.08	1.37	0.17	0.06	0.34	109.0	94.22	94.22	1.30
210	211	92.583	92.507	1.200	1.800	76.5	0.1	0.013	95.410	95.290	1.23	2.66	2.79	1.05	0.08	1.2	93.86	93.78	1.55	2.68	1.01	0.30	12.2	94.08	94.00	1.33	0.18	0.07	0.44	109.0	94.22	94.22	1.19
211	212	92.487	92.389	1.200	2.400	82.0	0.1	0.013	95.290	95.165	1.45	4.17	4.92	1.18	0.09	1.2	93.78	93.77	1.51	4.71	1.13	0.32	12.2	94.00	93.88	1.29	0.32	0.08	0.54	109.0	94.22	94.22	1.07
212	213	92.369	92.290	1.200	2.400	65.5	0.1	0.013	95.165	95.108	1.45	4.17	5.03	1.21	0.20	1.2	93.77	93.77	1.39	4.83	1.16	0.31	12.2	93.88	93.81	1.29	0.33	0.08	0.65	109.0	94.22	94.22	0.94
213	214	92.210	92.172	1.200	2.400	31.5	0.1	0.013	95.108	95.454	1.45	4.17	5.02	1.20	0.36	1.2	93.77	93.77	1.34	4.82	1.16	0.40	12.2	93.81	93.81	1.30	0.33	0.08	0.81	109.0	94.22	94.22	0.89
214	215	92.122	92.081	1.200	2.400	34.5	0.1	0.013	95.454	95.479	1.45	4.17	5.01	1.20	0.45	1.2	93.77	93.77	1.68	4.82	1.16	0.49	12.2	93.81	93.81	1.64	0.33	0.08	0.90	109.0	94.22	94.22	1.23
215	Pond1	92.001	91.950	1.200	2.400	42.5	0.1	0.013	95.479	94.500	1.45	4.17	4.97	1.19	0.57	1.2	93.77	93.77	1.71	4.82	1.16	0.61	12.2	93.81	93.81	1.67	0.33	0.08	1.02	109.0	94.22	94.22	1.26
251	252	93.818	93.748	0.675	N/A	70.5	0.1	0.013	96.160	96.054	0.74	0.27	0.18	0.68	-0.22	1.1	94.28	94.24	1.88	0.17	0.64	-0.03	12.0	94.47	94.44	1.69	0.01	0.04	-0.27	109.0	94.23	94.23	1.94
252	253	93.673	93.603	0.750	N/A	70.5	0.1	0.013	96.054	95.949	0.80	0.35	0.26	0.74	-0.18	1.1	94.24	94.20	1.82	0.25	0.71	0.02	12.0	94.44	94.40	1.61	0.02	0.06	-0.20	109.1	94.23	94.23	1.83
253	254	93.303	93.227	1.050	N/A	76.5	0.1	0.013	95.949	95.834	1.00	0.86	0.82	0.95	-0.15	1.1	94.20	94.07	1.75	0.79	0.91	0.05	12.0	94.40	94.29	1.55	0.05	0.06	-0.13	109.0	94.23	94.22	1.72
254	206	93.077	92.989	1.200	1.800	88.0	0.1	0.013	95.834	95.700	1.23	2.66	1.53	0.58	-0.21	1.1	94.07	94.05	1.76	1.47	0.55	0.01	12.0	94.29	94.26	1.55	0.10	0.04	-0.05	109.0	94.22	94.22	1.61
261	262	93.851	93.753	0.675	N/A	97.5	0.1	0.013	96.161	96.015	0.74	0.27	0.21	0.79	-0.13	1.1	94.39	94.34	1.77	0.21	0.79	0.05	12.0	94.58	94.53	1.58	0.01	0.04	-0.30	109.0	94.23	94.23	1.94
262	263	93.603	93.484	0.825	N/A	119.0	0.1	0.013	96.015	95.837	0.85	0.45	0.40	0.88	-0.09	1.1	94.34	94.27	1.68	0.38	0.84	0.11	12.0	94.53	94.45	1.48	0.03	0.07	-0.20	109.1	94.23	94.23	1.79
263	264	93.409	93.302	0.900	N/A	107.0	0.1	0.013	95.837	95.676	0.90	0.57	0.56	0.98	-0.04	1.3	94.27	94.14	1.57	0.54	0.94	0.14	12.2	94.45	94.33	1.39	0.03	0.05	-0.08	109.1	94.23	94.23	1.61
264	265	93.222	93.182	0.900	N/A	40.5	0.1	0.013	95.676	95.616	0.90	0.57	0.67	1.17	0.02	1.3	94.14	94.09	1.53	0.64	1.12	0.21	12.2	94.33	94.29	1.34	0.04	0.07	0.10	109.1	94.23	94.22	1.45
265	266	93.032	93.002	1.050	N/A	30.0	0.1	0.013	95.616	95.571	1.00	0.86	1.01	1.17	0.01	1.3	94.09	94.05	1.52	0.99	1.15	0.21	12.2	94.29	94.26	1.33	0.06	0.07	0.14	109.1	94.22	94.22	1.39
266	267	92.852	92.753	1.200	N/A	99.0	0.1	0.013	95.571	95.421	1.09	1.23	1.52	1.23	0.00	1.1	94.05	93.89	1.52	1.44	1.17	0.20	12.2	94.26	94.12	1.32	0.10	0.08	0.17	109.0	94.22	94.22	1.35
267	268	92.703	92.657	1.200	1.800	45.5	0.1	0.013	95.421	95.342	1.23	2.66	2.07	0.78	-0.01	1.1	93.89	93.85	1.53	1.96	0.74	0.21	12.1	94.12	94.08	1.30	0.13	0.05	0.32	109.0	94.22	94.22	1.20
268	211	92.607	92.567	1.200	1.800	40.5	0.1	0.013	95.342	95.290	1.23	2.66	2.05	0.77	0.05	1.1	93.85	93.78	1.49	1.96	0.74	0.27	12.1	94.08	94.00	1.26	0.13	0.05	0.42	109.0	94.22	94.22	1.12
301	302	93.248	93.194	0.600	N/A	54.0	0.1	0.013	95.797	95.829	0.69	0.19	0.14	0.72	-0.07	1.1	93.78	93.77	2.02	0.14	0.72	-0.03	12.0	93.81	93.81	1.98	0.01	0.05	0.38	108.8	94.22	94.22	1.57
302	303	93.164	93.153	0.600	N/A	11.0	0.1	0.013	95.829	95.808	0.69	0.19	0.14	0.72	0.01	1.1	93.77	93.77	2.05	0.13	0.67	0.05	12.0	93.81	93.81	2.02	0.01	0.05	0.46	108.9	94.22	94.22	1.61
303	304	93.123	93.055	0.600	N/A	67.5	0.1	0.013	95.808	95.729	0.69	0.19	0.14	0.72	0.05	1.1	93.77	93.77	2.03	0.13	0.67	0.09	12.1	93.81	93.81	2.00	0.01	0.05	0.50	109.0	94.22	94.22	1.59
304	305	92.980	92.906	0.675	N/A	73.5	0.1	0.013	95.729	95.410	0.74	0.27	0.24	0.90	0.12	1.1	93.77	93.77	1.96	0.23	0.87	0.16	12.1	93.81	93.81	1.92	0.01	0.04	0.57	109.0	94.22	94.22	1.51
305	306	92.681	92.605	0.900	N/A	76.5	0.1	0.013	95.410	95.300	0.90	0.57	0.50	0.87	0.19	1.1	93.77	93.77	1.64	0.48	0.84	0.23	12.1	93.81	93.81	1.60	0.03	0.05	0.64	108.9	94.22	94.22	1.19
306	307	92.545	92.465	0.900	1.800	79.5	0.1	0.013	95.300	94.860	1.09	1.77	0.79	0.45	0.33	1.1	93.77	93.77	1.53	0.73	0.41	0.36	12.0	93.81	93.81	1.49	0.05	0.03	0.78	108.9	94.22	94.22	1.08
307	308	92.445	92.368	0.900	1.800	77.0	0.1	0.013	94.860	94.823	1.09	1.77	1.18	0.67	0.43	1.1	93.77	93.77	1.09	1.09	0.62	0.47	12.2	93.81	93.81	1.05	0.07	0.04	0.88	109.0	94.22	94.22	0.64
308	407	92.338	92.244	0.900	1.800	94.5	0.1	0.013	94.823	95.199	1.09	1.77	1.14	0.65	0.53	1.1	93.77	93.77	1.05	1.08	0.61	0.57	12.2	93.81	93.81	1.01	0.07	0.04	0.98	109.0	94.22	94.22	0.60
401	402	92.920	92.891	0.675	N/A	26.5	0.1	0.013	95.844	95.746	0.78	0.28	0.24	0.86	0.18	1.1	93.78	93.78	2.07	0.23	0.82	0.22	12.0	93.81	93.81	2.03	0.01	0.04	0.63	109.0	94.22	94.22	1.62
402	403	92.816	92.805	0.750	N/A	11.0	0.1	0.013	95.746	95.689	0.80	0.35	0.24	0.68	0.21	1.1	93.78	93.78	1.97	0.23	0.65	0.25	12.0	93.81	93.81	1.93	0.01	0.03	0.66	108.9	94.22	94.22	1.52
403	404	92.775	92.695	0.750	N/A	80.0	0.1	0.013	95.689	95.577	0.80	0.35	0.24	0.68	0.25	1.1	93.78	93.77	1.91	0.22	0.62	0.29	12.1	93.81	93.81	1.88	0.01	0.03	0.70	109.0	94.22	94.22	1.47
404	405	92.620	92.513	0.825	N/A	106.5	0.1	0.013	95.577	95.150	0.85	0.45	0.42	0.93	0.33	1.1	93.77	93.77	1.80	0.39	0.86	0.36	12.1	93.81	93.81	1.77	0.03	0.07	0.78	109.0	94.22	94.22	1.36
405	406	92.438	92.350	0.900	N/A	68.0	0.1	0.013	95.150	95.005	1.03	0.65	0.70	1.07	0.44	1.1	93.77	93.77	1.38	0.65	1.00	0.47	12.1	93.81	93.81	1.34	0.04	0.06	0.88	109.0	94.22	94.22	0.93
406	407	92.330	92.244	0.900	N/A	66.5	0.1	0.013	95.005	95.199	1.03	0.65	0.69	1.06	0.54	1.1	93.77	93.77	1.23	0.64	0.98	0.58	12.1	93.81	93.81	1.19	0.04	0.06	0.99	109.0	94.22	94.22	0.78
407	408	92.214	92.196	0.900	1.800	17.5	0.1	0.013	95.199	94.558	1.09	1.77	1.77	1.00	0.66	1.1	93.77	93.77	1.43	1.71	0.97	0.70	12.2	93.81	93.81	1.39	0.12	0.07	1.11	109.0	94.22	94.22	0.98
408	Pond1	92.166	92.150	0.900	1.800	16.5	0.1	0.013	94.558	94.500	1.09	1.77	1.76	1.00	0.70	1.1	93.77	93.77	0.79	1.71	0.97	0.74	12.2	93.81	93.81	0.75	0.12	0.07	1.16	109.0	94.22	94.22	0.34
500	501	93.525	93.444	0.750	N/A	81.0	0.1	0.013	96.026	95.903	0.80	0.35	0.24	0.68	-0.25	1.1	94.03	93.98	2.00	0.23	0.65	-0.13	12.0	94.15	94.12	1.88	0.01	0.03	-0.05	108.9	94.22	94.22	1.80
501	502	93.424	93.352	0.750	N/A	71.5	0.1	0.013	95.903	95.772	0.80	0.35	0.23	0.65	-0.19	1.2	93.98	93.94	1.92	0.21	0.60	-0.06	12.0	94.12	94.09	1.78	0.01	0.03	0.05	109.1	94.22	94.22	1.68
502	503	93.277	93.266	0.825	N/A	11.0	0.1	0.013	95.772	95.653	0.85	0.45	0.33	0.73	-0.16	1.2	93.94	93.92	1.83	0.32	0.70	-0.01	12.2	94.09	94.08	1.68	0.02	0.04	0.12	109.0	94.22	94.22	1.55
503	504	93.236	93.175	0.825	N/A	61.0	0.1	0.013	95.653	95.567	0.85	0.45	0.34	0.75	-0.14	1.3	93.92	93.88	1.73	0.33	0.73	0.02	12.2	94.08	94.05	1.57	0.02	0.04	0.16	109.0	94.22	94.22	1.43
504	505	93.145	93.134	0.825	N/A	11.0	0.1	0.013	95.567	95.525	0.85	0.45	0.35	0.77	-0.09	1.3	93.88	93.87	1.69	0.34	0.75	0.08	12.2	94.05	94.04	1.52	0.02	0.04	0.25	109.0	94.22	94.22	1.34

Table 6: Pipe Data and Hydraulic Simulation Results (1)

U/S MH	D/S MH	U/S Invert (m)	D/S Invert (m)	Pipe Diameter / Height (m)	Pipe Width (m)	Pipe Length (m)	Pipe Slope (%)	n	U/S MH Cover Elev. (m)	D/S MH Cover Elev. (m)	Design Velocity (m/s)	Design Flow (m^3/s)	100-year, 3-hour Chicago Storm							100-year, 12-hour SCS Type II Storm							100-year, 10-day Spring Snowmelt + Rainfall Event						
													Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S (2) (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S (2) (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S (2) (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)
505	506	93.104	93.068	0.825	N/A	36.0	0.1	0.013	95.525	95.404	0.85	0.45	0.36	0.79	-0.06	1.3	93.87	93.82	1.66	0.35	0.77	0.11	12.2	94.04	94.00	1.49	0.02	0.04	0.30	109.1	94.22	94.22	1.30
506	507	92.993	92.917	0.900	N/A	76.0	0.1	0.013	95.404	95.290	0.90	0.57	0.52	0.91	-0.07	1.2	93.82	93.77	1.58	0.49	0.86	0.11	12.2	94.00	93.95	1.41	0.03	0.05	0.33	109.0	94.22	94.22	1.18
507	508	92.767	92.687	1.050	N/A	79.5	0.1	0.013	95.290	95.170	1.00	0.86	0.86	1.00	-0.04	1.2	93.77	93.77	1.52	0.80	0.93	0.13	12.2	93.95	93.83	1.34	0.05	0.06	0.41	108.9	94.22	94.22	1.07
508	509	92.627	92.597	1.050	N/A	30.0	0.1	0.013	95.170	95.121	1.00	0.86	0.88	1.02	0.10	1.2	93.77	93.77	1.40	0.82	0.95	0.15	12.2	93.83	93.81	1.34	0.06	0.07	0.55	109.0	94.22	94.22	0.95
509	510	92.447	92.374	1.200	N/A	72.5	0.1	0.013	95.121	95.016	1.09	1.23	1.11	0.90	0.13	1.2	93.77	93.77	1.35	1.04	0.84	0.16	12.1	93.81	93.81	1.31	0.07	0.06	0.58	109.0	94.22	94.22	0.90
510	511	92.354	92.267	1.200	N/A	86.5	0.1	0.013	95.016	94.413	1.09	1.23	1.25	1.01	0.22	1.2	93.77	93.77	1.24	1.16	0.94	0.26	12.1	93.81	93.81	1.21	0.08	0.06	0.67	109.0	94.22	94.22	0.79
511	611	92.207	92.100	1.200	N/A	106.5	0.1	0.013	94.413	94.667	1.09	1.23	1.41	1.14	0.37	1.2	93.77	93.77	0.64	1.31	1.06	0.40	12.1	93.81	93.81	0.60	0.09	0.07	0.81	109.0	94.22	94.22	0.19
601	602	93.362	93.294	0.600	N/A	68.5	0.1	0.013	96.029	95.603	0.69	0.19	0.17	0.88	-0.18	1.1	93.78	93.78	2.25	0.17	0.88	-0.13	12.0	93.83	93.81	2.20	0.01	0.05	0.26	109.0	94.22	94.22	1.81
602	603	93.264	93.253	0.600	N/A	11.0	0.1	0.013	95.603	95.530	0.69	0.19	0.17	0.88	-0.09	1.1	93.78	93.78	1.83	0.16	0.82	-0.05	12.0	93.81	93.81	1.79	0.01	0.05	0.36	108.6	94.22	94.22	1.38
603	604	93.103	93.035	0.750	N/A	68.5	0.1	0.013	95.530	95.400	0.80	0.35	0.26	0.74	-0.08	1.1	93.78	93.78	1.75	0.24	0.68	-0.04	12.1	93.81	93.81	1.72	0.02	0.06	0.37	109.0	94.22	94.22	1.31
604	605	92.885	92.809	0.900	N/A	76.5	0.1	0.013	95.400	95.290	0.90	0.57	0.46	0.80	-0.01	1.1	93.78	93.78	1.62	0.43	0.75	0.03	12.1	93.81	93.81	1.59	0.03	0.05	0.44	109.0	94.22	94.22	1.18
605	606	92.734	92.654	0.975	N/A	79.5	0.1	0.013	95.290	95.170	0.95	0.71	0.65	0.92	0.07	1.1	93.78	93.77	1.52	0.61	0.86	0.10	12.1	93.81	93.81	1.48	0.04	0.06	0.51	108.8	94.22	94.22	1.07
606	607	92.579	92.499	1.050	N/A	79.5	0.1	0.013	95.170	95.050	1.00	0.86	0.88	1.02	0.15	1.1	93.77	93.77	1.40	0.82	0.95	0.18	12.1	93.81	93.81	1.36	0.05	0.06	0.59	109.0	94.22	94.22	0.95
607	608	92.479	92.410	1.050	N/A	69.0	0.1	0.013	95.050	94.944	1.00	0.86	0.97	1.12	0.24	1.1	93.77	93.77	1.28	0.90	1.04	0.28	12.1	93.81	93.81	1.24	0.06	0.07	0.69	109.0	94.22	94.22	0.83
608	609	92.380	92.369	1.050	N/A	11.0	0.1	0.013	94.944	94.835	1.00	0.86	0.96	1.11	0.34	1.1	93.77	93.77	1.17	0.89	1.03	0.38	12.1	93.81	93.81	1.13	0.06	0.07	0.79	109.0	94.22	94.22	0.72
609	610	92.219	92.185	1.200	N/A	34.5	0.1	0.013	94.835	94.875	1.09	1.23	1.10	0.89	0.35	1.1	93.77	93.77	1.06	1.02	0.83	0.39	12.1	93.81	93.81	1.02	0.07	0.06	0.80	108.9	94.22	94.22	0.61
610	611	92.125	92.100	1.200	N/A	25.0	0.1	0.013	94.875	94.667	1.09	1.23	1.32	1.07	0.45	1.1	93.77	93.77	1.10	1.24	1.01	0.48	12.1	93.81	93.81	1.07	0.08	0.06	0.90	109.0	94.22	94.22	0.65
611	612	92.070	92.048	1.200	1.800	22.0	0.1	0.013	94.667	94.113	1.23	2.66	2.70	1.02	0.50	1.1	93.77	93.77	0.90	2.53	0.95	0.54	12.1	93.81	93.81	0.86	0.17	0.06	0.95	109.0	94.22	94.22	0.45
612	Pond1	92.018	92.000	1.200	1.800	18.0	0.1	0.013	94.113	94.500	1.23	2.66	2.68	1.01	0.55	1.1	93.77	93.77	0.34	2.52	0.95	0.59	12.1	93.81	93.81	0.30	0.17	0.06	1.00	109.0	94.22	94.22	-0.11
700	701	93.517	93.443	0.975	N/A	74.0	0.1	0.013	96.786	96.740	0.95	0.71	0.64	0.90	0.63	1.1	95.12	94.96	1.66	0.65	0.92	0.73	12.1	95.22	95.11	1.57	0.06	0.08	-0.26	0.0	94.23	94.23	2.56
701	702	93.368	93.328	1.050	N/A	40.0	0.1	0.013	96.740	96.530	1.00	0.86	0.80	0.93	0.55	1.1	94.96	94.91	1.78	0.81	0.94	0.69	12.1	95.11	95.08	1.63	0.07	0.08	-0.19	0.0	94.23	94.23	2.51
702	703	93.178	93.106	1.200	1.800	71.5	0.1	0.013	96.530	96.420	1.23	2.66	1.62	0.61	0.53	1.1	94.91	94.86	1.62	1.64	0.62	0.70	12.1	95.08	95.05	1.45	0.10	0.04	-0.15	109.0	94.23	94.23	2.30
703	704	93.086	92.980	1.200	1.800	105.5	0.1	0.013	96.420	96.260	1.23	2.66	2.13	0.80	0.57	1.1	94.86	94.75	1.56	2.17	0.82	0.77	12.1	95.05	94.98	1.37	0.13	0.05	-0.06	109.0	94.23	94.23	2.19
704	705	92.960	92.888	1.200	1.800	71.5	0.1	0.013	96.260	96.150	1.23	2.66	2.95	1.11	0.59	1.1	94.75	94.64	1.51	2.98	1.12	0.82	12.1	94.98	94.88	1.28	0.18	0.07	0.07	109.0	94.23	94.23	2.03
705	706	92.868	92.801	1.200	2.400	67.0	0.1	0.013	96.150	95.902	1.32	3.80	4.35	1.14	0.57	1.1	94.64	94.54	1.51	4.26	1.12	0.81	12.1	94.88	94.79	1.27	0.26	0.07	0.16	109.0	94.23	94.23	1.92
706	707	92.781	92.697	1.200	2.400	83.5	0.1	0.013	95.902	95.430	1.32	3.80	4.47	1.18	0.56	1.1	94.54	94.27	1.36	4.38	1.15	0.80	12.1	94.79	94.51	1.12	0.27	0.07	0.24	109.0	94.23	94.23	1.68
707	708	92.637	92.567	1.200	2.400	70.0	0.1	0.013	95.430	95.320	1.32	3.80	5.02	1.32	0.43	1.1	94.27	94.12	1.16	4.90	1.29	0.67	12.1	94.51	94.36	0.92	0.30	0.08	0.39	109.0	94.23	94.23	1.21
708	709	92.547	92.481	1.200	2.400	66.0	0.1	0.013	95.320	95.221	1.32	3.80	5.22	1.37	0.37	1.1	94.12	93.97	1.20	5.09	1.34	0.61	12.1	94.36	94.20	0.96	0.32	0.08	0.48	109.0	94.23	94.22	1.10
709	710	92.461	92.391	1.200	2.400	70.5	0.1	0.013	95.221	95.112	1.32	3.80	5.22	1.37	0.31	1.1	93.97	93.77	1.25	5.09	1.34	0.54	12.1	94.20	93.82	1.02	0.32	0.08	0.56	109.0	94.22	94.22	1.00
710	711	92.331	92.200	1.200	2.400	131.0	0.1	0.013	95.112	94.385	1.32	3.80	5.57	1.46	0.24	1.1	93.77	93.77	1.34	5.46	1.44	0.29	12.1	93.82	93.81	1.29	0.34	0.09	0.69	109.0	94.22	94.22	0.89
711	712	92.140	92.118	1.200	2.400	21.5	0.1	0.013	94.385	94.000	1.32	3.80	5.50	1.45	0.43	1.1	93.77	93.79	0.61	5.44	1.43	0.47	12.1	93.81	93.81	0.58	0.34	0.09	0.88	109.0	94.22	94.22	0.16
712	Pond1	92.058	92.050	1.200	2.400	8.5	0.1	0.013	94.000	94.500	1.32	3.80	5.50	1.45	0.53	1.1	93.79	93.77	0.21	5.42	1.43	0.55	12.1	93.81	93.81	0.19	-0.37	-0.10	0.96	196.2	94.22	94.22	-0.22
751	752	93.846	93.752	0.600	N/A	94.0	0.1	0.013	96.720	96.579	0.69	0.19	0.17	0.88	0.78	1.1	95.23	95.02	1.49	0.15	0.77	0.57	12.1	95.02	94.95	1.70	0.02	0.10	-0.22	0.1	94.23	94.23	2.49
752	753	93.602	93.520	0.750	N/A	82.0	0.1	0.013	96.579	96.452	0.80	0.35	0.28	0.80	0.67	1.1	95.02	94.93	1.56	0.27	0.77	0.60	12.1	94.95	94.97	1.63	-0.04	-0.11	-0.13	0.2	94.23	94.23	2.35
753	754	93.295	93.209	0.975	N/A	86.0	0.1	0.013	96.452	96.324	0.95	0.71	0.77	1.09	0.66	1.1	94.93	94.82	1.53	0.75	1.06	0.70	12.1	94.97	94.96	1.49	0.04	0.06	-0.04	109.0	94.23	94.23	2.23
754	705	93.134	93.018	1.050	N/A	116.0	0.1	0.013	96.324	96.150	1.00	0.86	0.93	1.08	0.64	1.1	94.82	94.64	1.51	0.92	1.07	0.78	12.1	94.96	94.88	1.36	0.05	0.06	0.04	109.0	94.23	94.23	2.10
806	807	93.069	93.047	0.600	N/A	21.5	0.1	0.013	95.186	95.180	0.69	0.19	0.06	0.31	0.10	1.1	93.77	93.77	1.41	0.05	0.26	0.22	12.1	93.89	93.89	1.29	-0.01	-0.05	0.56	190.2	94.23	94.22	0.96
807	808	92.897	92.861	0.750	N/A	35.5	0.1	0.013	95.180	95.133	0.80	0.35	0.30	0.85	0.13	1.1	93.77	93.77	1.41	0.27	0.77	0.24	12.1	93.89	93.87	1.29	0.02	0.06	0.58	108.9	94.22	94.22	0.96
808	809	92.786	92.775	0.825	N/A	11.0	0.1	0.013	95.133	95.095	0.85	0.45	0.43	0.95	0.16	1.1	93.77	93.77	1.36	0.39	0.86	0.26	12.1	93.87	93.86	1.26	0.03	0.07	0.61	108.9	94.22	94.22	0.91
809	710	92.725	92.706	0.825	N/A	18.5	0.1	0.013	95.095	95.112	0.85	0.45	0.44	0.97	0.22	1.1	93.77	93.77	1.32	0.39	0.86	0.31	12.1	93.86	93.82	1.24	0.03	0.07	0.67	190.2	94.22	94.22	0.87
901	902	94.501	94.436	0.600	N/A	65.0	0.1	0.013	96.268	96.259	0.69	0.19	0.12	0.62	-0.16	1.1	94.94	94.91	1.33	0.12	0.62	-0.17	12.0	94.93	94.90	1.34	0.01	0.05	-0.25	108.0	94.85	94.85	1.42
902	903	94.416	94.346	0.675	N/A	70.0	0.1	0.013	96.259	96.151	0.74	0.27	0.21	0.79	-0.18	1.1	94.91	94.85	1.35	0.21	0.79	-0.19	12.1	94.90	94.84	1.36	0.01	0.04	-0.24	107.4	94.85	94.85	1.41
903	904	94.271	94.201	0.750	N/A	69.5	0.1	0.013	96.151	96.259	0.80	0.35	0.29	0.82	-0.17	1.1	94.85	94.75	1.30	0.28	0.80	-0.18	12.1	94.84	94.74	1.31	0.01	0.03	-0.17	109.1	94.85	94.85	1.30
904	905	94.121	94.005	0.900	N/A	116.0	0.1	0.013	96.259	96.162	0.90	0.57	0.56	0.98	-0.27	1.1	94.75	94.52	1.51	0.55	0.96	-0.28	12.1	94.74	94.51	1.52	0.03	0.05	-0.17	109.1	94.85	94.85	1.41
905	1007	93.955	93.939	0.900	N/A	16.0	0.1	0.013	96.162	96.196	0.90	0.57	0.56	0.98	-0.34	1.1	94.52	94.31	1.65	0.55	0.96	-0.35	12.1	94.51	94.30	1.65	0.03	0.05	0.00	111.0	94.85	94.85	1.31
1001	1002	94.618	94.545	0.525	N/A	72.5	0.1	0.013	96.638	96.632	0.63	0.14	0.11	0.81	-0.14	1.1	95.00	94.90	1.64	0.11	0.81	-0.15	12.0	94.99	94.89	1.65	0.01	0.07	-0.29	108.4	94.85	94.85	1.79
1002	1003	94.465	94.388	0.600	N/A	77.0	0.1	0.013	96.632	96.593	0.69	0.19	0.17	0.88	-0.16	1.1	94.90	94.82	1.73	0.17	0.88	-0.17	12.1	94.89	94.81	1.74	0.01	0.05	-0.21	108.1	94.85	94.85	1.78
1003	1004	94.238	94.166	0.750	N/A	71.5	0.1	0.013	96.593	96.490	0.80	0.35	0.34	0.97	-0.17	1.1	94.82	94.74	1.77	0.34	0.97	-0.18	12.1	94.81	94.73	1.79	0.02	0.06	-0.14	108.1	94.85	94.85	1.74
1004	1005	94.016	93.942	0.900	N/A	74.0	0.1	0.013	96.490	96.380	0.90	0.57	0.51	0.89	-0.18	1.1	94.74	94.61	1.75	0.50	0.87	-0.19	12.1	94.73	94.59	1.76	0.03	0.05	-0.06	109.0	94.85	94.85	1.64
1005	1006	93.862	93.791	0.975	N/A	71.0	0.1	0.013	96.380	96.270	0.95	0.71	0.63	0.89	-0.23	1.2	94.61	94.44	1.77	0.61	0.86	-0.24	12.1	94.59	94.43	1.79	0.03	0.04	0.02	109.1	94.85	94.85	1.53
1006	1007	93.566	93.482	1.200	N/A	83.5	0.1	0.013	96.270	96.196	1.09	1.23	1.24	1.01	-0.33	1.1	94.44	94.31	1.83	1.22	0.99	-0.34	12.1	94.43	94.30	1.84	0.07	0.06	0.09	109.1	94.85	94.85	1.42
1007	1008	93.361	93.303	1.200	N/A	29.0	0.2	0.013	96.196	96.057	1.54	1.74	1.84	1.06	-0.25	1.2	94.31	94.04	1.88	1.80	1.03	-0.26	12.1	94.30	94.21	1.89	0.10	0.06	0.29	111.0	94.85	94.85	1.34
1008	Pond2	93.223	93.200	1.200	2.400	23.0	0.1	0.013	96.057	96.000	1.32	3.80	2.61	0.69	-0.38	1.1	94.04	94.04	2.01	2.55	0.67	-0.22	12.1	94.21	94.20	1.85	1.41	0.37	0.43	111.0	94.85	94.84	1.20
1101	1102	95.258	95.139	0.750	N/A	119.0	0.1	0.013	97.798	97.953	0.80	0.35	0.25	0.71	-0.29	1.1	95.72	95.57	2.08	0.24	0.68	-0.29	12.0	95.71	95.56	2.08	0.01	0.03	-0.64	109.0	95.37	95.20	2.43
1102	1103	95.109	95.101	0.750	N/A	8.0	0.1	0.013	97.953	97.864	0.80	0.35	0.25	0.71	-0.29	1.1	95.57	95.54	2.38	0.24	0.68	-0.30	12.1	95.56	95.53	2.39	0.01	0.03	-0.66	109.0	95.20	95.08	2.75
1103	1104	94.951	94.885	0.900	N/A	65.5	0.1	0.013	97.864	98.100	0.90	0.57	0.43	0.75	-0.31	1.1	95.54	95.48	2.32	0.42	0.73	-0.32	12.1	95.53	95.47	2.33	0.03	0.05	-0.77	109.0	95.08	94.98	2.78

Table 6: Pipe Data and Hydraulic Simulation Results [1]

U/S MH	D/S MH	U/S Invert (m)	D/S Invert (m)	Pipe Diameter / Height (m)	Pipe Width (m)	Pipe Length (m)	Pipe Slope (%)	n	U/S MH Cover Elev. (m)	D/S MH Cover Elev. (m)	Design Velocity (m/s)	Design Flow (m^3/s)	100-year, 3-hour Chicago Storm							100-year, 12-hour SCS Type II Storm							100-year, 10-day Spring Snowmelt + Rainfall Event						
													Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)
1104	1105	94.825	94.753	0.900	N/A	71.5	0.1	0.013	98.100	98.730	0.90	0.57	0.52	0.91	-0.25	1.1	95.48	95.27	2.62	0.51	0.89	-0.25	12.1	95.47	95.26	2.63	0.03	0.05	-0.75	109.0	94.98	94.85	3.13
1105	1106	94.673	94.619	0.900	N/A	54.5	0.1	0.013	98.730	98.710	0.90	0.57	0.52	0.91	-0.31	1.2	95.27	95.14	3.46	0.51	0.89	-0.31	12.1	95.26	95.13	3.47	0.03	0.05	-0.73	109.1	94.85	94.84	3.89
1106	1107	94.469	94.462	1.050	N/A	7.0	0.1	0.013	98.710	98.708	1.00	0.86	0.93	1.08	-0.38	1.1	95.14	95.03	3.57	0.91	1.05	-0.39	12.1	95.13	95.02	3.58	0.06	0.07	-0.68	109.0	94.84	94.84	3.87
1107	1108	94.432	93.978	1.050	N/A	113.5	0.4	0.013	98.708	96.666	1.99	1.73	0.92	0.53	-0.45	1.1	95.03	94.78	3.68	0.90	0.52	-0.46	12.1	95.02	94.77	3.69	0.05	0.03	-0.64	109.0	94.84	94.84	3.87
1108	1109	93.528	93.381	1.500	N/A	73.5	0.2	0.013	96.666	96.511	1.79	3.16	4.22	1.33	-0.24	1.2	94.78	94.24	1.88	4.15	1.31	-0.26	12.1	94.77	94.26	1.90	0.24	0.08	-0.19	109.1	94.84	94.84	1.83
1109	1110	93.351	93.283	1.500	2.400	67.5	0.1	0.013	96.511	95.479	1.45	5.23	4.22	0.81	-0.62	1.2	94.24	94.04	2.28	4.14	0.79	-0.59	12.1	94.26	94.20	2.25	0.24	0.05	-0.01	109.1	94.84	94.84	1.67
1110	Pond2	93.223	93.200	1.500	2.400	22.5	0.1	0.013	95.479	96.000	1.45	5.23	4.21	0.80	-0.69	1.2	94.04	94.04	1.44	4.11	0.79	-0.52	12.1	94.20	94.20	1.28	0.24	0.05	0.12	109.1	94.84	94.84	0.64
1201	1202	95.554	95.502	0.825	N/A	51.5	0.1	0.013	98.081	98.047	0.85	0.45	0.34	0.75	-0.28	1.1	96.10	96.04	1.98	0.34	0.75	-0.30	12.0	96.08	96.02	2.00	0.02	0.04	-0.70	109.0	95.68	95.58	2.41
1202	1203	95.472	95.461	0.825	N/A	11.0	0.1	0.013	98.047	98.014	0.85	0.45	0.34	0.75	-0.25	1.1	96.04	96.02	2.00	0.33	0.73	-0.28	12.0	96.02	95.99	2.03	0.02	0.04	-0.72	109.0	95.58	95.45	2.47
1203	1204	95.311	95.243	0.975	N/A	67.5	0.1	0.013	98.014	97.847	0.95	0.71	0.51	0.72	-0.27	1.1	96.02	95.98	1.99	0.50	0.71	-0.30	12.0	95.99	95.95	2.02	0.03	0.04	-0.84	109.0	95.45	95.36	2.56
1204	1205	95.223	95.149	0.975	N/A	67.5	0.1	0.013	97.847	97.741	1.00	0.74	0.49	0.66	-0.21	1.1	95.98	95.94	1.86	0.49	0.66	-0.25	12.2	95.95	95.90	1.90	0.03	0.04	-0.84	109.1	95.36	95.26	2.49
1205	1206	95.119	95.108	0.975	N/A	11.0	0.1	0.013	97.741	97.718	0.95	0.71	0.60	0.85	-0.15	1.1	95.94	95.92	1.80	0.59	0.83	-0.19	12.1	95.90	95.87	1.84	0.04	0.06	-0.83	109.0	95.26	95.23	2.48
1206	1207	95.078	95.022	0.975	N/A	56.0	0.1	0.013	97.718	97.549	0.95	0.71	0.61	0.86	-0.14	1.3	95.92	95.87	1.80	0.59	0.83	-0.18	12.2	95.87	95.82	1.85	0.04	0.06	-0.82	109.0	95.23	95.13	2.49
1207	1208	94.992	94.981	0.975	N/A	11.0	0.1	0.013	97.549	97.613	0.95	0.71	0.62	0.87	-0.10	1.3	95.87	95.85	1.68	0.61	0.86	-0.15	12.2	95.82	95.79	1.73	0.04	0.06	-0.83	109.1	95.13	95.10	2.42
1208	1209	94.951	94.915	0.975	N/A	36.0	0.1	0.013	97.613	97.564	0.95	0.71	0.63	0.89	-0.08	1.3	95.85	95.81	1.77	0.62	0.87	-0.13	12.2	95.79	95.76	1.82	0.04	0.06	-0.82	109.1	95.10	94.99	2.51
1209	1210	94.840	94.832	1.050	N/A	7.5	0.1	0.013	97.564	97.551	1.00	0.86	0.75	0.87	-0.08	1.3	95.81	95.79	1.75	0.72	0.83	-0.13	12.2	95.76	95.74	1.81	0.05	0.06	-0.90	109.0	94.99	94.96	2.57
1210	1211	94.802	94.773	1.050	N/A	29.0	0.1	0.013	97.551	96.869	1.00	0.86	0.76	0.88	-0.06	1.3	95.79	95.76	1.76	0.73	0.85	-0.12	12.2	95.74	95.70	1.82	0.05	0.06	-0.89	109.0	94.96	94.89	2.59
1211	1212	94.743	94.735	1.050	N/A	7.5	0.1	0.013	96.869	96.861	1.00	0.86	0.77	0.89	-0.03	1.3	95.76	95.74	1.11	0.74	0.86	-0.09	12.2	95.70	95.68	1.17	0.05	0.06	-0.90	109.0	94.89	94.87	1.97
1212	1213	94.705	94.641	1.050	N/A	64.0	0.1	0.013	96.861	97.400	1.00	0.86	0.80	0.93	-0.01	1.3	95.74	95.70	1.12	0.76	0.88	-0.07	12.2	95.68	95.65	1.18	0.05	0.06	-0.89	109.1	94.87	94.84	1.99
1213	1214	94.491	94.419	1.200	N/A	71.5	0.1	0.013	97.400	97.290	1.09	1.23	1.09	0.88	0.01	1.3	95.70	95.65	1.70	1.05	0.85	-0.05	12.2	95.65	95.60	1.76	0.07	0.06	-0.85	109.0	94.84	94.84	2.56
1214	1215	94.399	94.327	1.200	N/A	71.5	0.1	0.013	97.290	97.190	1.09	1.23	1.24	1.01	0.05	1.2	95.65	95.50	1.64	1.21	0.98	0.00	12.2	95.60	95.45	1.69	0.08	0.06	-0.76	109.0	94.84	94.84	2.45
1215	1216	94.177	94.105	1.350	N/A	71.5	0.1	0.013	97.190	97.080	1.18	1.69	1.64	0.97	-0.03	1.2	95.50	95.44	1.69	1.62	0.96	-0.08	12.2	95.45	95.39	1.74	0.10	0.06	-0.69	109.0	94.84	94.84	2.35
1216	1217	93.955	93.871	1.500	N/A	84.5	0.1	0.013	97.080	96.955	1.26	2.24	2.22	0.99	-0.02	1.2	95.44	95.35	1.65	2.19	0.98	-0.06	12.2	95.39	95.32	1.69	0.13	0.06	-0.61	109.0	94.84	94.84	2.24
1217	1218	93.851	93.768	1.500	N/A	83.0	0.1	0.013	96.955	96.830	1.26	2.24	2.47	1.10	0.00	1.2	95.35	95.25	1.60	2.45	1.10	-0.03	12.2	95.32	95.22	1.64	0.15	0.07	-0.51	109.1	94.84	94.84	2.11
1218	1219	93.748	93.702	1.500	N/A	46.5	0.1	0.013	96.830	96.760	1.26	2.24	2.55	1.14	0.00	1.2	95.25	95.19	1.58	2.53	1.13	-0.02	12.2	95.22	95.17	1.61	0.15	0.07	-0.41	109.1	94.84	94.84	1.99
1219	1108	93.682	93.608	1.500	N/A	74.0	0.1	0.013	96.760	96.666	1.26	2.24	3.03	1.36	0.00	1.2	95.19	94.78	1.57	2.99	1.34	-0.02	12.2	95.17	94.77	1.59	0.18	0.08	-0.34	109.1	94.84	94.84	1.92
1600	1601	94.197	94.025	1.200	N/A	107.5	0.2	0.013	97.015	96.875	1.38	1.56	1.60	1.03	-0.14	3.4	95.26	95.09	1.76	2.02	1.30	0.59	14.0	95.99	95.70	1.03	1.31	0.84	0.16	111.2	95.55	95.44	1.46
1601	1602	94.025	93.873	1.200	N/A	95.0	0.2	0.013	96.875	96.627	1.38	1.56	1.60	1.03	-0.13	3.4	95.09	94.95	1.78	2.02	1.30	0.47	14.0	95.70	95.44	1.18	1.31	0.84	0.22	111.2	95.44	95.34	1.43
1602	1603	93.873	93.721	1.200	N/A	95.0	0.2	0.013	96.627	96.665	1.38	1.56	1.60	1.03	-0.12	3.4	94.95	94.81	1.68	2.02	1.30	0.37	14.0	95.44	95.18	1.19	1.32	0.85	0.27	111.0	95.34	95.25	1.29
1603	1604	93.721	93.569	1.200	N/A	95.0	0.2	0.013	96.665	96.891	1.38	1.56	1.60	1.03	-0.11	3.4	94.81	94.67	1.86	2.02	1.30	0.26	14.0	95.18	94.95	1.48	1.33	0.85	0.32	111.0	95.25	95.13	1.42
1604	1605	93.569	93.431	1.200	N/A	86.5	0.2	0.013	96.891	96.700	1.38	1.56	1.60	1.03	-0.10	3.4	94.67	94.32	2.22	2.02	1.30	0.18	14.1	94.95	94.50	1.95	1.33	0.85	0.36	111.0	95.13	94.97	1.76
1605	1008	93.431	93.316	1.200	N/A	72.0	0.2	0.013	96.700	96.057	1.38	1.56	1.60	1.03	-0.31	3.4	94.32	94.04	2.38	2.02	1.30	-0.14	14.1	94.50	94.21	2.21	1.33	0.85	0.34	111.0	94.97	94.85	1.73
7000	700	93.664	93.592	0.900	N/A	71.5	0.1	0.013	96.997	96.786	0.90	0.57	0.52	0.91	0.67	1.1	95.23	95.12	1.77	0.52	0.91	0.71	12.1	95.27	95.22	1.72	0.07	0.12	-0.34	0.0	94.23	94.23	2.77
Pond1	Out1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	2.17	N/A	N/A	N/A	N/A	N/A	N/A	3.67	N/A	N/A	N/A	N/A	N/A	N/A	2.91	N/A	N/A	N/A	N/A	N/A	N/A
Pond2	Out2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.56	N/A	N/A	N/A	N/A	N/A	N/A	2.06	N/A	N/A	N/A	N/A	N/A	N/A	1.46	N/A	N/A	N/A	N/A	N/A	N/A
1500	1501	92.698	92.577	1.200	1.800	120.0	0.1	0.013	95.914	95.720	1.23	2.66	2.56	0.96	0.16	3.0	94.06	93.95	1.86	3.20	1.20	0.41	13.8	94.31	94.13	1.61	1.87	0.70	0.49	110.6	94.39	94.33	1.53
1501	21400	92.577	92.450	1.200	1.800	127.0	0.1	0.013	95.720	95.562	1.23	2.66	2.56	0.96	0.17	3.0	93.95	93.82	1.77	3.20	1.20	0.35	13.8	94.13	93.91	1.59	1.87	0.70	0.55	110.6	94.33	94.26	1.39
21400	Pond1	92.420	92.350	1.200	1.800	70.0	0.1	0.013	95.562	94.500	1.23	2.66	2.56	0.96	0.20	3.0	93.82	93.77	1.74	3.20	1.20	0.29	13.8	93.91	93.81	1.65	1.87	0.70	0.63	110.6	94.26	94.22	1.31

Note: [1] Based on restrictive downstream conditions; Summer = 93.68 m / Spring = 94.11 m for SWM Facility 1, and Spring = 94.18 m for Pond 2. All other results based on free outfall conditions.

[2] A negative surcharge implies that the pipe is not flowing full

0.21 Freeboard less than 0.5 m.

Table 6: Pipe Data and Hydraulic Simulation Results [1]

U/S MH	D/S MH	U/S Invert (m)	D/S Invert (m)	Pipe Diameter / Height (m)	Pipe Width (m)	Pipe Length (m)	Pipe Slope (%)	n	U/S MH Cover Elev. (m)	D/S MH Cover Elev. (m)	Design Velocity (m/s)	Design Flow (m^3/s)	100-year, 3-hour Chicago Storm + 20%							100-year, 12-hour SCS Type II Storm + 20%							July 1st, 1979 Historical Event						
													Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)
101	102	93.228	93.161	0.825	N/A	67.0	0.1	0.013	95.598	95.587	0.85	0.45	0.33	0.73	-0.23	1.1	93.83	93.83	1.77	0.31	0.68	-0.08	12.0	93.97	93.95	1.63	0.27	0.59	-0.24	1.7	93.81	93.81	1.79
102	103	93.141	93.061	0.825	N/A	80.0	0.1	0.013	95.587	95.470	0.85	0.45	0.33	0.73	-0.14	1.1	93.83	93.82	1.76	0.30	0.66	-0.01	12.2	93.95	93.93	1.63	0.28	0.62	-0.15	1.7	93.81	93.81	1.77
103	104	92.911	92.850	0.975	N/A	60.5	0.1	0.013	95.470	95.373	0.95	0.71	0.67	0.95	-0.06	1.1	93.82	93.82	1.65	0.61	0.86	0.04	12.0	93.93	93.88	1.54	0.56	0.79	-0.07	1.7	93.81	93.81	1.66
104	105	92.820	92.796	0.975	N/A	23.5	0.1	0.013	95.373	95.331	0.95	0.71	0.67	0.95	0.03	1.1	93.82	93.82	1.55	0.60	0.85	0.09	12.1	93.88	93.86	1.49	0.55	0.78	0.02	1.7	93.81	93.81	1.56
105	106	92.721	92.619	1.050	N/A	101.5	0.1	0.013	95.331	95.180	1.00	0.86	0.85	0.98	0.05	1.1	93.82	93.82	1.51	0.76	0.88	0.09	12.1	93.86	93.85	1.47	0.69	0.80	0.04	1.6	93.81	93.81	1.52
106	107	92.559	92.483	1.050	N/A	76.5	0.1	0.013	95.180	95.070	1.00	0.86	0.98	1.13	0.21	1.1	93.82	93.81	1.36	0.85	0.98	0.24	12.1	93.85	93.85	1.33	0.78	0.90	0.20	1.6	93.81	93.81	1.37
107	108	92.333	92.258	1.200	N/A	74.5	0.1	0.013	95.070	95.057	1.09	1.23	1.27	1.03	0.28	1.1	93.81	93.81	1.26	1.12	0.91	0.32	12.0	93.85	93.85	1.22	1.01	0.82	0.28	1.6	93.81	93.81	1.26
108	109	92.238	92.211	1.200	N/A	26.5	0.1	0.013	95.057	95.487	1.09	1.23	1.40	1.14	0.38	1.1	93.81	93.81	1.24	1.24	1.01	0.41	12.0	93.85	93.85	1.21	1.13	0.92	0.37	1.6	93.81	93.81	1.25
109	Pond1	92.181	92.150	1.200	N/A	31.0	0.1	0.013	95.487	94.500	1.09	1.23	1.39	1.13	0.43	1.1	93.81	93.81	1.68	1.23	1.00	0.47	12.0	93.85	93.85	1.64	1.11	0.90	0.43	1.6	93.81	93.81	1.68
202	203	93.826	93.706	0.675	N/A	120.0	0.1	0.013	96.212	96.030	0.74	0.27	0.27	1.02	0.84	1.1	95.34	95.07	0.87	0.24	0.90	1.27	12.0	95.77	95.69	0.45	0.20	0.75	-0.15	1.6	94.35	94.28	1.86
203	204	93.556	93.479	0.825	N/A	77.0	0.1	0.013	96.030	95.920	0.85	0.45	0.52	1.15	0.69	1.1	95.07	94.99	0.96	0.50	1.10	1.31	12.0	95.69	95.58	0.34	0.40	0.88	-0.11	1.6	94.28	94.23	1.76
204	205	93.404	93.332	0.900	N/A	72.0	0.1	0.013	95.920	95.812	0.90	0.57	0.65	1.14	0.68	1.1	94.99	94.90	0.93	0.65	1.14	1.27	12.0	95.58	95.40	0.34	0.49	0.86	-0.08	1.6	94.23	94.19	1.69
205	206	93.257	93.184	0.975	N/A	73.0	0.1	0.013	95.812	95.700	0.95	0.71	0.78	1.10	0.66	1.1	94.90	94.71	0.92	0.79	1.11	1.17	12.0	95.40	95.14	0.41	0.57	0.80	-0.05	1.6	94.19	94.12	1.63
206	207	92.959	92.925	1.200	1.800	34.0	0.1	0.013	95.700	95.898	1.23	2.66	2.80	1.05	0.55	1.1	94.71	94.65	0.99	2.73	1.03	0.98	12.0	95.14	95.07	0.56	2.10	0.79	-0.04	1.5	94.12	94.09	1.58
207	208	92.875	92.819	1.200	1.800	55.5	0.1	0.013	95.898	95.684	1.23	2.66	2.89	1.09	0.58	1.1	94.65	94.55	1.25	2.81	1.06	1.00	12.0	95.07	94.96	0.83	2.15	0.81	0.01	1.7	94.09	94.04	1.81
208	209	92.769	92.729	1.200	1.800	39.5	0.1	0.013	95.684	95.520	1.23	2.66	2.91	1.09	0.58	1.1	94.55	94.49	1.13	2.81	1.06	0.99	12.0	94.96	94.88	0.73	2.15	0.81	0.07	1.7	94.04	94.02	1.64
209	210	92.679	92.603	1.200	1.800	76.5	0.1	0.013	95.520	95.410	1.23	2.66	3.26	1.23	0.61	1.1	94.49	94.35	1.03	3.15	1.18	1.00	12.0	94.88	94.71	0.64	2.37	0.89	0.14	1.7	94.02	93.95	1.51
210	211	92.583	92.507	1.200	1.800	76.5	0.1	0.013	95.410	95.290	1.23	2.66	3.52	1.32	0.56	1.1	94.35	94.18	1.07	3.39	1.27	0.92	12.0	94.71	94.51	0.70	2.54	0.96	0.17	1.7	93.95	93.88	1.46
211	212	92.487	92.389	1.200	2.400	82.0	0.1	0.013	95.290	95.165	1.45	4.17	6.05	1.45	0.49	1.1	94.18	93.95	1.11	5.93	1.42	0.82	12.1	94.51	94.26	0.78	4.57	1.10	0.19	1.7	93.88	93.81	1.41
212	213	92.369	92.290	1.200	2.400	65.5	0.1	0.013	95.165	95.108	1.45	4.17	6.19	1.49	0.38	1.1	93.95	93.81	1.22	6.07	1.46	0.69	12.1	94.26	94.02	0.90	4.68	1.12	0.24	1.7	93.81	93.81	1.36
213	214	92.210	92.172	1.200	2.400	31.5	0.1	0.013	95.108	95.454	1.45	4.17	6.20	1.49	0.40	1.1	93.81	93.81	1.30	6.07	1.46	0.61	12.1	94.02	93.91	1.09	4.68	1.12	0.40	1.7	93.81	93.81	1.30
214	215	92.122	92.081	1.200	2.400	34.5	0.1	0.013	95.454	95.479	1.45	4.17	6.20	1.49	0.49	1.1	93.81	93.81	1.64	6.07	1.46	0.59	12.1	93.91	93.85	1.54	4.68	1.12	0.49	1.7	93.81	93.81	1.65
215	Pond1	92.001	91.950	1.200	2.400	42.5	0.1	0.013	95.479	94.500	1.45	4.17	6.25	1.50	0.61	1.1	93.81	93.81	1.67	6.07	1.46	0.65	12.1	93.85	93.85	1.63	4.67	1.12	0.61	1.7	93.81	93.81	1.67
251	252	93.818	93.748	0.675	N/A	70.5	0.1	0.013	96.160	96.054	0.74	0.27	0.22	0.83	0.61	1.1	95.10	95.08	1.06	0.20	0.75	0.82	12.0	95.31	95.28	0.85	0.16	0.60	-0.21	1.6	94.29	94.26	1.88
252	253	93.673	93.603	0.750	N/A	70.5	0.1	0.013	96.054	95.949	0.80	0.35	0.32	0.91	0.65	1.1	95.08	94.95	0.98	0.30	0.85	0.86	12.1	95.28	95.30	0.77	0.24	0.68	-0.17	1.6	94.26	94.23	1.80
253	254	93.303	93.227	1.050	N/A	76.5	0.1	0.013	95.949	95.834	1.00	0.86	0.96	1.11	0.60	1.1	94.95	94.76	1.00	0.93	1.08	0.94	12.1	95.30	95.17	0.65	0.75	0.87	-0.12	1.6	94.23	94.14	1.72
254	206	93.077	92.989	1.200	1.800	88.0	0.1	0.013	95.834	95.700	1.23	2.66	1.83	0.69	0.48	1.1	94.76	94.71	1.08	1.76	0.66	0.90	12.1	95.17	95.14	0.66	1.42	0.53	-0.14	1.6	94.14	94.12	1.70
261	262	93.851	93.753	0.675	N/A	97.5	0.1	0.013	96.161	96.015	0.74	0.27	0.26	0.98	0.66	1.1	95.19	95.08	0.97	0.25	0.94	0.83	12.1	95.36	95.27	0.81	0.20	0.75	-0.10	1.6	94.43	94.38	1.74
262	263	93.603	93.484	0.825	N/A	119.0	0.1	0.013	96.015	95.837	0.85	0.45	0.49	1.08	0.65	1.1	95.08	94.92	0.93	0.49	1.08	0.84	12.1	95.27	95.14	0.74	0.37	0.82	-0.05	1.6	94.38	94.31	1.63
263	264	93.409	93.302	0.900	N/A	107.0	0.1	0.013	95.837	95.676	0.90	0.57	0.65	1.14	0.61	1.1	94.92	94.71	0.92	0.65	1.14	0.83	12.1	95.14	94.94	0.70	0.50	0.87	0.00	1.7	94.31	94.20	1.53
264	265	93.222	93.182	0.900	N/A	40.5	0.1	0.013	95.676	95.616	0.90	0.57	0.78	1.36	0.59	1.1	94.71	94.64	0.96	0.78	1.36	0.82	12.1	94.94	94.90	0.73	0.60	1.05	0.08	1.8	94.20	94.16	1.48
265	266	93.032	93.002	1.050	N/A	30.0	0.1	0.013	95.616	95.571	1.00	0.86	1.19	1.38	0.56	1.1	94.64	94.58	0.98	1.18	1.37	0.82	12.1	94.90	94.86	0.72	0.93	1.08	0.08	1.7	94.16	94.12	1.46
266	267	92.852	92.753	1.200	N/A	99.0	0.1	0.013	95.571	95.421	1.09	1.23	1.81	1.47	0.53	1.1	94.58	94.35	0.99	1.77	1.44	0.80	12.1	94.86	94.66	0.71	1.41	1.14	0.07	1.6	94.12	93.99	1.45
267	268	92.703	92.657	1.200	1.800	45.5	0.1	0.013	95.421	95.342	1.23	2.66	2.47	0.93	0.45	1.1	94.35	94.30	1.07	2.40	0.90	0.76	12.1	94.66	94.62	0.76	1.93	0.73	0.08	1.7	93.99	93.95	1.44
268	211	92.607	92.567	1.200	1.800	40.5	0.1	0.013	95.342	95.290	1.23	2.66	2.48	0.93	0.49	1.1	94.30	94.18	1.04	2.41	0.91	0.81	12.1	94.62	94.51	0.73	1.93	0.73	0.14	1.7	93.95	93.88	1.39
301	302	93.248	93.194	0.600	N/A	54.0	0.1	0.013	95.797	95.829	0.69	0.19	0.16	0.82	-0.03	1.1	93.82	93.82	1.97	0.15	0.77	0.12	12.1	93.97	93.94	1.83	0.13	0.67	-0.03	1.7	93.82	93.81	1.98
302	303	93.164	93.153	0.600	N/A	11.0	0.1	0.013	95.829	95.808	0.69	0.19	0.16	0.82	0.06	1.1	93.82	93.82	2.01	0.14	0.72	0.18	12.1	93.94	93.94	1.88	0.13	0.67	0.05	1.6	93.81	93.81	2.02
303	304	93.123	93.055	0.600	N/A	67.5	0.1	0.013	95.808	95.729	0.69	0.19	0.16	0.82	0.10	1.1	93.82	93.82	1.99	0.14	0.72	0.21	12.2	93.94	93.91	1.87	0.13	0.67	0.09	1.6	93.81	93.81	2.00
304	305	92.980	92.906	0.675	N/A	73.5	0.1	0.013	95.729	95.410	0.74	0.27	0.27	1.02	0.16	1.1	93.82	93.82	1.91	0.24	0.90	0.25	12.1	93.91	93.86	1.82	0.22	0.83	0.16	1.6	93.81	93.81	1.92
305	306	92.681	92.605	0.900	N/A	76.5	0.1	0.013	95.410	95.300	0.90	0.57	0.55	0.96	0.24	1.1	93.82	93.82	1.59	0.51	0.89	0.27	12.1	93.86	93.85	1.55	0.47	0.82	0.23	1.6	93.81	93.81	1.60
306	307	92.545	92.465	0.900	1.800	79.5	0.1	0.013	95.300	94.860	1.09	1.77	0.88	0.50	0.37	1.1	93.82	93.81	1.49	0.82	0.46	0.41	12.1	93.85	93.85	1.45	0.71	0.40	0.37	1.6	93.81	93.81	1.49
307	308	92.445	92.368	0.900	1.800	77.0	0.1	0.013	94.860	94.823	1.09	1.77	1.30	0.74	0.47	1.1	93.81	93.81	1.05	1.25	0.71	0.51	12.0	93.85	93.85	1.01	1.04	0.59	0.47	1.5	93.81	93.81	1.05
308	407	92.338	92.244	0.900	1.800	94.5	0.1	0.013	94.823	95.199	1.09	1.77	1.25	0.71	0.58	1.1	93.81	93.81	1.01	1.25	0.71	0.61	12.0	93.85	93.85	0.97	1.00	0.57	0.57	1.7	93.81	93.81	1.01
401	402	92.920	92.891	0.675	N/A	26.5	0.1	0.013	95.844	95.746	0.78	0.28	0.27	0.97	0.22	1.0	93.81	93.81	2.03	0.27	0.97	0.44	12.0	94.03	94.02	1.81	0.22	0.79	0.22	1.7	93.81	93.81	2.03
402	403	92.816	92.805	0.750	N/A	11.0	0.1	0.013	95.746	95.689	0.80	0.35	0.27	0.77	0.25	1.1	93.81	93.81	1.93	0.27	0.77	0.45	12.0	94.02	94.01	1.73	0.22	0.62	0.24	1.7	93.81	93.81	1.94
403	404	92.775	92.695	0.750	N/A	80.0	0.1	0.013	95.689	95.577	0.80	0.35	0.27	0.77	0.29	1.1	93.81	93.81	1.88	0.27	0.77	0.48	12.0	94.01	93.96	1.68	0.21	0.60	0.28	1.7	93.81	93.81	1.88
404	405	92.620	92.513	0.825	N/A	106.5	0.1	0.013	95.577	95.150	0.85	0.45	0.47	1.04	0.37	1.1	93.81	93.81	1.76	0.47	1.04	0.51	12.0	93.96	93.88	1.62	0.37	0.82	0.36	1.7	93.81	93.81	1.77
405	406	92.438	92.350	0.900	N/A	68.0	0.1	0.013	95.150	95.005	1.03	0.65	0.79	1.21	0.47	1.1	93.81	93.81	1.34	0.79	1.21	0.54	12.0	93.88	93.85	1.27	0.63	0.97	0.47	1.7	93.81	93.81	1.34
406	407	92.330	92.244	0.900	N/A	66.5	0.1	0.013	95.005	95.199	1.03	0.65	0.78	1.20	0.58	1.1	93.81	93.81	1.19	0.79	1.21	0.62	12.0	93.85	93.85	1.16	0.62	0.95	0.58	1.7	93.81	93.81	1.20
407	408	92.214	92.196	0.900	1.800	17.5	0.1	0.013	95.199	94.558	1.09	1.77	1.97	1.12	0.70	1.1	93.81	93.81	1.39	2.04	1.16	0.73	12.0	93.85	93.85	1.35	1.62	0.92	0.69	1.7	93.81	93.81	1.39
408	Pond1	92.166	92.150	0.900	1.800	16.5	0.1	0.013	94.558	94.500	1.09	1.77	1.96	1.11	0.75	1.1	93.81	93.81	0.75	2.04	1.16	0.78	12.0	93.85	93.85	0.71	1.62	0.92	0.74	1.7	93.81	93.81	0.75
500	501	93.525	93.444	0.750	N/A	81.0	0.1	0.013	96.026	95.903	0.80	0.35	0.27	0.77	-0.07	1.0	94.21	94.18	1.82	0.26	0.74	0.44	12.1	94.71	94.67	1.31	0.22	0.62	-0.21	1.6	94.06	94.03	1.96
501	502	93.424	93.352	0.750	N/A	71.5	0.1	0.013	95.903	95.772	0.80	0.35	0.25	0.71	0.00	1.2	94.18	94.14	1.73	0.26	0.74	0.49	12.1	94.67	94.62	1.24	0.22	0.62	-0.14	1.7	94.03	94.00	1.87
502	503	93.277	93.266	0.825	N/A	11.0	0.1	0.013	95.772	95.653	0.85	0.45	0.36	0.79	0.03	1.2	94.14	94.12	1.64	0.38	0.84	0.51	12.1	94.62	94.60	1.16	0.31	0.68	-0.11	1.6	94.00	93.99	1.78
503	504	93.236	93.175	0.825	N/A	61.0	0.1	0.013	95.653	95.567	0.85	0.45	0.36	0.79	0.06	1.2	94.12	94.08	1.53	0.38	0.84	0.54	12.1	94.60	94.54	1.06	0.32	0.70	-0.08	1.7	93.99	93.95	1.67
504	505	93.145	93.134	0.825	N/A	11.0	0.1	0.013	95.567	95.525	0.85	0.45	0.36	0.79	0.11	1.2	94.08	94.06	1.49	0.38	0.84	0.57	12.1	94.54	94.52	1.02	0.33	0.73	-0.02	1.7	93.95	93.94	1.61

Table 6: Pipe Data and Hydraulic Simulation Results (1)

U/S MH	D/S MH	U/S Invert (m)	D/S Invert (m)	Pipe Diameter / Height (m)	Pipe Width (m)	Pipe Length (m)	Pipe Slope (%)	n	U/S MH Cover Elev. (m)	D/S MH Cover Elev. (m)	Design Velocity (m/s)	Design Flow (m^3/s)	100-year, 3-hour Chicago Storm + 20%							100-year, 12-hour SCS Type II Storm + 20%							July 1st, 1979 Historical Event						
													Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S (2) (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S (2) (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S (2) (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)
505	506	93.104	93.068	0.825	N/A	36.0	0.1	0.013	95.525	95.404	0.85	0.45	0.37	0.82	0.13	1.3	94.06	94.01	1.46	0.38	0.84	0.59	12.1	94.52	94.46	1.00	0.33	0.73	0.01	1.7	93.94	93.91	1.58
506	507	92.993	92.917	0.900	N/A	76.0	0.1	0.013	95.404	95.290	0.90	0.57	0.56	0.98	0.12	1.2	94.01	93.95	1.39	0.59	1.03	0.57	12.1	94.46	94.39	0.94	0.48	0.84	0.01	1.7	93.91	93.86	1.50
507	508	92.767	92.687	1.050	N/A	79.5	0.1	0.013	95.290	95.170	1.00	0.86	0.93	1.08	0.13	1.2	93.95	93.81	1.35	0.96	1.11	0.57	12.1	94.39	94.22	0.91	0.79	0.91	0.04	1.6	93.86	93.81	1.43
508	509	92.627	92.597	1.050	N/A	30.0	0.1	0.013	95.170	95.121	1.00	0.86	0.94	1.09	0.14	1.2	93.81	93.81	1.36	0.98	1.13	0.55	12.1	94.22	94.11	0.95	0.80	0.93	0.13	1.6	93.81	93.81	1.36
509	510	92.447	92.374	1.200	N/A	72.5	0.1	0.013	95.121	95.016	1.09	1.23	1.21	0.98	0.17	1.1	93.81	93.81	1.31	1.25	1.01	0.47	12.1	94.11	94.05	1.01	1.02	0.83	0.16	1.6	93.81	93.81	1.31
510	511	92.354	92.267	1.200	N/A	86.5	0.1	0.013	95.016	94.413	1.09	1.23	1.35	1.09	0.26	1.1	93.81	93.81	1.20	1.40	1.14	0.50	12.1	94.05	93.89	0.97	1.14	0.92	0.25	1.6	93.81	93.81	1.21
511	611	92.207	92.100	1.200	N/A	106.5	0.1	0.013	94.413	94.667	1.09	1.23	1.54	1.25	0.40	1.1	93.81	93.81	0.60	1.60	1.30	0.48	12.1	93.89	93.85	0.52	1.28	1.04	0.40	1.7	93.81	93.81	0.60
601	602	93.362	93.294	0.600	N/A	68.5	0.1	0.013	96.029	95.603	0.69	0.19	0.20	1.03	-0.14	1.1	93.83	93.82	2.20	0.18	0.93	0.25	12.1	94.21	94.15	1.82	0.16	0.82	-0.15	1.6	93.81	93.81	2.22
602	603	93.264	93.253	0.600	N/A	11.0	0.1	0.013	95.603	95.530	0.69	0.19	0.19	0.98	-0.04	1.1	93.82	93.82	1.78	0.19	0.98	0.28	12.1	94.15	94.13	1.46	0.16	0.82	-0.05	1.6	93.81	93.81	1.79
603	604	93.103	93.035	0.750	N/A	68.5	0.1	0.013	95.530	95.400	0.80	0.35	0.28	0.80	-0.03	1.1	93.82	93.82	1.71	0.27	0.77	0.28	12.1	94.13	94.09	1.40	0.24	0.68	-0.04	1.6	93.81	93.81	1.72
604	605	92.885	92.809	0.900	N/A	76.5	0.1	0.013	95.400	95.290	0.90	0.57	0.50	0.87	0.03	1.1	93.82	93.82	1.58	0.49	0.86	0.31	12.1	94.09	94.05	1.31	0.42	0.73	0.03	1.6	93.81	93.81	1.59
605	606	92.734	92.654	0.975	N/A	79.5	0.1	0.013	95.290	95.170	0.95	0.71	0.72	1.02	0.11	1.1	93.82	93.82	1.47	0.71	1.00	0.34	12.1	94.05	93.98	1.25	0.60	0.85	0.10	1.6	93.81	93.81	1.48
606	607	92.579	92.499	1.050	N/A	79.5	0.1	0.013	95.170	95.050	1.00	0.86	0.98	1.13	0.19	1.1	93.82	93.81	1.35	0.95	1.10	0.36	12.1	93.98	93.91	1.19	0.80	0.93	0.18	1.6	93.81	93.81	1.36
607	608	92.479	92.410	1.050	N/A	69.0	0.1	0.013	95.050	94.944	1.00	0.86	1.07	1.24	0.28	1.1	93.81	93.81	1.24	1.05	1.22	0.39	12.1	93.91	93.86	1.14	0.87	1.01	0.28	1.6	93.81	93.81	1.24
608	609	92.380	92.369	1.050	N/A	11.0	0.1	0.013	94.944	94.835	1.00	0.86	1.06	1.23	0.38	1.1	93.81	93.81	1.13	1.05	1.22	0.43	12.1	93.86	93.86	1.09	0.86	1.00	0.38	1.6	93.81	93.81	1.14
609	610	92.219	92.185	1.200	N/A	34.5	0.1	0.013	94.835	94.875	1.09	1.23	1.22	0.99	0.39	1.1	93.81	93.81	1.02	1.20	0.97	0.44	12.1	93.86	93.85	0.98	0.99	0.80	0.39	1.6	93.81	93.81	1.03
610	611	92.125	92.100	1.200	N/A	25.0	0.1	0.013	94.875	94.667	1.09	1.23	1.47	1.19	0.49	1.1	93.81	93.81	1.06	1.44	1.17	0.52	12.1	93.85	93.85	1.03	1.20	0.97	0.48	1.7	93.81	93.81	1.07
611	612	92.070	92.048	1.200	1.800	22.0	0.1	0.013	94.667	94.113	1.23	2.66	2.98	1.12	0.54	1.1	93.81	93.81	0.86	2.99	1.12	0.58	12.1	93.85	93.85	0.82	2.47	0.93	0.54	1.7	93.81	93.81	0.86
612	Pond1	92.018	92.000	1.200	1.800	18.0	0.1	0.013	94.113	94.500	1.23	2.66	2.96	1.11	0.59	1.1	93.81	93.81	0.30	2.99	1.12	0.63	12.1	93.85	93.85	0.27	2.46	0.93	0.59	1.7	93.81	93.81	0.30
700	701	93.517	93.443	0.975	N/A	74.0	0.1	0.013	96.786	96.740	0.95	0.71	0.80	1.13	1.61	1.0	96.10	95.68	0.68	0.66	0.93	0.88	12.0	95.38	95.26	1.41	0.62	0.87	0.56	1.8	95.05	94.91	1.74
701	702	93.368	93.328	1.050	N/A	40.0	0.1	0.013	96.740	96.530	1.00	0.86	1.02	1.18	1.27	1.0	95.68	95.59	1.06	0.81	0.94	0.85	12.0	95.26	95.23	1.48	0.76	0.88	0.49	1.6	94.91	94.85	1.83
702	703	93.178	93.106	1.200	1.800	71.5	0.1	0.013	96.530	96.420	1.23	2.66	1.96	0.74	1.21	1.0	95.59	95.45	0.94	1.69	0.64	0.85	12.0	95.23	95.19	1.31	1.53	0.58	0.47	1.6	94.85	94.80	1.68
703	704	93.086	92.980	1.200	1.800	105.5	0.1	0.013	96.420	96.260	1.23	2.66	2.53	0.95	1.16	1.0	95.45	95.27	0.97	2.20	0.83	0.91	12.0	95.19	95.11	1.23	2.03	0.76	0.52	1.6	94.80	94.71	1.62
704	705	92.960	92.888	1.200	1.800	71.5	0.1	0.013	96.260	96.150	1.23	2.66	3.38	1.27	1.11	1.1	95.27	95.19	0.99	3.04	1.14	0.95	12.0	95.11	95.03	1.15	2.81	1.06	0.55	1.6	94.71	94.63	1.55
705	706	92.868	92.801	1.200	2.400	67.0	0.1	0.013	96.150	95.902	1.32	3.80	4.83	1.27	1.12	1.1	95.19	95.05	0.96	4.61	1.21	0.96	12.0	95.03	94.94	1.12	4.13	1.09	0.56	1.6	94.63	94.55	1.52
706	707	92.781	92.697	1.200	2.400	83.5	0.1	0.013	95.902	95.430	1.32	3.80	4.97	1.31	1.07	1.1	95.05	94.67	0.85	4.73	1.24	0.95	12.0	94.94	94.67	0.97	4.23	1.11	0.56	1.6	94.55	94.30	1.36
707	708	92.637	92.567	1.200	2.400	70.0	0.1	0.013	95.430	95.320	1.32	3.80	5.61	1.48	0.83	1.1	94.67	94.48	0.76	5.32	1.40	0.84	12.0	94.67	94.55	0.76	4.75	1.25	0.47	1.6	94.30	94.18	1.13
708	709	92.547	92.481	1.200	2.400	66.0	0.1	0.013	95.320	95.221	1.32	3.80	5.88	1.55	0.73	1.1	94.48	94.31	0.84	5.52	1.45	0.80	12.0	94.55	94.43	0.77	4.94	1.30	0.43	1.6	94.18	94.05	1.14
709	710	92.461	92.391	1.200	2.400	70.5	0.1	0.013	95.221	95.112	1.32	3.80	5.92	1.56	0.64	1.1	94.31	93.87	0.92	5.52	1.45	0.77	12.0	94.43	94.09	0.80	4.94	1.30	0.39	1.6	94.05	93.81	1.17
710	711	92.331	92.200	1.200	2.400	131.0	0.1	0.013	95.112	94.385	1.32	3.80	6.20	1.63	0.34	1.1	93.87	93.81	1.24	6.03	1.59	0.56	12.0	94.09	93.85	1.02	5.34	1.40	0.28	1.6	93.81	93.81	1.30
711	712	92.140	92.118	1.200	2.400	21.5	0.1	0.013	94.385	94.000	1.32	3.80	6.12	1.61	0.47	1.1	93.81	93.81	0.57	6.03	1.59	0.51	12.0	93.85	93.85	0.53	5.36	1.41	0.47	1.6	93.81	93.81	0.58
712	Pond1	92.058	92.050	1.200	2.400	8.5	0.1	0.013	94.000	94.500	1.32	3.80	6.12	1.61	0.56	1.1	93.81	93.81	0.19	6.01	1.58	0.59	12.0	93.85	93.85	0.15	5.35	1.41	0.55	1.6	93.81	93.81	0.19
751	752	93.846	93.752	0.600	N/A	94.0	0.1	0.013	96.720	96.579	0.69	0.19	0.23	1.18	1.54	1.1	95.99	95.64	0.73	0.15	0.77	1.63	12.0	96.07	95.67	0.65	0.16	0.82	0.71	1.6	95.15	95.02	1.57
752	753	93.602	93.520	0.750	N/A	82.0	0.1	0.013	96.579	96.452	0.80	0.35	0.35	0.99	1.28	1.1	95.64	95.52	0.94	0.26	0.74	1.32	12.0	95.67	95.46	0.91	0.26	0.74	0.66	1.6	95.02	94.89	1.56
753	754	93.295	93.209	0.975	N/A	86.0	0.1	0.013	96.452	96.324	0.95	0.71	0.91	1.28	1.25	1.1	95.52	95.36	0.93	0.79	1.11	1.19	12.0	95.46	95.25	1.00	0.73	1.03	0.62	1.6	94.89	94.79	1.56
754	705	93.134	93.018	1.050	N/A	116.0	0.1	0.013	96.324	96.150	1.00	0.86	1.09	1.26	1.18	1.1	95.36	95.19	0.96	0.97	1.12	1.07	12.0	95.25	95.03	1.07	0.88	1.02	0.61	1.6	94.79	94.63	1.53
806	807	93.069	93.047	0.600	N/A	21.5	0.1	0.013	95.186	95.180	0.69	0.19	0.12	0.62	0.63	1.1	94.30	94.16	0.89	0.10	0.52	0.59	12.0	94.26	94.18	0.92	0.06	0.31	0.25	1.6	93.92	93.87	1.27
807	808	92.897	92.861	0.750	N/A	35.5	0.1	0.013	95.180	95.133	0.80	0.35	0.41	1.16	0.51	1.1	94.16	94.01	1.03	0.37	1.05	0.54	12.0	94.18	94.16	1.00	0.29	0.82	0.23	1.6	93.87	93.81	1.31
808	809	92.786	92.775	0.825	N/A	11.0	0.1	0.013	95.133	95.095	0.85	0.45	0.56	1.23	0.40	1.1	94.01	94.00	1.13	0.50	1.10	0.54	12.0	94.16	94.14	0.98	0.42	0.93	0.20	1.6	93.81	93.81	1.32
809	710	92.725	92.706	0.825	N/A	18.5	0.1	0.013	95.095	95.112	0.85	0.45	0.57	1.26	0.45	1.1	94.00	93.87	1.10	0.51	1.12	0.59	12.0	94.14	94.09	0.96	0.42	0.93	0.26	1.6	93.81	93.81	1.29
901	902	94.501	94.436	0.600	N/A	65.0	0.1	0.013	96.268	96.259	0.69	0.19	0.14	0.72	-0.10	1.1	95.00	94.97	1.27	0.14	0.72	-0.13	12.0	94.98	94.95	1.29	0.12	0.62	-0.19	1.7	94.92	94.88	1.35
902	903	94.416	94.346	0.675	N/A	70.0	0.1	0.013	96.259	96.151	0.74	0.27	0.24	0.90	-0.12	1.1	94.97	94.91	1.29	0.23	0.87	-0.15	12.0	94.95	94.89	1.31	0.20	0.75	-0.21	1.7	94.88	94.83	1.38
903	904	94.271	94.201	0.750	N/A	69.5	0.1	0.013	96.151	96.259	0.80	0.35	0.32	0.91	-0.11	1.1	94.91	94.80	1.24	0.31	0.88	-0.13	12.1	94.89	94.78	1.26	0.27	0.77	-0.20	1.7	94.83	94.73	1.33
904	905	94.121	94.005	0.900	N/A	116.0	0.1	0.013	96.259	96.162	0.90	0.57	0.64	1.12	-0.22	1.1	94.80	94.56	1.46	0.61	1.07	-0.24	12.1	94.78	94.54	1.48	0.53	0.93	-0.29	1.7	94.73	94.50	1.53
905	1007	93.955	93.939	0.900	N/A	16.0	0.1	0.013	96.162	96.196	0.90	0.57	0.64	1.12	-0.30	1.1	94.56	94.38	1.61	0.61	1.07	-0.31	12.1	94.54	94.39	1.62	0.53	0.93	-0.36	1.7	94.50	94.28	1.67
1001	1002	94.618	94.545	0.525	N/A	72.5	0.1	0.013	96.638	96.632	0.63	0.14	0.12	0.88	-0.09	1.1	95.06	94.97	1.58	0.12	0.88	-0.11	12.0	95.03	94.94	1.61	0.10	0.74	-0.16	1.7	94.98	94.88	1.66
1002	1003	94.465	94.388	0.600	N/A	77.0	0.1	0.013	96.632	96.593	0.69	0.19	0.19	0.98	-0.10	1.1	94.97	94.90	1.66	0.18	0.93	-0.13	12.1	94.94	94.87	1.69	0.16	0.82	-0.19	1.7	94.88	94.79	1.76
1003	1004	94.238	94.166	0.750	N/A	71.5	0.1	0.013	96.593	96.490	0.80	0.35	0.38	1.08	-0.09	1.1	94.90	94.82	1.70	0.37	1.05	-0.12	12.1	94.87	94.79	1.73	0.33	0.94	-0.20	1.7	94.79	94.71	1.80
1004	1005	94.016	93.942	0.900	N/A	74.0	0.1	0.013	96.490	96.380	0.90	0.57	0.57	1.00	-0.10	1.1	94.82	94.68	1.67	0.55	0.96	-0.13	12.1	94.79	94.65	1.70	0.49	0.86	-0.20	1.7	94.71	94.58	1.78
1005	1006	93.862	93.791	0.975	N/A	71.0	0.1	0.013	96.380	96.270	0.95	0.71	0.70	0.99	-0.16	1.1	94.68	94.51	1.70	0.67	0.95	-0.18	12.1	94.65	94.49	1.73	0.60	0.85	-0.26	1.7	94.58	94.41	1.80
1006	1007	93.566	93.482	1.200	N/A	83.5	0.1	0.013	96.270	96.196	1.09	1.23	1.40	1.14	-0.25	1.1	94.51	94.38	1.76	1.34	1.09	-0.28	12.1	94.49	94.39	1.78	1.18	0.96	-0.36	1.7	94.41	94.28	1.86
1007	1008	93.361	93.303	1.200	N/A	29.0	0.2	0.013	96.196	96.057	1.54	1.74	2.07	1.19	-0.18	1.1	94.38	94.21	1.81	1.99	1.14	-0.17	12.1	94.39	94.39	1.80	1.74	1.00	-0.28	1.7	94.28	94.23	1.91
1008	Pond2	93.223	93.200	1.200	2.400	23.0	0.1	0.013	96.057	96.000	1.32	3.80	3.73	0.98	-0.21	1.1	94.21	94.20	1.85	3.51	0.92	-0.03	12.1	94.39	94.38	1.67	2.24	0.59	-0.20	1.7	94.23	94.22	1.83
1101	1102	95.258	95.139	0.750	N/A	119.0	0.1	0.013	97.798	97.953	0.80	0.35	0.28	0.80	-0.24	1.1	95.77	95.63	2.03	0.27	0.77	-0.26	12.0	95.75	95.61	2.05	0.23	0.65	-0.31	1.7	95.70	95.55	2.10
1102	1103	95.109	95.101	0.750	N/A	8.0	0.1	0.013	97.953	97.864	0.80	0.35	0.28	0.80	-0.23	1.1	95.63	95.60	2.33	0.27	0.77	-0.25	12.1	95.61	95.58	2.35	0.24	0.68	-0.31	1.7	95.55	95.52	2.41
1103	1104	94.951	94.885	0.900	N/A	65.5	0.1	0.013	97.864	98.100	0.90	0.57	0.49	0.86	-0.25	1.1	95.60	95.54	2.27	0.47	0.82	-0.27	12.1	95.58	95.52	2.28	0.40	0.70	-0.33	1.7	95.52	95.46	2.35

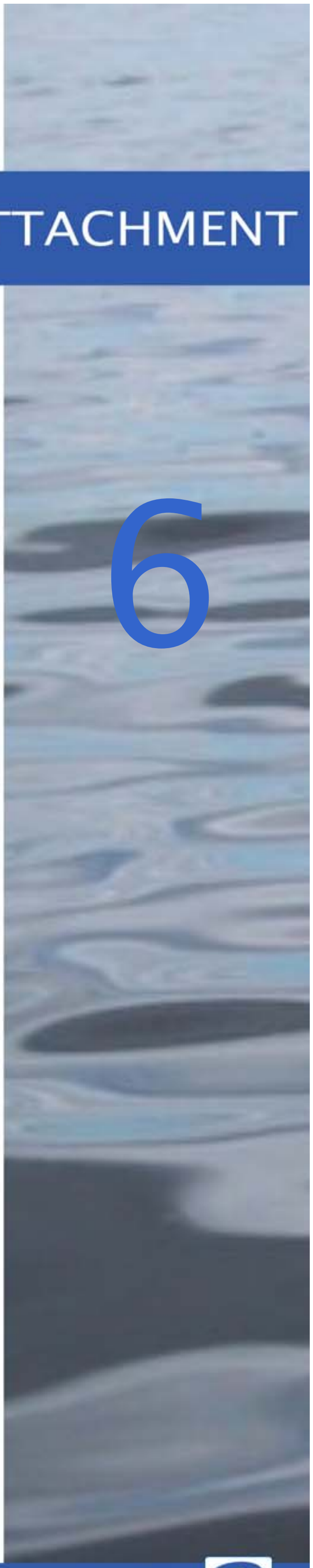
Table 6: Pipe Data and Hydraulic Simulation Results [1]

U/S MH	D/S MH	U/S Invert (m)	D/S Invert (m)	Pipe Diameter / Height (m)	Pipe Width (m)	Pipe Length (m)	Pipe Slope (%)	n	U/S MH Cover Elev. (m)	D/S MH Cover Elev. (m)	Design Velocity (m/s)	Design Flow (m^3/s)	100-year, 3-hour Chicago Storm + 20%							100-year, 12-hour SCS Type II Storm + 20%							July 1st, 1979 Historical Event						
													Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)	Peak Pipe Flow (m^3/s)	Peak / Design Flow	Surcharge U/S [2] (m)	Time to Peak (h)	Max. U/S HGL (m)	Max. D/S HGL (m)	Freeboard U/S HGL and MH Cover (m)
1104	1105	94.825	94.753	0.900	N/A	71.5	0.1	0.013	98.100	98.730	0.90	0.57	0.58	1.01	-0.19	1.1	95.54	95.33	2.56	0.56	0.98	-0.21	12.1	95.52	95.31	2.58	0.49	0.86	-0.27	1.7	95.46	95.25	2.65
1105	1106	94.673	94.619	0.900	N/A	54.5	0.1	0.013	98.730	98.710	0.90	0.57	0.58	1.01	-0.25	1.1	95.33	95.20	3.41	0.56	0.98	-0.27	12.1	95.31	95.18	3.43	0.49	0.86	-0.33	1.7	95.25	95.11	3.49
1106	1107	94.469	94.462	1.050	N/A	7.0	0.1	0.013	98.710	98.708	1.00	0.86	1.04	1.20	-0.32	1.1	95.20	95.10	3.51	1.00	1.16	-0.34	12.1	95.18	95.07	3.53	0.87	1.01	-0.41	1.7	95.11	95.01	3.60
1107	1108	94.432	93.978	1.050	N/A	113.5	0.4	0.013	98.708	96.666	1.99	1.73	1.04	0.60	-0.38	1.1	95.10	94.89	3.61	1.00	0.58	-0.41	12.1	95.07	94.85	3.63	0.87	0.50	-0.48	1.7	95.01	94.76	3.70
1108	1109	93.528	93.381	1.500	N/A	73.5	0.2	0.013	96.666	96.511	1.79	3.16	4.75	1.50	-0.14	1.2	94.89	94.31	1.78	4.56	1.44	-0.18	12.1	94.85	94.38	1.82	4.09	1.29	-0.27	1.7	94.76	94.25	1.91
1109	1110	93.351	93.283	1.500	2.400	67.5	0.1	0.013	96.511	95.479	1.45	5.23	4.75	0.91	-0.55	1.2	94.31	94.20	2.21	4.54	0.87	-0.47	12.1	94.38	94.38	2.13	4.08	0.78	-0.61	1.7	94.25	94.22	2.27
1110	Pond2	93.223	93.200	1.500	2.400	22.5	0.1	0.013	95.479	96.000	1.45	5.23	4.74	0.91	-0.52	1.2	94.20	94.20	1.28	4.50	0.86	-0.34	12.1	94.38	94.38	1.10	4.05	0.77	-0.50	1.7	94.22	94.22	1.26
1201	1202	95.554	95.502	0.825	N/A	51.5	0.1	0.013	98.081	98.047	0.85	0.45	0.39	0.86	0.03	1.0	96.41	96.37	1.67	0.37	0.82	-0.11	12.0	96.27	96.23	1.81	0.32	0.70	-0.31	1.7	96.07	96.00	2.01
1202	1203	95.472	95.461	0.825	N/A	11.0	0.1	0.013	98.047	98.014	0.85	0.45	0.39	0.86	0.07	1.0	96.37	96.36	1.68	0.36	0.79	-0.07	12.0	96.23	96.22	1.82	0.33	0.73	-0.30	1.7	96.00	95.97	2.05
1203	1204	95.311	95.243	0.975	N/A	67.5	0.1	0.013	98.014	97.847	0.95	0.71	0.57	0.80	0.07	1.0	96.36	96.32	1.66	0.54	0.76	-0.07	12.0	96.22	96.18	1.80	0.48	0.68	-0.32	1.7	95.97	95.92	2.05
1204	1205	95.223	95.149	0.975	N/A	67.5	0.1	0.013	97.847	97.741	1.00	0.74	0.56	0.75	0.12	1.2	96.32	96.27	1.53	0.53	0.71	-0.01	12.2	96.18	96.14	1.66	0.48	0.65	-0.28	1.7	95.92	95.86	1.93
1205	1206	95.119	95.108	0.975	N/A	11.0	0.1	0.013	97.741	97.718	0.95	0.71	0.69	0.97	0.17	1.2	96.27	96.24	1.47	0.65	0.92	0.04	12.2	96.14	96.11	1.61	0.58	0.82	-0.23	1.7	95.86	95.84	1.88
1206	1207	95.078	95.022	0.975	N/A	56.0	0.1	0.013	97.718	97.549	0.95	0.71	0.69	0.97	0.19	1.2	96.24	96.18	1.48	0.66	0.93	0.06	12.2	96.11	96.05	1.61	0.58	0.82	-0.22	1.7	95.84	95.78	1.88
1207	1208	94.992	94.981	0.975	N/A	11.0	0.1	0.013	97.549	97.613	0.95	0.71	0.70	0.99	0.21	1.2	96.18	96.15	1.37	0.67	0.95	0.09	12.3	96.05	96.03	1.50	0.58	0.82	-0.19	1.7	95.78	95.75	1.77
1208	1209	94.951	94.915	0.975	N/A	36.0	0.1	0.013	97.613	97.564	0.95	0.71	0.70	0.99	0.23	1.2	96.15	96.11	1.46	0.69	0.97	0.10	12.3	96.03	95.99	1.58	0.59	0.83	-0.18	1.7	95.75	95.71	1.86
1209	1210	94.840	94.832	1.050	N/A	7.5	0.1	0.013	97.564	97.551	1.00	0.86	0.84	0.97	0.22	1.2	96.11	96.08	1.46	0.81	0.94	0.10	12.3	95.99	95.97	1.58	0.69	0.80	-0.18	1.7	95.71	95.69	1.86
1210	1211	94.802	94.773	1.050	N/A	29.0	0.1	0.013	97.551	96.869	1.00	0.86	0.84	0.97	0.23	1.2	96.08	96.04	1.47	0.83	0.96	0.11	12.3	95.97	95.93	1.59	0.69	0.80	-0.17	1.7	95.69	95.65	1.87
1211	1212	94.743	94.735	1.050	N/A	7.5	0.1	0.013	96.869	96.861	1.00	0.86	0.84	0.97	0.25	1.2	96.04	96.02	0.83	0.84	0.97	0.14	12.3	95.93	95.91	0.94	0.70	0.81	-0.14	1.8	95.65	95.63	1.22
1212	1213	94.705	94.641	1.050	N/A	64.0	0.1	0.013	96.861	97.400	1.00	0.86	0.85	0.98	0.26	1.4	96.02	95.96	0.84	0.86	1.00	0.15	12.3	95.91	95.86	0.95	0.72	0.83	-0.13	1.8	95.63	95.59	1.23
1213	1214	94.491	94.419	1.200	N/A	71.5	0.1	0.013	97.400	97.290	1.09	1.23	1.20	0.97	0.27	1.2	95.96	95.89	1.44	1.14	0.92	0.17	12.2	95.86	95.80	1.54	1.02	0.83	-0.10	1.7	95.59	95.55	1.81
1214	1215	94.399	94.327	1.200	N/A	71.5	0.1	0.013	97.290	97.190	1.09	1.23	1.40	1.14	0.30	1.2	95.89	95.71	1.40	1.34	1.09	0.20	12.2	95.80	95.64	1.49	1.19	0.97	-0.05	1.7	95.55	95.41	1.74
1215	1216	94.177	94.105	1.350	N/A	71.5	0.1	0.013	97.190	97.080	1.18	1.69	1.85	1.10	0.19	1.2	95.71	95.63	1.48	1.78	1.05	0.11	12.1	95.64	95.56	1.55	1.59	0.94	-0.12	1.7	95.41	95.35	1.78
1216	1217	93.955	93.871	1.500	N/A	84.5	0.1	0.013	97.080	96.955	1.26	2.24	2.51	1.12	0.18	1.2	95.63	95.53	1.45	2.41	1.08	0.11	12.1	95.56	95.47	1.52	2.15	0.96	-0.10	1.7	95.35	95.29	1.73
1217	1218	93.851	93.768	1.500	N/A	83.0	0.1	0.013	96.955	96.830	1.26	2.24	2.79	1.25	0.18	1.2	95.53	95.41	1.42	2.68	1.20	0.12	12.1	95.47	95.35	1.49	2.41	1.08	-0.07	1.7	95.29	95.20	1.67
1218	1219	93.748	93.702	1.500	N/A	46.5	0.1	0.013	96.830	96.760	1.26	2.24	2.87	1.28	0.16	1.2	95.41	95.34	1.42	2.76	1.23	0.10	12.1	95.35	95.28	1.48	2.48	1.11	-0.05	1.7	95.20	95.15	1.63
1219	1108	93.682	93.608	1.500	N/A	74.0	0.1	0.013	96.760	96.666	1.26	2.24	3.41	1.53	0.16	1.2	95.34	94.89	1.42	3.28	1.47	0.10	12.1	95.28	94.85	1.48	2.95	1.32	-0.04	1.7	95.15	94.76	1.61
1600	1601	94.197	94.025	1.200	N/A	107.5	0.2	0.013	97.015	96.875	1.38	1.56	2.11	1.35	0.75	3.3	96.14	95.83	0.87	2.51	1.61	1.67	14.6	97.06	96.62	🟩 -0.05	2.12	1.36	0.78	3.5	96.18	95.86	0.84
1601	1602	94.025	93.873	1.200	N/A	95.0	0.2	0.013	96.875	96.627	1.38	1.56	2.11	1.35	0.60	3.3	95.83	95.55	1.05	2.51	1.61	1.40	14.6	96.62	96.23	🟩 0.25	2.12	1.36	0.63	3.5	95.86	95.58	1.02
1602	1603	93.873	93.721	1.200	N/A	95.0	0.2	0.013	96.627	96.665	1.38	1.56	2.11	1.35	0.48	3.3	95.55	95.27	1.08	2.51	1.61	1.15	14.6	96.23	95.83	🟩 0.40	2.12	1.36	0.50	3.5	95.58	95.29	1.05
1603	1604	93.721	93.569	1.200	N/A	95.0	0.2	0.013	96.665	96.891	1.38	1.56	2.11	1.35	0.35	3.3	95.27	95.00	1.40	2.51	1.61	0.91	14.6	95.83	95.44	0.83	2.12	1.36	0.37	3.5	95.29	95.01	1.37
1604	1605	93.569	93.431	1.200	N/A	86.5	0.2	0.013	96.891	96.700	1.38	1.56	2.11	1.35	0.23	3.3	95.00	94.52	1.89	2.51	1.61	0.67	14.6	95.44	94.77	1.46	2.12	1.36	0.24	3.5	95.01	94.53	1.88
1605	1008	93.431	93.316	1.200	N/A	72.0	0.2	0.013	96.700	96.057	1.38	1.56	2.10	1.35	-0.11	3.3	94.52	94.21	2.18	2.51	1.61	0.13	14.6	94.77	94.39	1.94	2.12	1.36	-0.10	3.5	94.53	94.23	2.17
7000	700	93.664	93.592	0.900	N/A	71.5	0.1	0.013	96.997	96.786	0.90	0.57	0.62	1.08	1.74	1.0	96.31	96.10	0.69	0.53	0.93	0.87	12.0	95.44	95.38	1.56	0.50	0.87	0.57	1.8	95.13	95.05	1.86
Pond1	Out1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	3.76	N/A	N/A	N/A	N/A	N/A	N/A	5.35	N/A	N/A	N/A	N/A	N/A	N/A	3.60	N/A	N/A	N/A	N/A	N/A	N/A
Pond2	Out2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	2.06	N/A	N/A	N/A	N/A	N/A	N/A	2.64	N/A	N/A	N/A	N/A	N/A	N/A	2.12	N/A	N/A	N/A	N/A	N/A	N/A
1500	1501	92.698	92.577	1.200	1.800	120.0	0.1	0.013	95.914	95.720	1.23	2.66	3.38	1.27	0.47	3.0	94.37	94.17	1.55	4.14	1.56	0.77	13.8	94.67	94.37	1.25	3.43	1.29	0.48	3.3	94.38	94.18	1.54
1501	21400	92.577	92.450	1.200	1.800	127.0	0.1	0.013	95.720	95.562	1.23	2.66	3.38	1.27	0.39	3.0	94.17	93.93	1.55	4.14	1.56	0.60	13.8	94.37	94.01	1.35	3.43	1.29	0.40	3.3	94.18	93.93	1.54
21400	Pond1	92.420	92.350	1.200	1.800	70.0	0.1	0.013	95.562	94.500	1.23	2.66	3.38	1.27	0.31	3.0	93.93	93.81	1.63	4.14	1.56	0.39	13.8	94.01	93.85	1.55	3.43	1.29	0.31	3.3	93.93	93.81	1.64

Note: [1] Based on restrictive downstream conditions; Summer = 93.68 m / Spring = 94.11 m for SWM Facility 1, and Spring = 94.18 m for Pond 2. All other results based on free outfall conditions.

[2] A negative surcharge implies that the pipe is not flowing full

🟩 0.21 Freeboard less than 0.5 m.



ATTACHMENT

6

100-YEAR FLOWS AND WATER LEVELS ON THE VAN GAAL DRAIN UNDER EXISTING AND PROPOSED CONDITIONS

JFSA Water Resources and Environmental Consultants

Table 7A: Existing and Proposed Conditions Flows on the Van Gaal Drain, Arbuckle Drain and Jock River (1)

River	Reach	River Station	Flow (m^3/s)						Difference in Flow (%)		
			Existing Conditions			Proposed Conditions					
			1	2	4	1	2	4	1	2	4
Jock River	Reach 1	22026	5.540	167.682	156.00	5.597	162.984	156.00	1.0	-2.8	0.0
Jock River	Reach 2	21359	5.773	74.628	161.76	5.850	72.528	161.76	1.3	-2.8	0.0
Jock River	Reach 3	18677	17.624	88.341	166.76	16.683	86.557	166.76	-5.3	-2.0	0.0
Jock River	Reach 4	16872	15.385	85.979	181.76	15.001	84.532	181.76	-2.5	-1.7	0.0
Jock River	Reach 5	16112	16.168	143.467	180.00	15.969	142.548	180.00	-1.2	-0.6	0.0
Jock River	Reach 6	11769	10.864	153.565	185.00	11.480	152.970	185.00	5.7	-0.4	0.0
Jock River	Reach 7	10144	8.741	152.939	196.00	9.599	152.261	196.00	9.8	-0.4	0.0
Jock River	Reach 7	6550	9.652	189.823	201.00	10.508	189.907	201.00	8.9	0.0	0.0
Jock River	Reach 7	3699	5.067	211.402	205.00	5.554	210.531	205.00	9.6	-0.4	0.0
Joys Road Trib	Reach 1	705	2.619	3.166	0.883	1.880	3.160	0.883	-28.2	-0.2	0.0
Moore Drain	Reach 2	555	0.9	0.683	0.033	0.064	0.102	0.001	-92.9	-85.1	-97.0
Moore Drain	Reach 1	298	2.172	2.132	0.322	1.452	1.699	0.143	-33.2	-20.3	-55.6
Moore Drain Trib	Reach 1	311 / 599	1.866	1.594	0.259	1.370	1.586	0.122	-26.6	-0.5	-52.8
Van Gaal Drain	Reach 3	3494	3.701	4.228	1.636	2.511	4.073	1.636	-32.1	-3.7	0.0
Van Gaal Drain	Reach 3	3322	4.021	4.588	1.651	2.837	4.488	1.651	-29.4	-2.2	0.0
Van Gaal Drain	Reach 3	3175	4.813	5.235	1.653	3.606	5.225	1.653	-25.1	-0.2	0.0
Van Gaal Drain	Reach 2	2554	7.272	8.316	2.857	5.263	8.182	2.857	-27.6	-1.6	0.0
Van Gaal Drain	Reach 2	2076	9.543	10.808	3.286	7.081	9.973	3.286	-25.8	-7.7	0.0
Van Gaal Drain	Reach 2	1340	11.434	11.619	3.426	7.121	10.024	3.424	-37.7	-13.7	-0.1
Van Gaal Drain	Reach 2	1312	12.2	12.204	3.439	7.121	10.024	3.424	-41.6	-17.9	-0.5
Van Gaal Drain	Reach 1	746	16.377	15.739	4.056	1.144	13.804	4.229	-93.0	-12.3	4.3
Van Gaal Drain	Reach 1	666	16.377	15.739	4.056	1.144	13.804	4.229	-93.0	-12.3	4.3
Van Gaal Drain	Reach 1	226	16.419	15.777	4.371	10.195	13.868	4.500	-37.9	-12.1	2.9

(1) Scenario Descriptions:

1. The Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River.
2. The Van Gaal Drain 100-year spring snowmelt plus rainfall peak flow reaches the Jock River.
4. The Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain.

Table 7B: Existing and Proposed Conditions Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Maximum Water Surface Elevation (m)								
			Existing Conditions			Proposed Conditions			Difference		
			1	2	4	1	2	4	1	2	4
Van Gaal Drain	Reach 3	3494	97.56	97.60	97.16	97.38	97.59	97.16	-0.18	-0.01	0.00
Van Gaal Drain	Reach 3	3322	97.43	97.47	97.06	97.27	97.47	97.06	-0.16	0.00	0.00
Van Gaal Drain	Reach 3	3312	97.42	97.45	97.05	97.26	97.45	97.05	-0.16	0.00	0.00
Van Gaal Drain	Reach 3	3311	Culvert								
Van Gaal Drain	Reach 3	3302	97.33	97.35	97.01	97.16	97.35	97.01	-0.17	0.00	0.00
Van Gaal Drain	Reach 3	3297	97.27	97.32	96.98	97.11	97.32	96.98	-0.16	0.00	0.00
Van Gaal Drain	Reach 3	3185	97.17	97.26	96.56	96.94	97.26	96.56	-0.23	0.00	0.00
Van Gaal Drain	Reach 3	3175	97.16	97.23	96.59	96.94	97.23	96.59	-0.22	0.00	0.00
Van Gaal Drain	Reach 3	3174	Culvert								
Van Gaal Drain	Reach 3	3165	96.75	96.74	96.55	96.72	96.74	96.55	-0.03	0.00	0.00
Van Gaal Drain	Reach 3	3149	96.72	96.72	96.51	96.67	96.72	96.51	-0.05	0.00	0.00
Van Gaal Drain	Reach 3	3086	96.65	96.63	96.40	96.59	96.63	96.40	-0.06	0.00	0.00
Van Gaal Drain	Reach 3	3016	96.61	96.59	96.33	96.55	96.59	96.32	-0.06	0.00	-0.01
Van Gaal Drain	Reach 3	2980	96.57	96.56	96.28	96.52	96.56	96.27	-0.05	0.00	-0.01
Van Gaal Drain	Reach 3	2851	96.41	96.42	96.03	96.29	96.41	95.99	-0.12	-0.01	-0.04
Van Gaal Drain	Reach 3	2808	96.38	96.39	96.01	96.26	96.39	95.95	-0.12	0.00	-0.06
Van Gaal Drain	Reach 3	2658	96.28	96.29	95.95	96.15	96.28	95.88	-0.13	-0.01	-0.07
Van Gaal Drain	Reach 2	2554	96.27	96.29	95.94	96.14	96.28	95.86	-0.13	-0.01	-0.08
Van Gaal Drain	Reach 2	2478	96.16	96.15	95.88	96.03	96.14	95.77	-0.13	-0.01	-0.11
Van Gaal Drain	Reach 2	2427.58*	N/A	N/A	N/A	95.95	96.04	95.71	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2377.17*	N/A	N/A	N/A	95.86	95.95	95.63	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2326.76*	N/A	N/A	N/A	95.77	95.87	95.53	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2276.35*	N/A	N/A	N/A	95.34	95.65	95.14	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2252	N/A	N/A	N/A	95.40	95.40	94.95	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2237	N/A	N/A	N/A	95.38	95.38	94.93	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2217	N/A	N/A	N/A	95.35	95.35	94.90	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2197	N/A	N/A	N/A	95.33	95.32	94.87	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2177	N/A	N/A	N/A	95.30	95.29	94.83	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2157	95.47	95.48	95.03	95.28	95.26	94.80	-0.19	-0.22	-0.23
Van Gaal Drain	Reach 2	2154	N/A	N/A	N/A	95.25	95.23	94.77	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2153	N/A	N/A	N/A	95.21	95.18	94.73	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2152	N/A	N/A	N/A	95.17	95.14	94.69	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2132	N/A	N/A	N/A	95.13	95.10	94.64	N/A	N/A	N/A

Table 7B: Existing and Proposed Conditions Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Maximum Water Surface Elevation (m)								
			Existing Conditions			Proposed Conditions			Difference		
			1	*2*	*4*	*1*	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	2112	N/A	N/A	N/A	95.08	95.06	94.60	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2092	N/A	N/A	N/A	95.04	95.02	94.57	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2072	N/A	N/A	N/A	95.00	94.98	94.53	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2052	N/A	N/A	N/A	94.96	94.94	94.49	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2032	N/A	N/A	N/A	94.92	94.90	94.45	N/A	N/A	N/A
Van Gaal Drain	Reach 2	2002	N/A	N/A	N/A	94.86	94.85	94.40	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1982	N/A	N/A	N/A	94.82	94.81	94.37	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1962	N/A	N/A	N/A	94.78	94.77	94.34	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1942	N/A	N/A	N/A	94.75	94.73	94.32	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1922	N/A	N/A	N/A	94.71	94.70	94.29	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1902	N/A	N/A	N/A	94.67	94.66	94.27	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1882	N/A	N/A	N/A	94.63	94.63	94.25	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1862	N/A	N/A	N/A	94.60	94.60	94.23	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1842	N/A	N/A	N/A	94.56	94.56	94.22	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1822	N/A	N/A	N/A	94.53	94.53	94.21	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1802	N/A	N/A	N/A	94.50	94.50	94.20	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1782	N/A	N/A	N/A	94.46	94.48	94.19	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1762	N/A	N/A	N/A	94.43	94.45	94.18	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1742	N/A	N/A	N/A	94.40	94.42	94.17	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1722	N/A	N/A	N/A	94.37	94.40	94.17	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1702	N/A	N/A	N/A	94.35	94.38	94.16	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1682	N/A	N/A	N/A	94.32	94.36	94.16	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1662	N/A	N/A	N/A	94.30	94.34	94.15	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1642	N/A	N/A	N/A	94.27	94.32	94.15	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1622	N/A	N/A	N/A	94.25	94.30	94.15	N/A	N/A	N/A
Van Gaal Drain	Reach 2	1615	94.60	94.61	94.24	94.24	94.30	94.15	-0.36	-0.31	-0.09
Van Gaal Drain	Reach 2	1555	94.53	94.55	94.21	94.19	94.26	94.14	-0.34	-0.29	-0.07
Van Gaal Drain	Reach 2	1488	94.45	94.45	94.18	94.14	94.22	94.14	-0.31	-0.23	-0.04
Van Gaal Drain	Reach 2	1416	94.39	94.41	94.14	94.10	94.19	94.13	-0.29	-0.22	-0.01
Van Gaal Drain	Reach 2	1400	94.36	94.36	94.14	94.08	94.18	94.13	-0.28	-0.18	-0.01
Van Gaal Drain	Reach 2	1364	94.31	94.29	94.13	94.07	94.17	94.13	-0.24	-0.12	0.00
Van Gaal Drain	Reach 2	1340	94.21	94.19	94.13	94.00	94.06	94.12	-0.21	-0.13	-0.01
Van Gaal Drain	Reach 2	1339	Culvert								

Table 7B: Existing and Proposed Conditions Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Maximum Water Surface Elevation (m)								
			Existing Conditions			Proposed Conditions			Difference		
			1	*2*	*4*	*1*	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	1312	94.14	94.12	94.12	93.99	94.04	94.12	-0.15	-0.08	0.00
Van Gaal Drain	Reach 2	1302	94.15	94.14	94.12	93.97	94.04	94.12	-0.18	-0.10	0.00
Van Gaal Drain	Reach 2	1268	94.14	94.14	94.12	93.93	94.01	94.12	-0.21	-0.13	0.00
Van Gaal Drain	Reach 2	1212	94.10	94.11	94.12	93.84	93.96	94.11	-0.26	-0.15	-0.01
Van Gaal Drain	Reach 2	1169	94.04	94.08	94.12	93.76	93.89	94.11	-0.28	-0.19	-0.01
Van Gaal Drain	Reach 2	1091	93.97	94.04	94.12	93.63	93.83	94.11	-0.34	-0.21	-0.01
Van Gaal Drain	Reach 2	1002	93.93	94.02	94.12	93.42	93.78	94.11	-0.51	-0.24	-0.01
Van Gaal Drain	Reach 2	961	93.92	94.02	94.12	93.30	93.77	94.11	-0.62	-0.25	-0.01
Van Gaal Drain	Reach 2	910	93.91	94.02	94.12	93.18	93.77	94.11	-0.73	-0.25	-0.01
Van Gaal Drain	Reach 2	840	93.91	94.02	94.12	92.72	93.76	94.11	-1.19	-0.26	-0.01
Van Gaal Drain	Reach 1	746	93.90	94.01	94.12	92.60	93.76	94.11	-1.30	-0.25	-0.01
Van Gaal Drain	Reach 1	705	93.89	94.01	94.11	92.59	93.75	94.11	-1.30	-0.26	0.00
Van Gaal Drain	Reach 1	668	93.84	93.99	94.11	92.57	93.73	94.11	-1.27	-0.26	0.00
Van Gaal Drain	Reach 1	666	93.32	93.68	94.10	92.57	93.59	94.10	-0.75	-0.09	0.00
Van Gaal Drain	Reach 1	656	Culvert								
Van Gaal Drain	Reach 1	647	93.07	93.58	94.10	92.57	93.59	94.10	-0.50	0.01	0.00
Van Gaal Drain	Reach 1	645	93.23	93.67	94.10	92.57	93.61	94.10	-0.66	-0.06	0.00
Van Gaal Drain	Reach 1	592	93.28	93.70	94.10	92.57	93.63	94.10	-0.71	-0.07	0.00
Van Gaal Drain	Reach 1	521	93.26	93.70	94.10	92.57	93.63	94.10	-0.69	-0.07	0.00
Van Gaal Drain	Reach 1	277	92.96	93.66	94.10	92.57	93.60	94.10	-0.39	-0.06	0.00
Van Gaal Drain	Reach 1	275	92.87	93.51	94.09	92.57	93.48	94.09	-0.30	-0.03	0.00
Van Gaal Drain	Reach 1	269	Culvert								
Van Gaal Drain	Reach 1	263	92.58	93.39	94.09	92.57	93.39	94.09	-0.01	0.00	0.00
Van Gaal Drain	Reach 1	226	92.40	93.44	94.09	92.01	93.43	94.09	-0.39	-0.01	0.00
Van Gaal Drain	Reach 1	0	91.28	93.45	94.09	91.26	93.43	94.09	-0.02	-0.02	0.00
Moore Drain Trib	Reach 1	600	N/A	N/A	N/A	95.74	95.80	95.15	N/A	N/A	N/A
Moore Drain Trib	Reach 1	553	N/A	N/A	N/A	95.71	95.78	95.11	N/A	N/A	N/A
Moore Drain Trib	Reach 1	503	N/A	N/A	N/A	95.69	95.76	95.07	N/A	N/A	N/A
Moore Drain Trib	Reach 1	492	N/A	N/A	N/A	95.64	95.70	95.06	N/A	N/A	N/A
Moore Drain Trib	Reach 1	477	Culvert								
Moore Drain Trib	Reach 1	462	N/A	N/A	N/A	95.52	95.58	94.99	N/A	N/A	N/A
Moore Drain Trib	Reach 1	453	N/A	N/A	N/A	95.54	95.60	94.98	N/A	N/A	N/A
Moore Drain Trib	Reach 1	403	N/A	N/A	N/A	95.51	95.57	94.94	N/A	N/A	N/A

Table 7B: Existing and Proposed Conditions Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Maximum Water Surface Elevation (m)								
			Existing Conditions			Proposed Conditions			Difference		
			1	*2*	*4*	*1*	*2*	*4*	*1*	*2*	*4*
Moore Drain Trib	Reach 1	353	N/A	N/A	N/A	95.40	95.46	94.88	N/A	N/A	N/A
Moore Drain Trib	Reach 1	338.5	Culvert								
Moore Drain Trib	Reach 1	324	N/A	N/A	N/A	95.20	95.24	94.80	N/A	N/A	N/A
Moore Drain Trib	Reach 1	311	94.60	94.58	94.48	95.20	95.25	94.77	0.60	0.67	0.29
Moore Drain Trib	Reach 1	290	N/A	N/A	N/A	95.16	95.21	94.73	N/A	N/A	N/A
Moore Drain Trib	Reach 1	240	N/A	N/A	N/A	95.08	95.14	94.63	N/A	N/A	N/A
Moore Drain Trib	Reach 1	190	N/A	N/A	N/A	95.02	95.08	94.53	N/A	N/A	N/A
Moore Drain Trib	Reach 1	140	N/A	N/A	N/A	94.97	95.03	94.43	N/A	N/A	N/A
Moore Drain Trib	Reach 1	90	94.20	94.18	94.11	94.93	95.00	94.37	0.73	0.82	0.26
Moore Drain Trib	Reach 1	83	N/A	N/A	N/A	94.87	94.93	94.35	N/A	N/A	N/A
Moore Drain Trib	Reach 1	68	Culvert								
Moore Drain Trib	Reach 1	53	N/A	N/A	N/A	94.64	94.68	94.20	N/A	N/A	N/A
Moore Drain Trib	Reach 1	14	N/A	N/A	N/A	94.21	94.24	94.11	N/A	N/A	N/A
Moore Drain	Reach 2	555	94.67	94.63	94.33	94.37	94.41	94.21	-0.30	-0.22	-0.12
Moore Drain	Reach 2	500	94.44	94.41	94.23	94.26	94.28	94.17	-0.18	-0.13	-0.06
Moore Drain	Reach 1	298	93.90	94.02	94.12	93.91	93.82	94.11	0.01	-0.20	-0.01
Moore Drain	Reach 1	130	93.91	94.02	94.12	92.84	93.76	94.11	-1.07	-0.26	-0.01
Joys Road Trib	Reach 1	705	97.59	97.79	97.11	97.35	97.78	97.11	-0.24	-0.01	0.00
Joys Road Trib	Reach 1	664	97.60	97.79	97.04	97.36	97.79	97.04	-0.24	0.00	0.00
Joys Road Trib	Reach 1	635	97.50	97.68	97.00	97.28	97.67	97.00	-0.22	-0.01	0.00
Joys Road Trib	Reach 1	634	Culvert								
Joys Road Trib	Reach 1	622	97.21	97.26	96.96	97.13	97.26	96.96	-0.08	0.00	0.00
Joys Road Trib	Reach 1	602	97.21	97.27	96.95	97.12	97.27	96.95	-0.09	0.00	0.00
Joys Road Trib	Reach 1	322	96.65	96.71	96.45	96.57	96.71	96.45	-0.08	0.00	0.00
Joys Road Trib	Reach 1	275	96.50	96.56	96.20	96.37	96.56	96.20	-0.13	0.00	0.00
Joys Road Trib	Reach 1	30	96.29	96.30	95.95	96.16	96.29	95.88	-0.13	-0.01	-0.07

(1) Scenario Descriptions:

1. The Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River.
2. The Van Gaal Drain 100-year spring snowmelt plus rainfall peak flow reaches the Jock River.
4. The Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain.

ATTACHMENT

7

2 TO 25-YEAR FLOWS AND WATER LEVELS ON THE VAN GAAL DRAIN UNDER EXISTING AND PROPOSED CONDITIONS

JFSA Water Resources and Environmental Consultants

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 3	3494	2-Year	0.96	2.09	2.06	97.01	97.27	97.26
Van Gaal Drain	Reach 3	3494	5-Year	1.60	2.68	1.13	97.18	97.37	97.04
Van Gaal Drain	Reach 3	3494	10-Year	2.06	3.06	0.91	97.28	97.43	96.98
Van Gaal Drain	Reach 3	3494	25-Year	2.67	3.53	1.27	97.39	97.50	97.08
Van Gaal Drain	Reach 3	3322	2-Year	1.05	2.27	2.21	96.92	97.16	97.16
Van Gaal Drain	Reach 3	3322	5-Year	1.73	2.90	1.15	97.07	97.26	96.95
Van Gaal Drain	Reach 3	3322	10-Year	2.24	3.32	0.92	97.17	97.31	96.90
Van Gaal Drain	Reach 3	3322	25-Year	2.90	3.83	1.29	97.28	97.38	96.98
Van Gaal Drain	Reach 3	3312	2-Year	1.05	2.27	2.21	96.92	97.16	97.15
Van Gaal Drain	Reach 3	3312	5-Year	1.73	2.90	1.15	97.07	97.24	96.95
Van Gaal Drain	Reach 3	3312	10-Year	2.24	3.32	0.92	97.17	97.29	96.90
Van Gaal Drain	Reach 3	3312	25-Year	2.90	3.83	1.29	97.27	97.36	96.98
Van Gaal Drain	Reach 3	3311	Culvert						
Van Gaal Drain	Reach 3	3302	2-Year	1.05	2.27	2.21	96.91	97.08	97.08
Van Gaal Drain	Reach 3	3302	5-Year	1.73	2.90	1.15	97.03	97.14	96.93
Van Gaal Drain	Reach 3	3302	10-Year	2.24	3.32	0.92	97.09	97.17	96.89
Van Gaal Drain	Reach 3	3302	25-Year	2.90	3.83	1.29	97.17	97.24	96.96
Van Gaal Drain	Reach 3	3297	2-Year	1.05	2.27	2.21	96.87	97.04	97.05
Van Gaal Drain	Reach 3	3297	5-Year	1.73	2.90	1.15	96.98	97.09	96.91
Van Gaal Drain	Reach 3	3297	10-Year	2.24	3.32	0.92	97.04	97.13	96.87
Van Gaal Drain	Reach 3	3297	25-Year	2.90	3.83	1.29	97.11	97.20	96.93
Van Gaal Drain	Reach 3	3185	2-Year	1.05	2.27	2.21	96.52	96.72	96.68
Van Gaal Drain	Reach 3	3185	5-Year	1.73	2.90	1.15	96.65	96.87	96.46
Van Gaal Drain	Reach 3	3185	10-Year	2.24	3.32	0.92	96.76	96.97	96.39
Van Gaal Drain	Reach 3	3185	25-Year	2.90	3.83	1.29	96.93	97.09	96.49
Van Gaal Drain	Reach 3	3175	2-Year	1.33	2.59	2.40	96.53	96.74	96.70
Van Gaal Drain	Reach 3	3175	5-Year	2.15	3.31	1.17	96.67	96.87	96.49
Van Gaal Drain	Reach 3	3175	10-Year	2.74	3.79	0.92	96.77	96.96	96.43
Van Gaal Drain	Reach 3	3175	25-Year	3.51	4.37	1.30	96.93	97.07	96.53
Van Gaal Drain	Reach 3	3174	Culvert						
Van Gaal Drain	Reach 3	3165	2-Year	1.33	2.59	2.40	96.50	96.65	96.64
Van Gaal Drain	Reach 3	3165	5-Year	2.15	3.31	1.17	96.61	96.70	96.47
Van Gaal Drain	Reach 3	3165	10-Year	2.74	3.79	0.92	96.67	96.72	96.42
Van Gaal Drain	Reach 3	3165	25-Year	3.51	4.37	1.30	96.72	96.74	96.50
Van Gaal Drain	Reach 3	3149	2-Year	1.33	2.59	2.40	96.46	96.60	96.58
Van Gaal Drain	Reach 3	3149	5-Year	2.15	3.31	1.17	96.57	96.64	96.43
Van Gaal Drain	Reach 3	3149	10-Year	2.74	3.79	0.92	96.62	96.66	96.37
Van Gaal Drain	Reach 3	3149	25-Year	3.51	4.37	1.30	96.66	96.68	96.46
Van Gaal Drain	Reach 3	3086	2-Year	1.33	2.59	2.40	96.35	96.50	96.49
Van Gaal Drain	Reach 3	3086	5-Year	2.15	3.31	1.17	96.47	96.55	96.31
Van Gaal Drain	Reach 3	3086	10-Year	2.74	3.79	0.92	96.53	96.58	96.25
Van Gaal Drain	Reach 3	3086	25-Year	3.51	4.37	1.30	96.58	96.60	96.34
Van Gaal Drain	Reach 3	3016	2-Year	1.33	2.59	2.40	96.26	96.46	96.44

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 3	3016	5-Year	2.15	3.31	1.17	96.41	96.52	96.23
Van Gaal Drain	Reach 3	3016	10-Year	2.74	3.79	0.92	96.48	96.54	96.17
Van Gaal Drain	Reach 3	3016	25-Year	3.51	4.37	1.30	96.54	96.56	96.26
Van Gaal Drain	Reach 3	2980	2-Year	1.33	2.59	2.40	96.21	96.42	96.40
Van Gaal Drain	Reach 3	2980	5-Year	2.15	3.31	1.17	96.36	96.49	96.18
Van Gaal Drain	Reach 3	2980	10-Year	2.74	3.79	0.92	96.44	96.51	96.12
Van Gaal Drain	Reach 3	2980	25-Year	3.51	4.37	1.30	96.51	96.54	96.21
Van Gaal Drain	Reach 3	2851	2-Year	1.33	2.59	2.40	95.90	96.21	96.19
Van Gaal Drain	Reach 3	2851	5-Year	2.15	3.31	1.17	96.10	96.29	95.89
Van Gaal Drain	Reach 3	2851	10-Year	2.74	3.79	0.92	96.21	96.34	95.80
Van Gaal Drain	Reach 3	2851	25-Year	3.51	4.37	1.30	96.31	96.38	95.93
Van Gaal Drain	Reach 3	2808	2-Year	1.33	2.59	2.40	95.86	96.18	96.16
Van Gaal Drain	Reach 3	2808	5-Year	2.15	3.31	1.17	96.07	96.26	95.86
Van Gaal Drain	Reach 3	2808	10-Year	2.74	3.79	0.92	96.18	96.31	95.76
Van Gaal Drain	Reach 3	2808	25-Year	3.51	4.37	1.30	96.28	96.35	95.90
Van Gaal Drain	Reach 3	2658	2-Year	1.33	2.59	2.40	95.78	96.11	96.10
Van Gaal Drain	Reach 3	2658	5-Year	2.15	3.31	1.17	95.99	96.18	95.80
Van Gaal Drain	Reach 3	2658	10-Year	2.74	3.79	0.92	96.10	96.22	95.70
Van Gaal Drain	Reach 3	2658	25-Year	3.51	4.37	1.30	96.19	96.25	95.85
Van Gaal Drain	Reach 2	2554	2-Year	1.95	4.13	3.97	95.77	96.10	96.09
Van Gaal Drain	Reach 2	2554	5-Year	3.20	5.24	2.02	95.98	96.17	95.79
Van Gaal Drain	Reach 2	2554	10-Year	4.11	6.00	1.57	96.09	96.21	95.69
Van Gaal Drain	Reach 2	2554	25-Year	5.28	6.94	2.25	96.18	96.24	95.84
Van Gaal Drain	Reach 2	2478	2-Year	1.95	4.13	3.97	95.72	96.03	96.02
Van Gaal Drain	Reach 2	2478	5-Year	3.20	5.24	2.02	95.92	96.08	95.74
Van Gaal Drain	Reach 2	2478	10-Year	4.11	6.00	1.57	96.02	96.11	95.64
Van Gaal Drain	Reach 2	2478	25-Year	5.28	6.94	2.25	96.09	96.13	95.78
Van Gaal Drain	Reach 2	2157	2-Year	1.95	4.13	3.97	94.93	95.19	95.17
Van Gaal Drain	Reach 2	2157	5-Year	3.20	5.24	2.02	95.12	95.34	94.94
Van Gaal Drain	Reach 2	2157	10-Year	4.11	6.00	1.57	95.25	95.38	94.88
Van Gaal Drain	Reach 2	2157	25-Year	5.28	6.94	2.25	95.37	95.43	94.97
Van Gaal Drain	Reach 2	2076	2-Year	2.80	5.00	4.78	94.74	95.03	95.01
Van Gaal Drain	Reach 2	2076	5-Year	4.41	6.32	2.33	94.97	95.13	94.65
Van Gaal Drain	Reach 2	2076	10-Year	5.53	7.24	1.78	95.08	95.17	94.53
Van Gaal Drain	Reach 2	2076	25-Year	6.96	8.38	2.60	95.17	95.21	94.70
Van Gaal Drain	Reach 2	1974	2-Year	2.80	5.00	4.78	94.61	94.89	94.87
Van Gaal Drain	Reach 2	1974	5-Year	4.41	6.32	2.33	94.84	94.97	94.50
Van Gaal Drain	Reach 2	1974	10-Year	5.53	7.24	1.78	94.93	95.02	94.37
Van Gaal Drain	Reach 2	1974	25-Year	6.96	8.38	2.60	95.01	95.06	94.56
Van Gaal Drain	Reach 2	1922	2-Year	2.80	5.00	4.78	94.55	94.82	94.81
Van Gaal Drain	Reach 2	1922	5-Year	4.41	6.32	2.33	94.79	94.89	94.44
Van Gaal Drain	Reach 2	1922	10-Year	5.53	7.24	1.78	94.85	94.92	94.30
Van Gaal Drain	Reach 2	1922	25-Year	6.96	8.38	2.60	94.92	94.95	94.50
Van Gaal Drain	Reach 2	1833	2-Year	2.80	5.00	4.78	94.47	94.71	94.71

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	1833	5-Year	4.41	6.32	2.33	94.69	94.76	94.35
Van Gaal Drain	Reach 2	1833	10-Year	5.53	7.24	1.78	94.74	94.78	94.20
Van Gaal Drain	Reach 2	1833	25-Year	6.96	8.38	2.60	94.78	94.80	94.42
Van Gaal Drain	Reach 2	1796	2-Year	2.80	5.00	4.78	94.44	94.68	94.68
Van Gaal Drain	Reach 2	1796	5-Year	4.41	6.32	2.33	94.66	94.73	94.32
Van Gaal Drain	Reach 2	1796	10-Year	5.53	7.24	1.78	94.70	94.74	94.16
Van Gaal Drain	Reach 2	1796	25-Year	6.96	8.38	2.60	94.74	94.77	94.39
Van Gaal Drain	Reach 2	1735	2-Year	2.80	5.00	4.78	94.41	94.64	94.64
Van Gaal Drain	Reach 2	1735	5-Year	4.41	6.32	2.33	94.63	94.67	94.28
Van Gaal Drain	Reach 2	1735	10-Year	5.53	7.24	1.78	94.65	94.68	94.12
Van Gaal Drain	Reach 2	1735	25-Year	6.96	8.38	2.60	94.68	94.69	94.36
Van Gaal Drain	Reach 2	1728	2-Year	2.80	5.00	4.78	94.41	94.64	94.63
Van Gaal Drain	Reach 2	1728	5-Year	4.41	6.32	2.33	94.62	94.67	94.28
Van Gaal Drain	Reach 2	1728	10-Year	5.53	7.24	1.78	94.65	94.67	94.11
Van Gaal Drain	Reach 2	1728	25-Year	6.96	8.38	2.60	94.67	94.68	94.35
Van Gaal Drain	Reach 2	1727	Culvert						
Van Gaal Drain	Reach 2	1717	2-Year	2.80	5.00	4.78	94.15	94.42	94.40
Van Gaal Drain	Reach 2	1717	5-Year	4.41	6.32	2.33	94.37	94.55	94.05
Van Gaal Drain	Reach 2	1717	10-Year	5.53	7.24	1.78	94.49	94.59	93.92
Van Gaal Drain	Reach 2	1717	25-Year	6.96	8.38	2.60	94.59	94.63	94.10
Van Gaal Drain	Reach 2	1615	2-Year	2.80	5.00	4.78	94.04	94.29	94.26
Van Gaal Drain	Reach 2	1615	5-Year	4.41	6.32	2.33	94.24	94.40	93.92
Van Gaal Drain	Reach 2	1615	10-Year	5.53	7.24	1.78	94.36	94.46	93.78
Van Gaal Drain	Reach 2	1615	25-Year	6.96	8.38	2.60	94.46	94.52	93.98
Van Gaal Drain	Reach 2	1555	2-Year	2.80	5.00	4.78	93.99	94.22	94.19
Van Gaal Drain	Reach 2	1555	5-Year	4.41	6.32	2.33	94.19	94.33	93.87
Van Gaal Drain	Reach 2	1555	10-Year	5.53	7.24	1.78	94.29	94.39	93.72
Van Gaal Drain	Reach 2	1555	25-Year	6.96	8.38	2.60	94.40	94.45	93.93
Van Gaal Drain	Reach 2	1488	2-Year	2.80	5.00	4.78	93.96	94.16	94.13
Van Gaal Drain	Reach 2	1488	5-Year	4.41	6.32	2.33	94.13	94.26	93.82
Van Gaal Drain	Reach 2	1488	10-Year	5.53	7.24	1.78	94.23	94.31	93.66
Van Gaal Drain	Reach 2	1488	25-Year	6.96	8.38	2.60	94.33	94.36	93.88
Van Gaal Drain	Reach 2	1416	2-Year	2.80	5.00	4.78	93.89	94.02	93.99
Van Gaal Drain	Reach 2	1416	5-Year	4.41	6.32	2.33	94.03	94.10	93.74
Van Gaal Drain	Reach 2	1416	10-Year	5.53	7.24	1.78	94.10	94.17	93.58
Van Gaal Drain	Reach 2	1416	25-Year	6.96	8.38	2.60	94.20	94.25	93.81
Van Gaal Drain	Reach 2	1400	2-Year	2.80	5.00	4.78	93.89	94.02	93.99
Van Gaal Drain	Reach 2	1400	5-Year	4.41	6.32	2.33	94.03	94.11	93.74
Van Gaal Drain	Reach 2	1400	10-Year	5.53	7.24	1.78	94.11	94.16	93.58
Van Gaal Drain	Reach 2	1400	25-Year	6.96	8.38	2.60	94.20	94.23	93.80
Van Gaal Drain	Reach 2	1364	2-Year	2.80	5.00	4.78	93.87	93.99	93.96
Van Gaal Drain	Reach 2	1364	5-Year	4.41	6.32	2.33	94.00	94.06	93.72
Van Gaal Drain	Reach 2	1364	10-Year	5.53	7.24	1.78	94.07	94.11	93.56
Van Gaal Drain	Reach 2	1364	25-Year	6.96	8.38	2.60	94.16	94.18	93.79

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	1340	2-Year	3.64	5.79	5.26	93.85	93.96	93.93
Van Gaal Drain	Reach 2	1340	5-Year	5.57	7.32	2.44	93.97	94.01	93.71
Van Gaal Drain	Reach 2	1340	10-Year	6.92	8.33	1.86	94.03	94.05	93.55
Van Gaal Drain	Reach 2	1340	25-Year	8.58	9.65	2.71	94.09	94.10	93.78
Van Gaal Drain	Reach 2	1339	Culvert						
Van Gaal Drain	Reach 2	1312	2-Year	3.87	6.08	5.46	93.84	93.94	93.91
Van Gaal Drain	Reach 2	1312	5-Year	5.93	7.69	2.47	93.95	93.98	93.71
Van Gaal Drain	Reach 2	1312	10-Year	7.38	8.76	1.87	94.00	94.01	93.55
Van Gaal Drain	Reach 2	1312	25-Year	9.17	10.15	2.73	94.05	94.04	93.77
Van Gaal Drain	Reach 2	1302	2-Year	3.87	6.08	5.46	93.81	93.91	93.89
Van Gaal Drain	Reach 2	1302	5-Year	5.93	7.69	2.47	93.92	93.97	93.68
Van Gaal Drain	Reach 2	1302	10-Year	7.38	8.76	1.87	93.98	94.00	93.51
Van Gaal Drain	Reach 2	1302	25-Year	9.17	10.15	2.73	94.04	94.04	93.75
Van Gaal Drain	Reach 2	1268	2-Year	3.87	6.08	5.46	93.75	93.87	93.84
Van Gaal Drain	Reach 2	1268	5-Year	5.93	7.69	2.47	93.88	93.93	93.59
Van Gaal Drain	Reach 2	1268	10-Year	7.38	8.76	1.87	93.94	93.96	93.44
Van Gaal Drain	Reach 2	1268	25-Year	9.17	10.15	2.73	94.01	94.00	93.69
Van Gaal Drain	Reach 2	1212	2-Year	3.87	6.08	5.46	93.61	93.78	93.74
Van Gaal Drain	Reach 2	1212	5-Year	5.93	7.69	2.47	93.78	93.85	93.41
Van Gaal Drain	Reach 2	1212	10-Year	7.38	8.76	1.87	93.85	93.89	93.32
Van Gaal Drain	Reach 2	1212	25-Year	9.17	10.15	2.73	93.93	93.94	93.58
Van Gaal Drain	Reach 2	1169	2-Year	3.87	6.08	5.46	93.53	93.70	93.67
Van Gaal Drain	Reach 2	1169	5-Year	5.93	7.69	2.47	93.70	93.76	93.32
Van Gaal Drain	Reach 2	1169	10-Year	7.38	8.76	1.87	93.77	93.80	93.26
Van Gaal Drain	Reach 2	1169	25-Year	9.17	10.15	2.73	93.85	93.87	93.54
Van Gaal Drain	Reach 2	1091	2-Year	3.87	6.08	5.46	93.40	93.57	93.54
Van Gaal Drain	Reach 2	1091	5-Year	5.93	7.69	2.47	93.57	93.64	93.19
Van Gaal Drain	Reach 2	1091	10-Year	7.38	8.76	1.87	93.65	93.69	93.17
Van Gaal Drain	Reach 2	1091	25-Year	9.17	10.15	2.73	93.75	93.78	93.50
Van Gaal Drain	Reach 2	1002	2-Year	3.87	6.08	5.46	93.21	93.38	93.33
Van Gaal Drain	Reach 2	1002	5-Year	5.93	7.69	2.47	93.39	93.49	93.01
Van Gaal Drain	Reach 2	1002	10-Year	7.38	8.76	1.87	93.50	93.59	93.09
Van Gaal Drain	Reach 2	1002	25-Year	9.17	10.15	2.73	93.65	93.71	93.47
Van Gaal Drain	Reach 2	961	2-Year	3.87	6.08	5.46	93.14	93.28	93.23
Van Gaal Drain	Reach 2	961	5-Year	5.93	7.69	2.47	93.31	93.41	92.97
Van Gaal Drain	Reach 2	961	10-Year	7.38	8.76	1.87	93.44	93.52	93.06
Van Gaal Drain	Reach 2	961	25-Year	9.17	10.15	2.73	93.61	93.69	93.46
Van Gaal Drain	Reach 2	910	2-Year	3.87	6.08	5.46	93.07	93.22	93.17
Van Gaal Drain	Reach 2	910	5-Year	5.93	7.69	2.47	93.25	93.38	92.91
Van Gaal Drain	Reach 2	910	10-Year	7.38	8.76	1.87	93.40	93.49	93.05
Van Gaal Drain	Reach 2	910	25-Year	9.17	10.15	2.73	93.59	93.68	93.46
Van Gaal Drain	Reach 2	840	2-Year	3.87	6.08	5.46	93.00	93.18	93.11
Van Gaal Drain	Reach 2	840	5-Year	5.93	7.69	2.47	93.22	93.35	92.83

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	840	10-Year	7.38	8.76	1.87	93.38	93.48	93.03
Van Gaal Drain	Reach 2	840	25-Year	9.17	10.15	2.73	93.58	93.67	93.45
Van Gaal Drain	Reach 1	746	2-Year	5.36	7.86	7.06	92.96	93.17	93.10
Van Gaal Drain	Reach 1	746	5-Year	8.09	9.97	2.99	93.20	93.35	92.82
Van Gaal Drain	Reach 1	746	10-Year	10.02	11.34	2.11	93.37	93.48	93.03
Van Gaal Drain	Reach 1	746	25-Year	12.40	13.11	3.31	93.57	93.67	93.45
Van Gaal Drain	Reach 1	705	2-Year	5.36	7.86	7.06	92.88	93.14	93.06
Van Gaal Drain	Reach 1	705	5-Year	8.09	9.97	2.99	93.16	93.34	92.78
Van Gaal Drain	Reach 1	705	10-Year	10.02	11.34	2.11	93.34	93.47	93.03
Van Gaal Drain	Reach 1	705	25-Year	12.40	13.11	3.31	93.55	93.66	93.45
Van Gaal Drain	Reach 1	668	2-Year	5.36	7.86	7.06	92.72	93.05	92.94
Van Gaal Drain	Reach 1	668	5-Year	8.09	9.97	2.99	93.04	93.28	92.74
Van Gaal Drain	Reach 1	668	10-Year	10.02	11.34	2.11	93.25	93.42	93.02
Van Gaal Drain	Reach 1	668	25-Year	12.40	13.11	3.31	93.48	93.63	93.45
Van Gaal Drain	Reach 1	666	2-Year	5.36	7.86	7.06	92.48	92.64	92.59
Van Gaal Drain	Reach 1	666	5-Year	8.09	9.97	2.99	92.66	92.83	92.72
Van Gaal Drain	Reach 1	666	10-Year	10.02	11.34	2.11	92.80	93.01	93.01
Van Gaal Drain	Reach 1	666	25-Year	12.40	13.11	3.31	93.03	93.26	93.43
Van Gaal Drain	Reach 1	656	Culvert						
Van Gaal Drain	Reach 1	647	2-Year	5.36	7.86	7.06	92.51	92.59	92.58
Van Gaal Drain	Reach 1	647	5-Year	8.09	9.97	2.99	92.66	92.78	92.72
Van Gaal Drain	Reach 1	647	10-Year	10.02	11.34	2.11	92.77	92.92	93.01
Van Gaal Drain	Reach 1	647	25-Year	12.40	13.11	3.31	92.90	93.17	93.43
Van Gaal Drain	Reach 1	645	2-Year	5.36	7.86	7.06	92.53	92.66	92.63
Van Gaal Drain	Reach 1	645	5-Year	8.09	9.97	2.99	92.72	92.87	92.72
Van Gaal Drain	Reach 1	645	10-Year	10.02	11.34	2.11	92.85	93.02	93.01
Van Gaal Drain	Reach 1	645	25-Year	12.40	13.11	3.31	93.00	93.26	93.43
Van Gaal Drain	Reach 1	592	2-Year	5.36	7.86	7.06	92.42	92.66	92.62
Van Gaal Drain	Reach 1	592	5-Year	8.09	9.97	2.99	92.68	92.89	92.73
Van Gaal Drain	Reach 1	592	10-Year	10.02	11.34	2.11	92.84	93.05	93.01
Van Gaal Drain	Reach 1	592	25-Year	12.40	13.11	3.31	93.02	93.29	93.44
Van Gaal Drain	Reach 1	521	2-Year	5.36	7.86	7.06	92.41	92.65	92.61
Van Gaal Drain	Reach 1	521	5-Year	8.09	9.97	2.99	92.67	92.88	92.72
Van Gaal Drain	Reach 1	521	10-Year	10.02	11.34	2.11	92.83	93.04	93.01
Van Gaal Drain	Reach 1	521	25-Year	12.40	13.11	3.31	93.00	93.29	93.43
Van Gaal Drain	Reach 1	277	2-Year	5.36	7.86	7.06	92.24	92.49	92.48
Van Gaal Drain	Reach 1	277	5-Year	8.09	9.97	2.99	92.46	92.74	92.71
Van Gaal Drain	Reach 1	277	10-Year	10.02	11.34	2.11	92.61	92.89	93.01
Van Gaal Drain	Reach 1	277	25-Year	12.40	13.11	3.31	92.76	93.19	93.43
Van Gaal Drain	Reach 1	275	2-Year	5.36	7.86	7.06	92.24	92.46	92.46
Van Gaal Drain	Reach 1	275	5-Year	8.09	9.97	2.99	92.45	92.68	92.70
Van Gaal Drain	Reach 1	275	10-Year	10.02	11.34	2.11	92.57	92.83	93.01
Van Gaal Drain	Reach 1	275	25-Year	12.40	13.11	3.31	92.69	93.09	93.43

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	2	4	1	2	4
Van Gaal Drain	Reach 1	269	Culvert						
Van Gaal Drain	Reach 1	263	2-Year	5.36	7.86	7.06	92.20	92.39	92.41
Van Gaal Drain	Reach 1	263	5-Year	8.09	9.97	2.99	92.38	92.60	92.70
Van Gaal Drain	Reach 1	263	10-Year	10.02	11.34	2.11	92.47	92.75	93.01
Van Gaal Drain	Reach 1	263	25-Year	12.40	13.11	3.31	92.54	93.00	93.42
Van Gaal Drain	Reach 1	226	2-Year	5.37	7.88	7.23	92.14	92.35	92.37
Van Gaal Drain	Reach 1	226	5-Year	8.11	10.01	3.23	92.31	92.59	92.70
Van Gaal Drain	Reach 1	226	10-Year	10.03	11.37	2.30	92.41	92.76	93.01
Van Gaal Drain	Reach 1	226	25-Year	12.43	13.14	3.54	92.47	93.04	93.42
Van Gaal Drain	Reach 1	0	2-Year	5.37	7.88	7.23	91.09	92.41	92.42
Van Gaal Drain	Reach 1	0	5-Year	8.11	10.01	3.23	91.11	92.63	92.70
Van Gaal Drain	Reach 1	0	10-Year	10.03	11.37	2.30	91.13	92.79	93.01
Van Gaal Drain	Reach 1	0	25-Year	12.43	13.14	3.54	91.17	93.06	93.42
Moore Drain Trib	Reach 1	311	2-Year	0.52	0.79	0.62	94.55	94.56	94.55
Moore Drain Trib	Reach 1	311	5-Year	0.84	1.01	0.17	94.57	94.56	94.48
Moore Drain Trib	Reach 1	311	10-Year	1.07	1.33	0.10	94.57	94.57	94.42
Moore Drain Trib	Reach 1	311	25-Year	1.36	1.33	0.15	94.58	94.57	94.47
Moore Drain Trib	Reach 1	90	2-Year	0.52	0.79	0.62	94.01	94.07	94.04
Moore Drain Trib	Reach 1	90	5-Year	0.84	1.01	0.17	94.08	94.15	93.89
Moore Drain Trib	Reach 1	90	10-Year	1.07	1.33	0.10	94.16	94.17	93.85
Moore Drain Trib	Reach 1	90	25-Year	1.36	1.33	0.15	94.18	94.17	93.88
Moore Drain	Reach 2	555	2-Year	0.26	0.34	0.24	94.50	94.53	94.49
Moore Drain	Reach 2	555	5-Year	0.41	0.43	1755.00	94.56	94.56	96.67
Moore Drain	Reach 2	555	10-Year	0.52	0.49	0.02	94.59	94.58	94.32
Moore Drain	Reach 2	555	25-Year	0.66	0.57	0.05	94.63	94.60	94.35
Moore Drain	Reach 2	500	2-Year	0.26	0.34	0.24	94.33	94.35	94.33
Moore Drain	Reach 2	500	5-Year	0.41	0.43	1755.00	94.37	94.37	95.87
Moore Drain	Reach 2	500	10-Year	0.52	0.49	0.02	94.39	94.38	94.22
Moore Drain	Reach 2	500	25-Year	0.66	0.57	0.05	94.41	94.40	94.24
Moore Drain	Reach 1	298	2-Year	0.67	1.07	1.01	93.68	93.79	93.78
Moore Drain	Reach 1	298	5-Year	1.05	1.36	0.37	93.78	93.81	93.60
Moore Drain	Reach 1	298	10-Year	1.32	1.55	0.15	93.81	93.82	93.52
Moore Drain	Reach 1	298	25-Year	1.64	1.78	0.36	93.83	93.83	93.60
Moore Drain	Reach 1	130	2-Year	0.67	1.07	1.01	93.00	93.18	93.11
Moore Drain	Reach 1	130	5-Year	1.05	1.36	0.37	93.22	93.36	92.84
Moore Drain	Reach 1	130	10-Year	1.32	1.55	0.15	93.38	93.48	93.03
Moore Drain	Reach 1	130	25-Year	1.64	1.78	0.36	93.58	93.67	93.45
Joys Road Trib	Reach 1	705	2-Year	0.66	1.57	1.46	97.07	97.24	97.20
Joys Road Trib	Reach 1	705	5-Year	1.11	2.00	0.69	97.14	97.39	97.08
Joys Road Trib	Reach 1	705	10-Year	1.44	2.29	0.51	97.19	97.49	97.04
Joys Road Trib	Reach 1	705	25-Year	1.87	2.64	0.75	97.35	97.60	97.08
Joys Road Trib	Reach 1	664	2-Year	0.66	1.57	1.46	96.97	97.26	97.23
Joys Road Trib	Reach 1	664	5-Year	1.11	2.00	0.69	97.12	97.40	96.98
Joys Road Trib	Reach 1	664	10-Year	1.44	2.29	0.51	97.22	97.49	96.92

Table 8A: Existing Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Joys Road Trib	Reach 1	664	25-Year	1.87	2.64	0.75	97.36	97.61	97.00
Joys Road Trib	Reach 1	635	2-Year	0.66	1.57	1.46	96.94	97.19	97.16
Joys Road Trib	Reach 1	635	5-Year	1.11	2.00	0.69	97.07	97.31	96.95
Joys Road Trib	Reach 1	635	10-Year	1.44	2.29	0.51	97.16	97.40	96.89
Joys Road Trib	Reach 1	635	25-Year	1.87	2.64	0.75	97.28	97.50	96.96
Joys Road Trib	Reach 1	634	Culvert						
Joys Road Trib	Reach 1	622	2-Year	0.66	1.57	1.46	96.91	97.08	97.06
Joys Road Trib	Reach 1	622	5-Year	1.11	2.00	0.69	97.00	97.14	96.92
Joys Road Trib	Reach 1	622	10-Year	1.44	2.29	0.51	97.06	97.18	96.87
Joys Road Trib	Reach 1	622	25-Year	1.87	2.64	0.75	97.13	97.21	96.93
Joys Road Trib	Reach 1	602	2-Year	0.66	1.57	1.46	96.90	97.07	97.05
Joys Road Trib	Reach 1	602	5-Year	1.11	2.00	0.69	96.99	97.14	96.91
Joys Road Trib	Reach 1	602	10-Year	1.44	2.29	0.51	97.05	97.17	96.86
Joys Road Trib	Reach 1	602	25-Year	1.87	2.64	0.75	97.12	97.22	96.92
Joys Road Trib	Reach 1	322	2-Year	0.66	1.57	1.46	96.41	96.54	96.53
Joys Road Trib	Reach 1	322	5-Year	1.11	2.00	0.69	96.49	96.58	96.41
Joys Road Trib	Reach 1	322	10-Year	1.44	2.29	0.51	96.53	96.61	96.38
Joys Road Trib	Reach 1	322	25-Year	1.87	2.64	0.75	96.57	96.65	96.42
Joys Road Trib	Reach 1	275	2-Year	0.66	1.57	1.46	96.18	96.31	96.28
Joys Road Trib	Reach 1	275	5-Year	1.11	2.00	0.69	96.23	96.39	96.19
Joys Road Trib	Reach 1	275	10-Year	1.44	2.29	0.51	96.28	96.44	96.16
Joys Road Trib	Reach 1	275	25-Year	1.87	2.64	0.75	96.37	96.49	96.19
Joys Road Trib	Reach 1	30	2-Year	0.66	1.57	1.46	95.78	96.12	96.10
Joys Road Trib	Reach 1	30	5-Year	1.11	2.00	0.69	96.00	96.19	95.80
Joys Road Trib	Reach 1	30	10-Year	1.44	2.29	0.51	96.11	96.22	95.70
Joys Road Trib	Reach 1	30	25-Year	1.87	2.64	0.75	96.20	96.26	95.85

(1) Scenario Descriptions:

1. The Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River.
2. The Van Gaal Drain 100-year spring snowmelt plus rainfall peak flow reaches the Jock River.
4. The Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain.

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 3	3494	2-Year	0.79	2.08	0.87	96.97	97.27	96.98
Van Gaal Drain	Reach 3	3494	5-Year	1.25	2.68	0.79	97.10	97.37	96.95
Van Gaal Drain	Reach 3	3494	10-Year	1.57	2.97	0.91	97.18	97.42	96.98
Van Gaal Drain	Reach 3	3494	25-Year	1.92	3.40	1.06	97.26	97.49	97.02
Van Gaal Drain	Reach 3	3322	2-Year	0.89	2.27	0.90	96.88	97.16	96.89
Van Gaal Drain	Reach 3	3322	5-Year	1.39	2.90	0.80	97.00	97.26	96.87
Van Gaal Drain	Reach 3	3322	10-Year	1.74	3.27	0.92	97.07	97.30	96.90
Van Gaal Drain	Reach 3	3322	25-Year	2.16	3.75	1.07	97.16	97.37	96.93
Van Gaal Drain	Reach 3	3312	2-Year	0.89	2.27	0.90	96.88	97.16	96.89
Van Gaal Drain	Reach 3	3312	5-Year	1.39	2.90	0.80	97.00	97.24	96.88
Van Gaal Drain	Reach 3	3312	10-Year	1.74	3.27	0.92	97.07	97.29	96.90
Van Gaal Drain	Reach 3	3312	25-Year	2.16	3.75	1.07	97.15	97.35	96.93
Van Gaal Drain	Reach 3	3311	Culvert						
Van Gaal Drain	Reach 3	3302	2-Year	0.89	2.27	0.90	96.87	97.08	96.88
Van Gaal Drain	Reach 3	3302	5-Year	1.39	2.90	0.80	96.97	97.14	96.87
Van Gaal Drain	Reach 3	3302	10-Year	1.74	3.27	0.92	97.03	97.17	96.89
Van Gaal Drain	Reach 3	3302	25-Year	2.16	3.75	1.07	97.08	97.24	96.92
Van Gaal Drain	Reach 3	3297	2-Year	0.89	2.27	0.90	96.84	97.04	96.86
Van Gaal Drain	Reach 3	3297	5-Year	1.39	2.90	0.80	96.93	97.09	96.85
Van Gaal Drain	Reach 3	3297	10-Year	1.74	3.27	0.92	96.98	97.13	96.87
Van Gaal Drain	Reach 3	3297	25-Year	2.16	3.75	1.07	97.03	97.20	96.90
Van Gaal Drain	Reach 3	3185	2-Year	0.89	2.27	0.90	96.48	96.72	96.39
Van Gaal Drain	Reach 3	3185	5-Year	1.39	2.90	0.80	96.59	96.86	96.35
Van Gaal Drain	Reach 3	3185	10-Year	1.74	3.27	0.92	96.65	96.97	96.39
Van Gaal Drain	Reach 3	3185	25-Year	2.16	3.75	1.07	96.76	97.09	96.43
Van Gaal Drain	Reach 3	3175	2-Year	1.17	2.58	0.90	96.50	96.73	96.43
Van Gaal Drain	Reach 3	3175	5-Year	1.72	3.25	0.80	96.60	96.86	96.40
Van Gaal Drain	Reach 3	3175	10-Year	2.12	3.78	0.92	96.66	96.96	96.43
Van Gaal Drain	Reach 3	3175	25-Year	2.72	4.36	1.07	96.77	97.07	96.47
Van Gaal Drain	Reach 3	3174	Culvert						
Van Gaal Drain	Reach 3	3165	2-Year	1.17	2.58	0.90	96.47	96.65	96.41
Van Gaal Drain	Reach 3	3165	5-Year	1.72	3.25	0.80	96.56	96.70	96.38
Van Gaal Drain	Reach 3	3165	10-Year	2.12	3.78	0.92	96.61	96.72	96.42
Van Gaal Drain	Reach 3	3165	25-Year	2.72	4.36	1.07	96.67	96.74	96.45
Van Gaal Drain	Reach 3	3149	2-Year	1.17	2.58	0.90	96.43	96.60	96.37
Van Gaal Drain	Reach 3	3149	5-Year	1.72	3.25	0.80	96.52	96.64	96.34
Van Gaal Drain	Reach 3	3149	10-Year	2.12	3.78	0.92	96.56	96.66	96.37
Van Gaal Drain	Reach 3	3149	25-Year	2.72	4.36	1.07	96.62	96.68	96.41
Van Gaal Drain	Reach 3	3086	2-Year	1.17	2.58	0.90	96.31	96.50	96.25
Van Gaal Drain	Reach 3	3086	5-Year	1.72	3.25	0.80	96.41	96.55	96.22
Van Gaal Drain	Reach 3	3086	10-Year	2.12	3.78	0.92	96.47	96.58	96.25
Van Gaal Drain	Reach 3	3086	25-Year	2.72	4.36	1.07	96.52	96.60	96.29
Van Gaal Drain	Reach 3	3016	2-Year	1.17	2.58	0.90	96.23	96.45	96.17

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 3	3016	5-Year	1.72	3.25	0.80	96.34	96.51	96.14
Van Gaal Drain	Reach 3	3016	10-Year	2.12	3.78	0.92	96.40	96.54	96.17
Van Gaal Drain	Reach 3	3016	25-Year	2.72	4.36	1.07	96.47	96.56	96.21
Van Gaal Drain	Reach 3	2980	2-Year	1.17	2.58	0.90	96.18	96.41	96.12
Van Gaal Drain	Reach 3	2980	5-Year	1.72	3.25	0.80	96.29	96.48	96.09
Van Gaal Drain	Reach 3	2980	10-Year	2.12	3.78	0.92	96.35	96.51	96.12
Van Gaal Drain	Reach 3	2980	25-Year	2.72	4.36	1.07	96.43	96.54	96.16
Van Gaal Drain	Reach 3	2851	2-Year	1.17	2.58	0.90	95.83	96.17	95.77
Van Gaal Drain	Reach 3	2851	5-Year	1.72	3.25	0.80	95.97	96.27	95.73
Van Gaal Drain	Reach 3	2851	10-Year	2.12	3.78	0.92	96.06	96.32	95.77
Van Gaal Drain	Reach 3	2851	25-Year	2.72	4.36	1.07	96.17	96.37	95.82
Van Gaal Drain	Reach 3	2808	2-Year	1.17	2.58	0.90	95.78	96.13	95.72
Van Gaal Drain	Reach 3	2808	5-Year	1.72	3.25	0.80	95.93	96.24	95.69
Van Gaal Drain	Reach 3	2808	10-Year	2.12	3.78	0.92	96.03	96.29	95.73
Van Gaal Drain	Reach 3	2808	25-Year	2.72	4.36	1.07	96.14	96.34	95.78
Van Gaal Drain	Reach 3	2658	2-Year	1.17	2.58	0.90	95.67	96.05	95.63
Van Gaal Drain	Reach 3	2658	5-Year	1.72	3.25	0.80	95.83	96.14	95.59
Van Gaal Drain	Reach 3	2658	10-Year	2.12	3.78	0.92	95.93	96.19	95.64
Van Gaal Drain	Reach 3	2658	25-Year	2.72	4.36	1.07	96.04	96.24	95.70
Van Gaal Drain	Reach 2	2554	2-Year	1.72	4.13	1.55	95.66	96.03	95.63
Van Gaal Drain	Reach 2	2554	5-Year	2.61	5.22	1.36	95.82	96.13	95.58
Van Gaal Drain	Reach 2	2554	10-Year	3.25	5.93	1.57	95.92	96.18	95.63
Van Gaal Drain	Reach 2	2554	25-Year	4.04	6.83	1.84	96.02	96.23	95.69
Van Gaal Drain	Reach 2	2478	2-Year	1.72	4.13	1.55	95.58	95.93	95.55
Van Gaal Drain	Reach 2	2478	5-Year	2.61	5.22	1.36	95.74	96.01	95.51
Van Gaal Drain	Reach 2	2478	10-Year	3.25	5.93	1.57	95.83	96.06	95.55
Van Gaal Drain	Reach 2	2478	25-Year	4.04	6.83	1.84	95.92	96.09	95.61
Van Gaal Drain	Reach 2	2427.58*	2-Year	1.72	4.13	1.55	95.52	95.85	95.49
Van Gaal Drain	Reach 2	2427.58*	5-Year	2.61	5.22	1.36	95.67	95.94	95.45
Van Gaal Drain	Reach 2	2427.58*	10-Year	3.25	5.93	1.57	95.76	95.97	95.49
Van Gaal Drain	Reach 2	2427.58*	25-Year	4.04	6.83	1.84	95.85	96.00	95.55
Van Gaal Drain	Reach 2	2377.17*	2-Year	1.72	4.13	1.55	95.46	95.77	95.42
Van Gaal Drain	Reach 2	2377.17*	5-Year	2.61	5.22	1.36	95.60	95.85	95.38
Van Gaal Drain	Reach 2	2377.17*	10-Year	3.25	5.93	1.57	95.68	95.88	95.43
Van Gaal Drain	Reach 2	2377.17*	25-Year	4.04	6.83	1.84	95.77	95.92	95.48
Van Gaal Drain	Reach 2	2326.76*	2-Year	1.72	4.13	1.55	95.37	95.67	95.34
Van Gaal Drain	Reach 2	2326.76*	5-Year	2.61	5.22	1.36	95.50	95.76	95.30
Van Gaal Drain	Reach 2	2326.76*	10-Year	3.25	5.93	1.57	95.58	95.80	95.34
Van Gaal Drain	Reach 2	2326.76*	25-Year	4.04	6.83	1.84	95.66	95.85	95.39
Van Gaal Drain	Reach 2	2276.35*	2-Year	1.72	4.13	1.55	95.02	95.25	95.00
Van Gaal Drain	Reach 2	2276.35*	5-Year	2.61	5.22	1.36	95.12	95.33	94.97
Van Gaal Drain	Reach 2	2276.35*	10-Year	3.25	5.93	1.57	95.18	95.38	95.00
Van Gaal Drain	Reach 2	2276.35*	25-Year	4.04	6.83	1.84	95.25	95.45	95.04
Van Gaal Drain	Reach 2	2252	2-Year	1.72	4.13	1.55	94.93	95.08	94.80

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	2252	5-Year	2.61	5.22	1.36	95.07	95.17	94.77
Van Gaal Drain	Reach 2	2252	10-Year	3.25	5.93	1.57	95.16	95.23	94.80
Van Gaal Drain	Reach 2	2252	25-Year	4.04	6.83	1.84	95.26	95.30	94.84
Van Gaal Drain	Reach 2	2237	2-Year	1.72	4.13	1.55	94.91	95.06	94.78
Van Gaal Drain	Reach 2	2237	5-Year	2.61	5.22	1.36	95.05	95.15	94.75
Van Gaal Drain	Reach 2	2237	10-Year	3.25	5.93	1.57	95.14	95.21	94.78
Van Gaal Drain	Reach 2	2237	25-Year	4.04	6.83	1.84	95.24	95.28	94.81
Van Gaal Drain	Reach 2	2217	2-Year	1.72	4.13	1.55	94.89	95.03	94.75
Van Gaal Drain	Reach 2	2217	5-Year	2.61	5.22	1.36	95.03	95.12	94.72
Van Gaal Drain	Reach 2	2217	10-Year	3.25	5.93	1.57	95.11	95.18	94.75
Van Gaal Drain	Reach 2	2217	25-Year	4.04	6.83	1.84	95.22	95.25	94.78
Van Gaal Drain	Reach 2	2197	2-Year	1.72	4.13	1.55	94.86	95.00	94.71
Van Gaal Drain	Reach 2	2197	5-Year	2.61	5.22	1.36	95.00	95.09	94.69
Van Gaal Drain	Reach 2	2197	10-Year	3.25	5.93	1.57	95.09	95.15	94.72
Van Gaal Drain	Reach 2	2197	25-Year	4.04	6.83	1.84	95.19	95.22	94.75
Van Gaal Drain	Reach 2	2177	2-Year	1.72	4.13	1.55	94.84	94.97	94.68
Van Gaal Drain	Reach 2	2177	5-Year	2.61	5.22	1.36	94.98	95.06	94.65
Van Gaal Drain	Reach 2	2177	10-Year	3.25	5.93	1.57	95.06	95.12	94.68
Van Gaal Drain	Reach 2	2177	25-Year	4.04	6.83	1.84	95.17	95.19	94.72
Van Gaal Drain	Reach 2	2157	2-Year	1.72	4.13	1.55	94.81	94.94	94.64
Van Gaal Drain	Reach 2	2157	5-Year	2.61	5.22	1.36	94.95	95.03	94.61
Van Gaal Drain	Reach 2	2157	10-Year	3.25	5.93	1.57	95.04	95.09	94.65
Van Gaal Drain	Reach 2	2157	25-Year	4.04	6.83	1.84	95.14	95.16	94.68
Van Gaal Drain	Reach 2	2154	2-Year	1.72	4.13	1.55	94.79	94.91	94.61
Van Gaal Drain	Reach 2	2154	5-Year	2.61	5.22	1.36	94.93	95.00	94.58
Van Gaal Drain	Reach 2	2154	10-Year	3.25	5.93	1.57	95.01	95.06	94.61
Van Gaal Drain	Reach 2	2154	25-Year	4.04	6.83	1.84	95.12	95.13	94.65
Van Gaal Drain	Reach 2	2153	2-Year	2.50	4.99	1.78	94.75	94.86	94.57
Van Gaal Drain	Reach 2	2153	5-Year	3.63	6.30	1.50	94.89	94.96	94.54
Van Gaal Drain	Reach 2	2153	10-Year	4.44	7.15	1.78	94.97	95.01	94.57
Van Gaal Drain	Reach 2	2153	25-Year	5.53	8.24	2.09	95.08	95.08	94.61
Van Gaal Drain	Reach 2	2152	2-Year	2.50	4.99	1.78	94.71	94.82	94.53
Van Gaal Drain	Reach 2	2152	5-Year	3.63	6.30	1.50	94.85	94.92	94.49
Van Gaal Drain	Reach 2	2152	10-Year	4.44	7.15	1.78	94.93	94.97	94.53
Van Gaal Drain	Reach 2	2152	25-Year	5.53	8.24	2.09	95.03	95.04	94.56
Van Gaal Drain	Reach 2	2132	2-Year	2.50	4.99	1.78	94.67	94.78	94.49
Van Gaal Drain	Reach 2	2132	5-Year	3.63	6.30	1.50	94.80	94.87	94.45
Van Gaal Drain	Reach 2	2132	10-Year	4.44	7.15	1.78	94.89	94.93	94.49
Van Gaal Drain	Reach 2	2132	25-Year	5.53	8.24	2.09	94.99	95.00	94.52
Van Gaal Drain	Reach 2	2112	2-Year	2.50	4.99	1.78	94.63	94.74	94.44
Van Gaal Drain	Reach 2	2112	5-Year	3.63	6.30	1.50	94.76	94.83	94.41
Van Gaal Drain	Reach 2	2112	10-Year	4.44	7.15	1.78	94.85	94.89	94.44
Van Gaal Drain	Reach 2	2112	25-Year	5.53	8.24	2.09	94.95	94.96	94.48
Van Gaal Drain	Reach 2	2092	2-Year	2.50	4.99	1.78	94.59	94.70	94.40

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	2092	5-Year	3.63	6.30	1.50	94.72	94.79	94.37
Van Gaal Drain	Reach 2	2092	10-Year	4.44	7.15	1.78	94.81	94.85	94.40
Van Gaal Drain	Reach 2	2092	25-Year	5.53	8.24	2.09	94.91	94.92	94.44
Van Gaal Drain	Reach 2	2072	2-Year	2.50	4.99	1.78	94.55	94.66	94.36
Van Gaal Drain	Reach 2	2072	5-Year	3.63	6.30	1.50	94.68	94.75	94.33
Van Gaal Drain	Reach 2	2072	10-Year	4.44	7.15	1.78	94.77	94.81	94.36
Van Gaal Drain	Reach 2	2072	25-Year	5.53	8.24	2.09	94.87	94.88	94.40
Van Gaal Drain	Reach 2	2052	2-Year	2.50	4.99	1.78	94.50	94.62	94.32
Van Gaal Drain	Reach 2	2052	5-Year	3.63	6.30	1.50	94.64	94.71	94.28
Van Gaal Drain	Reach 2	2052	10-Year	4.44	7.15	1.78	94.72	94.77	94.32
Van Gaal Drain	Reach 2	2052	25-Year	5.53	8.24	2.09	94.83	94.84	94.35
Van Gaal Drain	Reach 2	2032	2-Year	2.50	4.99	1.78	94.46	94.57	94.28
Van Gaal Drain	Reach 2	2032	5-Year	3.63	6.30	1.50	94.60	94.67	94.24
Van Gaal Drain	Reach 2	2032	10-Year	4.44	7.15	1.78	94.68	94.73	94.28
Van Gaal Drain	Reach 2	2032	25-Year	5.53	8.24	2.09	94.79	94.80	94.31
Van Gaal Drain	Reach 2	2002	2-Year	2.50	4.99	1.78	94.40	94.51	94.21
Van Gaal Drain	Reach 2	2002	5-Year	3.63	6.30	1.50	94.54	94.61	94.18
Van Gaal Drain	Reach 2	2002	10-Year	4.44	7.15	1.78	94.62	94.67	94.21
Van Gaal Drain	Reach 2	2002	25-Year	5.53	8.24	2.09	94.73	94.74	94.25
Van Gaal Drain	Reach 2	1982	2-Year	2.50	4.99	1.78	94.36	94.48	94.17
Van Gaal Drain	Reach 2	1982	5-Year	3.63	6.30	1.50	94.50	94.57	94.14
Van Gaal Drain	Reach 2	1982	10-Year	4.44	7.15	1.78	94.58	94.63	94.17
Van Gaal Drain	Reach 2	1982	25-Year	5.53	8.24	2.09	94.69	94.70	94.21
Van Gaal Drain	Reach 2	1962	2-Year	2.50	4.99	1.78	94.32	94.44	94.13
Van Gaal Drain	Reach 2	1962	5-Year	3.63	6.30	1.50	94.46	94.53	94.09
Van Gaal Drain	Reach 2	1962	10-Year	4.44	7.15	1.78	94.54	94.59	94.13
Van Gaal Drain	Reach 2	1962	25-Year	5.53	8.24	2.09	94.65	94.66	94.17
Van Gaal Drain	Reach 2	1942	2-Year	2.50	4.99	1.78	94.28	94.40	94.09
Van Gaal Drain	Reach 2	1942	5-Year	3.63	6.30	1.50	94.42	94.50	94.05
Van Gaal Drain	Reach 2	1942	10-Year	4.44	7.15	1.78	94.51	94.56	94.09
Van Gaal Drain	Reach 2	1942	25-Year	5.53	8.24	2.09	94.61	94.63	94.13
Van Gaal Drain	Reach 2	1922	2-Year	2.50	4.99	1.78	94.24	94.36	94.05
Van Gaal Drain	Reach 2	1922	5-Year	3.63	6.30	1.50	94.38	94.46	94.01
Van Gaal Drain	Reach 2	1922	10-Year	4.44	7.15	1.78	94.47	94.52	94.05
Van Gaal Drain	Reach 2	1922	25-Year	5.53	8.24	2.09	94.57	94.59	94.09
Van Gaal Drain	Reach 2	1902	2-Year	2.50	4.99	1.78	94.20	94.33	94.00
Van Gaal Drain	Reach 2	1902	5-Year	3.63	6.30	1.50	94.34	94.43	93.97
Van Gaal Drain	Reach 2	1902	10-Year	4.44	7.15	1.78	94.43	94.48	94.00
Van Gaal Drain	Reach 2	1902	25-Year	5.53	8.24	2.09	94.54	94.56	94.05
Van Gaal Drain	Reach 2	1882	2-Year	2.50	4.99	1.78	94.16	94.29	93.96
Van Gaal Drain	Reach 2	1882	5-Year	3.63	6.30	1.50	94.30	94.39	93.93
Van Gaal Drain	Reach 2	1882	10-Year	4.44	7.15	1.78	94.39	94.45	93.96
Van Gaal Drain	Reach 2	1882	25-Year	5.53	8.24	2.09	94.50	94.52	94.01
Van Gaal Drain	Reach 2	1862	2-Year	2.50	4.99	1.78	94.12	94.26	93.92

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	1862	5-Year	3.63	6.30	1.50	94.27	94.36	93.89
Van Gaal Drain	Reach 2	1862	10-Year	4.44	7.15	1.78	94.36	94.42	93.92
Van Gaal Drain	Reach 2	1862	25-Year	5.53	8.24	2.09	94.46	94.49	93.98
Van Gaal Drain	Reach 2	1842	2-Year	2.50	4.99	1.78	94.09	94.23	93.88
Van Gaal Drain	Reach 2	1842	5-Year	3.63	6.30	1.50	94.23	94.33	93.85
Van Gaal Drain	Reach 2	1842	10-Year	4.44	7.15	1.78	94.32	94.39	93.88
Van Gaal Drain	Reach 2	1842	25-Year	5.53	8.24	2.09	94.43	94.46	93.94
Van Gaal Drain	Reach 2	1822	2-Year	2.50	4.99	1.78	94.05	94.20	93.84
Van Gaal Drain	Reach 2	1822	5-Year	3.63	6.30	1.50	94.20	94.30	93.80
Van Gaal Drain	Reach 2	1822	10-Year	4.44	7.15	1.78	94.29	94.36	93.85
Van Gaal Drain	Reach 2	1822	25-Year	5.53	8.24	2.09	94.40	94.43	93.91
Van Gaal Drain	Reach 2	1802	2-Year	2.50	4.99	1.78	94.02	94.17	93.81
Van Gaal Drain	Reach 2	1802	5-Year	3.63	6.30	1.50	94.17	94.27	93.77
Van Gaal Drain	Reach 2	1802	10-Year	4.44	7.15	1.78	94.26	94.33	93.81
Van Gaal Drain	Reach 2	1802	25-Year	5.53	8.24	2.09	94.36	94.40	93.88
Van Gaal Drain	Reach 2	1782	2-Year	2.50	4.99	1.78	93.99	94.14	93.77
Van Gaal Drain	Reach 2	1782	5-Year	3.63	6.30	1.50	94.14	94.24	93.73
Van Gaal Drain	Reach 2	1782	10-Year	4.44	7.15	1.78	94.23	94.30	93.77
Van Gaal Drain	Reach 2	1782	25-Year	5.53	8.24	2.09	94.33	94.37	93.86
Van Gaal Drain	Reach 2	1762	2-Year	2.50	4.99	1.78	93.96	94.12	93.74
Van Gaal Drain	Reach 2	1762	5-Year	3.63	6.30	1.50	94.11	94.22	93.69
Van Gaal Drain	Reach 2	1762	10-Year	4.44	7.15	1.78	94.20	94.27	93.74
Van Gaal Drain	Reach 2	1762	25-Year	5.53	8.24	2.09	94.30	94.34	93.84
Van Gaal Drain	Reach 2	1742	2-Year	2.50	4.99	1.78	93.93	94.10	93.70
Van Gaal Drain	Reach 2	1742	5-Year	3.63	6.30	1.50	94.08	94.19	93.66
Van Gaal Drain	Reach 2	1742	10-Year	4.44	7.15	1.78	94.17	94.25	93.71
Van Gaal Drain	Reach 2	1742	25-Year	5.53	8.24	2.09	94.27	94.32	93.82
Van Gaal Drain	Reach 2	1722	2-Year	2.50	4.99	1.78	93.91	94.08	93.68
Van Gaal Drain	Reach 2	1722	5-Year	3.63	6.30	1.50	94.06	94.17	93.63
Van Gaal Drain	Reach 2	1722	10-Year	4.44	7.15	1.78	94.15	94.23	93.69
Van Gaal Drain	Reach 2	1722	25-Year	5.53	8.24	2.09	94.25	94.30	93.81
Van Gaal Drain	Reach 2	1702	2-Year	2.50	4.99	1.78	93.89	94.06	93.65
Van Gaal Drain	Reach 2	1702	5-Year	3.63	6.30	1.50	94.04	94.15	93.60
Van Gaal Drain	Reach 2	1702	10-Year	4.44	7.15	1.78	94.12	94.21	93.66
Van Gaal Drain	Reach 2	1702	25-Year	5.53	8.24	2.09	94.22	94.28	93.79
Van Gaal Drain	Reach 2	1682	2-Year	2.50	4.99	1.78	93.87	94.05	93.63
Van Gaal Drain	Reach 2	1682	5-Year	3.63	6.30	1.50	94.02	94.14	93.58
Van Gaal Drain	Reach 2	1682	10-Year	4.44	7.15	1.78	94.10	94.19	93.64
Van Gaal Drain	Reach 2	1682	25-Year	5.53	8.24	2.09	94.20	94.26	93.78
Van Gaal Drain	Reach 2	1662	2-Year	2.50	4.99	1.78	93.85	94.03	93.61
Van Gaal Drain	Reach 2	1662	5-Year	3.63	6.30	1.50	94.00	94.12	93.56
Van Gaal Drain	Reach 2	1662	10-Year	4.44	7.15	1.78	94.08	94.17	93.63
Van Gaal Drain	Reach 2	1662	25-Year	5.53	8.24	2.09	94.17	94.24	93.78
Van Gaal Drain	Reach 2	1642	2-Year	2.50	4.99	1.78	93.83	94.02	93.60

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	1642	5-Year	3.63	6.30	1.50	93.98	94.11	93.54
Van Gaal Drain	Reach 2	1642	10-Year	4.44	7.15	1.78	94.06	94.16	93.62
Van Gaal Drain	Reach 2	1642	25-Year	5.53	8.24	2.09	94.15	94.22	93.77
Van Gaal Drain	Reach 2	1622	2-Year	2.50	4.99	1.78	93.82	94.01	93.59
Van Gaal Drain	Reach 2	1622	5-Year	3.63	6.30	1.50	93.96	94.09	93.53
Van Gaal Drain	Reach 2	1622	10-Year	4.44	7.15	1.78	94.04	94.14	93.61
Van Gaal Drain	Reach 2	1622	25-Year	5.53	8.24	2.09	94.13	94.21	93.76
Van Gaal Drain	Reach 2	1615	2-Year	2.50	4.99	1.78	93.81	94.01	93.59
Van Gaal Drain	Reach 2	1615	5-Year	3.63	6.30	1.50	93.96	94.09	93.53
Van Gaal Drain	Reach 2	1615	10-Year	4.44	7.15	1.78	94.04	94.14	93.60
Van Gaal Drain	Reach 2	1615	25-Year	5.53	8.24	2.09	94.13	94.20	93.76
Van Gaal Drain	Reach 2	1555	2-Year	2.50	4.99	1.78	93.78	93.98	93.56
Van Gaal Drain	Reach 2	1555	5-Year	3.63	6.30	1.50	93.92	94.06	93.50
Van Gaal Drain	Reach 2	1555	10-Year	4.44	7.15	1.78	94.00	94.11	93.58
Van Gaal Drain	Reach 2	1555	25-Year	5.53	8.24	2.09	94.08	94.17	93.75
Van Gaal Drain	Reach 2	1488	2-Year	2.50	4.99	1.78	93.76	93.96	93.55
Van Gaal Drain	Reach 2	1488	5-Year	3.63	6.30	1.50	93.89	94.03	93.49
Van Gaal Drain	Reach 2	1488	10-Year	4.44	7.15	1.78	93.96	94.08	93.57
Van Gaal Drain	Reach 2	1488	25-Year	5.53	8.24	2.09	94.04	94.13	93.74
Van Gaal Drain	Reach 2	1416	2-Year	2.50	4.99	1.78	93.75	93.95	93.54
Van Gaal Drain	Reach 2	1416	5-Year	3.63	6.30	1.50	93.87	94.02	93.48
Van Gaal Drain	Reach 2	1416	10-Year	4.44	7.15	1.78	93.94	94.06	93.56
Van Gaal Drain	Reach 2	1416	25-Year	5.53	8.24	2.09	94.01	94.11	93.74
Van Gaal Drain	Reach 2	1400	2-Year	2.50	4.99	1.78	93.74	93.94	93.54
Van Gaal Drain	Reach 2	1400	5-Year	3.63	6.30	1.50	93.87	94.01	93.48
Van Gaal Drain	Reach 2	1400	10-Year	4.44	7.15	1.78	93.93	94.05	93.56
Van Gaal Drain	Reach 2	1400	25-Year	5.53	8.24	2.09	94.00	94.10	93.73
Van Gaal Drain	Reach 2	1364	2-Year	2.50	4.99	1.78	93.73	93.94	93.54
Van Gaal Drain	Reach 2	1364	5-Year	3.63	6.30	1.50	93.86	94.00	93.48
Van Gaal Drain	Reach 2	1364	10-Year	4.44	7.15	1.78	93.92	94.04	93.56
Van Gaal Drain	Reach 2	1364	25-Year	5.53	8.24	2.09	93.98	94.09	93.73
Van Gaal Drain	Reach 2	1340	2-Year	2.55	5.05	1.84	93.72	93.90	93.53
Van Gaal Drain	Reach 2	1340	5-Year	3.69	6.34	1.58	93.83	93.95	93.47
Van Gaal Drain	Reach 2	1340	10-Year	4.50	7.21	1.86	93.88	93.98	93.55
Van Gaal Drain	Reach 2	1340	25-Year	5.57	8.32	2.19	93.94	94.01	93.72
Van Gaal Drain	Reach 2	1339	Culvert						
Van Gaal Drain	Reach 2	1312	2-Year	2.55	5.05	1.84	93.72	93.90	93.52
Van Gaal Drain	Reach 2	1312	5-Year	3.69	6.34	1.58	93.83	93.94	93.47
Van Gaal Drain	Reach 2	1312	10-Year	4.50	7.21	1.86	93.88	93.97	93.55
Van Gaal Drain	Reach 2	1312	25-Year	5.57	8.32	2.19	93.93	94.00	93.72
Van Gaal Drain	Reach 2	1302	2-Year	2.55	5.05	1.84	93.69	93.87	93.48
Van Gaal Drain	Reach 2	1302	5-Year	3.69	6.34	1.58	93.80	93.92	93.42
Van Gaal Drain	Reach 2	1302	10-Year	4.50	7.21	1.86	93.85	93.95	93.51
Van Gaal Drain	Reach 2	1302	25-Year	5.57	8.32	2.19	93.90	93.99	93.70

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Van Gaal Drain	Reach 2	1268	2-Year	2.55	5.05	1.84	93.60	93.82	93.40
Van Gaal Drain	Reach 2	1268	5-Year	3.69	6.34	1.58	93.74	93.88	93.34
Van Gaal Drain	Reach 2	1268	10-Year	4.50	7.21	1.86	93.80	93.91	93.43
Van Gaal Drain	Reach 2	1268	25-Year	5.57	8.32	2.19	93.86	93.95	93.63
Van Gaal Drain	Reach 2	1212	2-Year	2.55	5.05	1.84	93.42	93.72	93.26
Van Gaal Drain	Reach 2	1212	5-Year	3.69	6.34	1.58	93.59	93.79	93.20
Van Gaal Drain	Reach 2	1212	10-Year	4.50	7.21	1.86	93.67	93.83	93.32
Van Gaal Drain	Reach 2	1212	25-Year	5.57	8.32	2.19	93.75	93.87	93.53
Van Gaal Drain	Reach 2	1169	2-Year	2.55	5.05	1.84	93.33	93.64	93.17
Van Gaal Drain	Reach 2	1169	5-Year	3.69	6.34	1.58	93.51	93.71	93.13
Van Gaal Drain	Reach 2	1169	10-Year	4.50	7.21	1.86	93.59	93.75	93.26
Van Gaal Drain	Reach 2	1169	25-Year	5.57	8.32	2.19	93.68	93.79	93.50
Van Gaal Drain	Reach 2	1091	2-Year	2.55	5.05	1.84	93.19	93.51	93.04
Van Gaal Drain	Reach 2	1091	5-Year	3.69	6.34	1.58	93.37	93.58	93.01
Van Gaal Drain	Reach 2	1091	10-Year	4.50	7.21	1.86	93.46	93.63	93.17
Van Gaal Drain	Reach 2	1091	25-Year	5.57	8.32	2.19	93.55	93.67	93.46
Van Gaal Drain	Reach 2	1002	2-Year	2.55	5.05	1.84	93.00	93.29	92.86
Van Gaal Drain	Reach 2	1002	5-Year	3.69	6.34	1.58	93.18	93.38	92.87
Van Gaal Drain	Reach 2	1002	10-Year	4.50	7.21	1.86	93.26	93.43	93.09
Van Gaal Drain	Reach 2	1002	25-Year	5.57	8.32	2.19	93.35	93.53	93.44
Van Gaal Drain	Reach 2	961	2-Year	2.55	5.05	1.84	92.94	93.19	92.80
Van Gaal Drain	Reach 2	961	5-Year	3.69	6.34	1.58	93.10	93.27	92.84
Van Gaal Drain	Reach 2	961	10-Year	4.50	7.21	1.86	93.18	93.33	93.07
Van Gaal Drain	Reach 2	961	25-Year	5.57	8.32	2.19	93.26	93.46	93.43
Van Gaal Drain	Reach 2	910	2-Year	2.55	5.05	1.84	92.87	93.11	92.74
Van Gaal Drain	Reach 2	910	5-Year	3.69	6.34	1.58	93.02	93.19	92.80
Van Gaal Drain	Reach 2	910	10-Year	4.50	7.21	1.86	93.09	93.27	93.05
Van Gaal Drain	Reach 2	910	25-Year	5.57	8.32	2.19	93.19	93.43	93.43
Van Gaal Drain	Reach 2	840	2-Year	2.55	5.05	1.84	92.76	93.01	92.63
Van Gaal Drain	Reach 2	840	5-Year	3.69	6.34	1.58	92.92	93.11	92.75
Van Gaal Drain	Reach 2	840	10-Year	4.50	7.21	1.86	93.01	93.22	93.04
Van Gaal Drain	Reach 2	840	25-Year	5.57	8.32	2.19	93.14	93.41	93.43
Van Gaal Drain	Reach 1	746	2-Year	3.34	6.19	2.36	92.74	92.99	92.62
Van Gaal Drain	Reach 1	746	5-Year	4.75	8.10	2.11	92.88	93.10	92.74
Van Gaal Drain	Reach 1	746	10-Year	5.74	9.87	2.41	92.97	93.21	93.04
Van Gaal Drain	Reach 1	746	25-Year	7.82	11.49	2.79	93.11	93.41	93.43
Van Gaal Drain	Reach 1	705	2-Year	3.34	6.19	2.36	92.64	92.90	92.53
Van Gaal Drain	Reach 1	705	5-Year	4.75	8.10	2.11	92.78	93.05	92.72
Van Gaal Drain	Reach 1	705	10-Year	5.74	9.87	2.41	92.87	93.18	93.03
Van Gaal Drain	Reach 1	705	25-Year	7.82	11.49	2.79	93.04	93.39	93.43
Van Gaal Drain	Reach 1	668	2-Year	3.34	6.19	2.36	92.40	92.52	92.39
Van Gaal Drain	Reach 1	668	5-Year	4.75	8.10	2.11	92.50	92.71	92.69
Van Gaal Drain	Reach 1	668	10-Year	5.74	9.87	2.41	92.50	93.01	93.02
Van Gaal Drain	Reach 1	668	25-Year	7.82	11.49	2.79	92.60	93.32	93.43

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	2	4	1	2	4
Van Gaal Drain	Reach 1	666	2-Year	3.34	6.19	2.36	92.32	92.50	92.38
Van Gaal Drain	Reach 1	666	5-Year	4.75	8.10	2.11	92.41	92.71	92.69
Van Gaal Drain	Reach 1	666	10-Year	5.74	9.87	2.41	92.47	92.90	93.02
Van Gaal Drain	Reach 1	666	25-Year	7.82	11.49	2.79	92.59	93.18	93.42
Van Gaal Drain	Reach 1	656	Culvert						
Van Gaal Drain	Reach 1	647	2-Year	3.34	6.19	2.36	92.43	92.61	92.41
Van Gaal Drain	Reach 1	647	5-Year	4.75	8.10	2.11	92.53	92.79	92.69
Van Gaal Drain	Reach 1	647	10-Year	5.74	9.87	2.41	92.60	92.95	93.02
Van Gaal Drain	Reach 1	647	25-Year	7.82	11.49	2.79	92.71	93.19	93.42
Van Gaal Drain	Reach 1	645	2-Year	3.34	6.19	2.36	92.40	92.58	92.39
Van Gaal Drain	Reach 1	645	5-Year	4.75	8.10	2.11	92.49	92.79	92.69
Van Gaal Drain	Reach 1	645	10-Year	5.74	9.87	2.41	92.56	92.96	93.02
Van Gaal Drain	Reach 1	645	25-Year	7.82	11.49	2.79	92.68	93.20	93.42
Van Gaal Drain	Reach 1	592	2-Year	3.34	6.19	2.36	92.16	92.57	92.38
Van Gaal Drain	Reach 1	592	5-Year	4.75	8.10	2.11	92.35	92.81	92.69
Van Gaal Drain	Reach 1	592	10-Year	5.74	9.87	2.41	92.46	92.98	93.02
Van Gaal Drain	Reach 1	592	25-Year	7.82	11.49	2.79	92.62	93.23	93.42
Van Gaal Drain	Reach 1	521	2-Year	3.34	6.19	2.36	92.17	92.56	92.38
Van Gaal Drain	Reach 1	521	5-Year	4.75	8.10	2.11	92.35	92.80	92.69
Van Gaal Drain	Reach 1	521	10-Year	5.74	9.87	2.41	92.45	92.97	93.02
Van Gaal Drain	Reach 1	521	25-Year	7.82	11.49	2.79	92.61	93.22	93.42
Van Gaal Drain	Reach 1	277	2-Year	3.34	6.19	2.36	92.03	92.45	92.35
Van Gaal Drain	Reach 1	277	5-Year	4.75	8.10	2.11	92.19	92.69	92.68
Van Gaal Drain	Reach 1	277	10-Year	5.74	9.87	2.41	92.28	92.85	93.02
Van Gaal Drain	Reach 1	277	25-Year	7.82	11.49	2.79	92.38	93.14	93.42
Van Gaal Drain	Reach 1	275	2-Year	3.34	6.19	2.36	92.03	92.43	92.35
Van Gaal Drain	Reach 1	275	5-Year	4.75	8.10	2.11	92.19	92.66	92.68
Van Gaal Drain	Reach 1	275	10-Year	5.74	9.87	2.41	92.28	92.81	93.01
Van Gaal Drain	Reach 1	275	25-Year	7.82	11.49	2.79	92.38	93.07	93.42
Van Gaal Drain	Reach 1	269	Culvert						
Van Gaal Drain	Reach 1	263	2-Year	3.34	6.19	2.36	92.01	92.40	92.34
Van Gaal Drain	Reach 1	263	5-Year	4.75	8.10	2.11	92.15	92.61	92.68
Van Gaal Drain	Reach 1	263	10-Year	5.74	9.87	2.41	92.23	92.74	93.01
Van Gaal Drain	Reach 1	263	25-Year	7.82	11.49	2.79	92.30	93.00	93.41
Van Gaal Drain	Reach 1	226	2-Year	3.35	6.22	2.49	91.96	92.37	92.34
Van Gaal Drain	Reach 1	226	5-Year	4.77	8.11	2.26	92.09	92.61	92.68
Van Gaal Drain	Reach 1	226	10-Year	5.76	9.90	2.57	92.17	92.75	93.01
Van Gaal Drain	Reach 1	226	25-Year	7.83	11.53	2.97	92.18	93.03	93.41
Van Gaal Drain	Reach 1	0	2-Year	3.35	6.22	2.49	91.06	92.40	92.35
Van Gaal Drain	Reach 1	0	5-Year	4.77	8.11	2.26	91.08	92.63	92.68
Van Gaal Drain	Reach 1	0	10-Year	5.76	9.90	2.57	91.09	92.78	93.01
Van Gaal Drain	Reach 1	0	25-Year	7.83	11.53	2.97	91.16	93.04	93.41

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	2	4	1	2	4
Moore Drain Trib	Reach 1	600	2-Year	0.45	0.74	0.13	95.36	95.50	95.16
Moore Drain Trib	Reach 1	600	5-Year	0.63	0.90	0.08	95.45	95.56	95.11
Moore Drain Trib	Reach 1	600	10-Year	0.77	1.14	0.09	95.51	95.65	95.12
Moore Drain Trib	Reach 1	600	25-Year	1.02	1.33	0.10	95.61	95.72	95.13
Moore Drain Trib	Reach 1	553	2-Year	0.45	0.74	0.13	95.32	95.47	95.11
Moore Drain Trib	Reach 1	553	5-Year	0.63	0.90	0.08	95.42	95.53	95.06
Moore Drain Trib	Reach 1	553	10-Year	0.77	1.14	0.09	95.48	95.63	95.07
Moore Drain Trib	Reach 1	553	25-Year	1.02	1.33	0.10	95.58	95.69	95.09
Moore Drain Trib	Reach 1	503	2-Year	0.45	0.74	0.13	95.29	95.44	95.08
Moore Drain Trib	Reach 1	503	5-Year	0.63	0.90	0.08	95.39	95.51	95.02
Moore Drain Trib	Reach 1	503	10-Year	0.77	1.14	0.09	95.45	95.60	95.04
Moore Drain Trib	Reach 1	503	25-Year	1.02	1.33	0.10	95.56	95.67	95.05
Moore Drain Trib	Reach 1	492	2-Year	0.45	0.74	0.13	95.27	95.41	95.06
Moore Drain Trib	Reach 1	492	5-Year	0.63	0.90	0.08	95.36	95.47	95.01
Moore Drain Trib	Reach 1	492	10-Year	0.77	1.14	0.09	95.42	95.56	95.02
Moore Drain Trib	Reach 1	492	25-Year	1.02	1.33	0.10	95.51	95.62	95.04
Moore Drain Trib	Reach 1	477	Culvert						
Moore Drain Trib	Reach 1	462	2-Year	0.45	0.74	0.13	95.19	95.31	95.00
Moore Drain Trib	Reach 1	462	5-Year	0.63	0.90	0.08	95.27	95.37	94.95
Moore Drain Trib	Reach 1	462	10-Year	0.77	1.14	0.09	95.32	95.45	94.96
Moore Drain Trib	Reach 1	462	25-Year	1.02	1.33	0.10	95.41	95.51	94.97
Moore Drain Trib	Reach 1	453	2-Year	0.45	0.74	0.13	95.18	95.32	94.99
Moore Drain Trib	Reach 1	453	5-Year	0.63	0.90	0.08	95.27	95.38	94.94
Moore Drain Trib	Reach 1	453	10-Year	0.77	1.14	0.09	95.33	95.46	94.95
Moore Drain Trib	Reach 1	453	25-Year	1.02	1.33	0.10	95.42	95.52	94.96
Moore Drain Trib	Reach 1	403	2-Year	0.45	0.74	0.13	95.15	95.28	94.95
Moore Drain Trib	Reach 1	403	5-Year	0.63	0.90	0.08	95.23	95.34	94.90
Moore Drain Trib	Reach 1	403	10-Year	0.77	1.14	0.09	95.29	95.43	94.91
Moore Drain Trib	Reach 1	403	25-Year	1.02	1.33	0.10	95.39	95.49	94.92
Moore Drain Trib	Reach 1	353	2-Year	0.45	0.74	0.13	95.07	95.19	94.89
Moore Drain Trib	Reach 1	353	5-Year	0.63	0.90	0.08	95.15	95.25	94.84
Moore Drain Trib	Reach 1	353	10-Year	0.77	1.14	0.09	95.20	95.33	94.85
Moore Drain Trib	Reach 1	353	25-Year	1.02	1.33	0.10	95.29	95.39	94.87
Moore Drain Trib	Reach 1	338.5	Culvert						
Moore Drain Trib	Reach 1	324	2-Year	0.45	0.74	0.13	94.95	95.04	94.81
Moore Drain Trib	Reach 1	324	5-Year	0.63	0.90	0.08	95.01	95.08	94.77
Moore Drain Trib	Reach 1	324	10-Year	0.77	1.14	0.09	95.05	95.14	94.78
Moore Drain Trib	Reach 1	324	25-Year	1.02	1.33	0.10	95.11	95.19	94.79
Moore Drain Trib	Reach 1	311	2-Year	0.45	0.74	0.13	94.92	95.02	94.78
Moore Drain Trib	Reach 1	311	5-Year	0.63	0.90	0.08	94.99	95.07	94.74
Moore Drain Trib	Reach 1	311	10-Year	0.77	1.14	0.09	95.03	95.14	94.75
Moore Drain Trib	Reach 1	311	25-Year	1.02	1.33	0.10	95.11	95.19	94.76
Moore Drain Trib	Reach 1	290	2-Year	0.45	0.74	0.13	94.88	94.98	94.73

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Moore Drain Trib	Reach 1	290	5-Year	0.63	0.90	0.08	94.95	95.03	94.70
Moore Drain Trib	Reach 1	290	10-Year	0.77	1.14	0.09	94.99	95.10	94.71
Moore Drain Trib	Reach 1	290	25-Year	1.02	1.33	0.10	95.07	95.15	94.72
Moore Drain Trib	Reach 1	240	2-Year	0.45	0.74	0.13	94.78	94.89	94.63
Moore Drain Trib	Reach 1	240	5-Year	0.63	0.90	0.08	94.85	94.94	94.60
Moore Drain Trib	Reach 1	240	10-Year	0.77	1.14	0.09	94.90	95.02	94.60
Moore Drain Trib	Reach 1	240	25-Year	1.02	1.33	0.10	94.98	95.07	94.61
Moore Drain Trib	Reach 1	190	2-Year	0.45	0.74	0.13	94.69	94.81	94.53
Moore Drain Trib	Reach 1	190	5-Year	0.63	0.90	0.08	94.77	94.87	94.50
Moore Drain Trib	Reach 1	190	10-Year	0.77	1.14	0.09	94.82	94.95	94.51
Moore Drain Trib	Reach 1	190	25-Year	1.02	1.33	0.10	94.91	95.00	94.52
Moore Drain Trib	Reach 1	140	2-Year	0.45	0.74	0.13	94.62	94.75	94.44
Moore Drain Trib	Reach 1	140	5-Year	0.63	0.90	0.08	94.70	94.81	94.39
Moore Drain Trib	Reach 1	140	10-Year	0.77	1.14	0.09	94.76	94.89	94.40
Moore Drain Trib	Reach 1	140	25-Year	1.02	1.33	0.10	94.85	94.95	94.41
Moore Drain Trib	Reach 1	90	2-Year	0.45	0.74	0.13	94.58	94.71	94.38
Moore Drain Trib	Reach 1	90	5-Year	0.63	0.90	0.08	94.66	94.77	94.33
Moore Drain Trib	Reach 1	90	10-Year	0.77	1.14	0.09	94.72	94.86	94.34
Moore Drain Trib	Reach 1	90	25-Year	1.02	1.33	0.10	94.81	94.92	94.36
Moore Drain Trib	Reach 1	83	2-Year	0.45	0.74	0.13	94.55	94.67	94.36
Moore Drain Trib	Reach 1	83	5-Year	0.63	0.90	0.08	94.63	94.72	94.32
Moore Drain Trib	Reach 1	83	10-Year	0.77	1.14	0.09	94.68	94.80	94.33
Moore Drain Trib	Reach 1	83	25-Year	1.02	1.33	0.10	94.76	94.86	94.34
Moore Drain Trib	Reach 1	68	Culvert						
Moore Drain Trib	Reach 1	53	2-Year	0.45	0.74	0.13	94.43	94.51	94.29
Moore Drain Trib	Reach 1	53	5-Year	0.63	0.90	0.08	94.49	94.55	94.25
Moore Drain Trib	Reach 1	53	10-Year	0.77	1.14	0.09	94.52	94.60	94.25
Moore Drain Trib	Reach 1	53	25-Year	1.02	1.33	0.10	94.58	94.64	94.26
Moore Drain Trib	Reach 1	14	2-Year	0.45	0.74	0.13	94.08	94.13	94.00
Moore Drain Trib	Reach 1	14	5-Year	0.63	0.90	0.08	94.11	94.15	93.99
Moore Drain Trib	Reach 1	14	10-Year	0.77	1.14	0.09	94.13	94.18	93.99
Moore Drain Trib	Reach 1	14	25-Year	1.02	1.33	0.10	94.17	94.21	93.99
Moore Drain	Reach 2	555	2-Year	0.03	0.04	0.00	94.32	94.34	94.21
Moore Drain	Reach 2	555	5-Year	0.03	0.03	0.00	94.32	94.33	94.21
Moore Drain	Reach 2	555	10-Year	0.03	0.07	0.00	94.33	94.38	94.21
Moore Drain	Reach 2	555	25-Year	0.05	0.09	0.00	94.35	94.39	94.21
Moore Drain	Reach 2	500	2-Year	0.03	0.04	0.00	94.22	94.23	94.17
Moore Drain	Reach 2	500	5-Year	0.03	0.03	0.00	94.23	94.23	94.17
Moore Drain	Reach 2	500	10-Year	0.03	0.07	0.00	94.23	94.26	94.17
Moore Drain	Reach 2	500	25-Year	0.05	0.09	0.00	94.25	94.27	94.17
Moore Drain	Reach 1	298	2-Year	0.48	0.79	0.15	93.73	93.74	93.54
Moore Drain	Reach 1	298	5-Year	0.68	0.95	0.09	93.71	93.77	93.49
Moore Drain	Reach 1	298	10-Year	0.82	1.22	0.10	93.73	93.79	93.50
Moore Drain	Reach 1	298	25-Year	1.08	1.42	0.11	93.79	93.80	93.51

Table 8B: Proposed Conditions Flows and Water Levels on the Van Gaal and Arbuckle Drains (1)

River	Reach	River Station	Profile	Flow (m^3/s)			Maximum Water Level (m)		
				1	*2*	*4*	*1*	*2*	*4*
Moore Drain	Reach 1	130	2-Year	0.48	0.79	0.15	92.75	93.01	92.64
Moore Drain	Reach 1	130	5-Year	0.68	0.95	0.09	92.92	93.12	92.76
Moore Drain	Reach 1	130	10-Year	0.82	1.22	0.10	93.01	93.23	93.04
Moore Drain	Reach 1	130	25-Year	1.08	1.42	0.11	93.14	93.41	93.43
Joys Road Trib	Reach 1	705	2-Year	0.57	1.56	0.54	97.05	97.24	97.05
Joys Road Trib	Reach 1	705	5-Year	0.90	1.97	0.45	97.11	97.38	97.03
Joys Road Trib	Reach 1	705	10-Year	1.12	2.29	0.51	97.14	97.49	97.04
Joys Road Trib	Reach 1	705	25-Year	1.42	2.64	0.60	97.18	97.60	97.06
Joys Road Trib	Reach 1	664	2-Year	0.57	1.56	0.54	96.94	97.26	96.93
Joys Road Trib	Reach 1	664	5-Year	0.90	1.97	0.45	97.05	97.39	96.89
Joys Road Trib	Reach 1	664	10-Year	1.12	2.29	0.51	97.12	97.49	96.92
Joys Road Trib	Reach 1	664	25-Year	1.42	2.64	0.60	97.22	97.61	96.95
Joys Road Trib	Reach 1	635	2-Year	0.57	1.56	0.54	96.91	97.19	96.90
Joys Road Trib	Reach 1	635	5-Year	0.90	1.97	0.45	97.01	97.30	96.87
Joys Road Trib	Reach 1	635	10-Year	1.12	2.29	0.51	97.07	97.40	96.89
Joys Road Trib	Reach 1	635	25-Year	1.42	2.64	0.60	97.15	97.50	96.92
Joys Road Trib	Reach 1	634	Culvert						
Joys Road Trib	Reach 1	622	2-Year	0.57	1.56	0.54	96.89	97.08	96.88
Joys Road Trib	Reach 1	622	5-Year	0.90	1.97	0.45	96.96	97.14	96.86
Joys Road Trib	Reach 1	622	10-Year	1.12	2.29	0.51	97.00	97.18	96.87
Joys Road Trib	Reach 1	622	25-Year	1.42	2.64	0.60	97.06	97.21	96.89
Joys Road Trib	Reach 1	602	2-Year	0.57	1.56	0.54	96.88	97.07	96.87
Joys Road Trib	Reach 1	602	5-Year	0.90	1.97	0.45	96.95	97.13	96.84
Joys Road Trib	Reach 1	602	10-Year	1.12	2.29	0.51	96.99	97.17	96.86
Joys Road Trib	Reach 1	602	25-Year	1.42	2.64	0.60	97.05	97.21	96.88
Joys Road Trib	Reach 1	322	2-Year	0.57	1.56	0.54	96.39	96.54	96.38
Joys Road Trib	Reach 1	322	5-Year	0.90	1.97	0.45	96.45	96.58	96.36
Joys Road Trib	Reach 1	322	10-Year	1.12	2.29	0.51	96.49	96.61	96.37
Joys Road Trib	Reach 1	322	25-Year	1.42	2.64	0.60	96.52	96.65	96.39
Joys Road Trib	Reach 1	275	2-Year	0.57	1.56	0.54	96.17	96.31	96.17
Joys Road Trib	Reach 1	275	5-Year	0.90	1.97	0.45	96.21	96.39	96.16
Joys Road Trib	Reach 1	275	10-Year	1.12	2.29	0.51	96.23	96.44	96.17
Joys Road Trib	Reach 1	275	25-Year	1.42	2.64	0.60	96.28	96.49	96.18
Joys Road Trib	Reach 1	30	2-Year	0.57	1.56	0.54	95.67	96.05	95.63
Joys Road Trib	Reach 1	30	5-Year	0.90	1.97	0.45	95.84	96.15	95.59
Joys Road Trib	Reach 1	30	10-Year	1.12	2.29	0.51	95.94	96.20	95.64
Joys Road Trib	Reach 1	30	25-Year	1.42	2.64	0.60	96.04	96.25	95.70

(1) Scenario Descriptions:

1. The Van Gaal Drain 100-year 24-hour SCS peak flow reaches the Jock River.
2. The Van Gaal Drain 100-year spring snowmelt plus rainfall peak flow reaches the Jock River.
4. The Jock River 100-year spring snowmelt plus rainfall peak flow reaches the outlet of the Van Gaal Drain.

CALCULATION SHEET 1A: REQUIRED CAPACITY OF OVERLAND FLOW ROUTE

OVERLAND FLOW ROUTE FROM MH 710 TO SWM FACILITY 1 - CURB CUT WEIR

Approaching flow = 0.218 m^3/s for 100-yr event (on MAJ road segment)

Curb cut width = 4 m as per DSEL grading plan
Curb cut height = 0.050 m as per DSEL
Maximum flow depth at gutter = 0.290 m (0.15 m +0.035×4.00 m = 0.290 m for flow contained within RW)
Average head of water over curb cut = 0.240 m (0.15 m +0.035×4.00 m - 0.05 m = 0.240 m)
Curb cut weir coefficient = 1.84
Maximum flow through cub cut = 0.865 m^3/s for 100-yr event

Therefore the capacity of the curb cut (0.865 m^3/s) is higher than the computed overland flow (0.218 m^3/s)

OVERLAND FLOW ROUTE DOWNSTREAM OF CURB CUT

$Q = 1/n \times AR^{2/3}S^{1/2}$

normal depth = **0.128** m
n = 0.03
Channel width = 3 m
A (area of flow) = 0.384 m^2
wetted perimeter = 3.256 m
R (hydraulic radius) = 0.118 m
S (slope) = 0.005 m/m
Q (flow) = 0.218 m^3/s
velocity = 0.57 m/s

CALCULATION SHEET 1B: REQUIRED CAPACITY OF OVERLAND FLOW ROUTE

OVERLAND FLOW ROUTE FROM MH 1108 TO SWM FACILITY 2 - CURB CUT WEIR

Approaching flow = 0.616 m^3/s for 100-yr event (on MAJ road segment)

Curb cut width = 4 m as per DSEL grading plan
Curb cut height = 0.050 m as per DSEL
Maximum flow depth at gutter = 0.290 m (0.15 m +0.035×4.00 m = 0.290 m for flow contained within RW)
Average head of water over curb cut = 0.240 m (0.15 m +0.035×4.00 m - 0.05 m = 0.240 m)
Curb cut weir coefficient = 1.84
Maximum flow through cub cut = 0.865 m^3/s for 100-yr event

Therefore the capacity of the curb cut (0.865 m^3/s) is higher than the computed overland flow (0.616 m^3/s)

OVERLAND FLOW ROUTE DOWNSTREAM OF CURB CUT

$Q = 1/n \times AR^{2/3}S^{1/2}$

normal depth = **0.246** m
n = 0.03
Channel width = 3 m
A (area of flow) = 0.737 m^2
wetted perimeter = 3.491 m
R (hydraulic radius) = 0.211 m
S (slope) = 0.005 m/m
Q (flow) = 0.616 m^3/s
velocity = 0.84 m/s

from MOE Stormwater Management Planning and Design Manual

Table 6.3: Annual Sediment Loadings

Catchment Imperviousness	**Annual Loading (kg/ha)**	**Wet Density (kg/m^3)**	**Annual Loading (m^3/ha . yr)**
35	770	1230	0.6
55	2300	1230	1.9
70	3495	1230	2.8
85	4680	1230	3.8

Facility	Richmond SWM Facility #1 -	
Catchment Area	91.82	ha
Catchment Imperviousness	51	
Target Removal Frequency	80	% TSS Removal
Anticipated Cleaning Frequency	10	years
Drying Depth	1	m
Conservative Estimate	1.62	m^3/ha . Yr
	1488.5	m^3
Sediment Drying Area	1489	m^2

Facility	Richmond SWM Facility #2	
Catchment Area	34.99	ha
Catchment Imperviousness	51	
Target Removal Frequency	80	% TSS Removal
Anticipated Cleaning Frequency	10	years
Drying Depth	1	m
Conservative Estimate	1.62	m^3/ha . Yr
	567.2	m^3
Sediment Drying Area	567	m^2

APPENDIX J

JFSA Water Balance

J.F. Sabourin and Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL
CONSULTANTS

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October 31, 2013

David Schaeffer Engineering Ltd.
120 Iber Road, Unit 203
Ottawa, Ontario K2S 1E9

Attention: **Kevin Murphy, P.Eng.**

Subject: **Richmond Village (South) Limited Subdivision / Water Balance Analysis** *our file: 922-11*

As requested by your office, we have evaluated, based on the provided information as described below, the average annual infiltration volumes for the subject site under existing and proposed conditions.

As per the October 2013 *Richmond Village (South) Limited Subdivision / Preliminary Stormwater Management Analysis*, the proposed Richmond Village (South) development consists of a 126.81 ha drainage area to be treated by two Stormwater Management (SWM) facilities; SWM Facility 1 (91.82 ha at 51% imperviousness) discharging to Van Gaal Drain, and SWM Facility 2 (34.99 ha at 51% imperviousness) discharging to the Jock River. Note that, on the assumption that approximately half of each proposed building roof will be serviced by roof leaders directed onto grassed areas, the directly connected imperviousness of the proposed subdivision is approximately 45% for the drainage area to SWM Facility 1, and 46% for the drainage area to SWM Facility 2.

Existing drainage characteristics of the subject site are as per the *Floodplain Mapping Report for the Van Gaal and Arbuckle Municipal Drains in the Village of Richmond* (November 2009, JFSA). Refer to the October 2013 *Richmond Village (South) Limited Subdivision / Preliminary Stormwater Management Plan* memo for existing and proposed drainage plans and further details.

Under existing and proposed conditions, by means of 36 years of continuous hydrologic simulations using hourly rainfall data from the Ottawa International Airport from 1967 to 2003 (excluding missing 2001 rainfall data), the average annual runoff volumes from the subject site were computed and compared. Continuous modelling parameters were set as follows for both existing and proposed conditions:

APII=[50], APIK=[0.90]/day;	used to compute the Antecedent Precipitation Index during the continuous simulation. Without model calibration these are the default values.
IaREC=[6](hrs);	the time that it takes for the Initial Abstraction over pervious areas to recover during a dry period in undeveloped areas.
SMIN=[-1], SMAX=[-1](mm);	the negative values indicate that the storage volume in the SCS procedure will vary between the "S" determined for AMC I and AMC III conditions of the entered CN value in undeveloped and urban areas.
SK=[0.03]/(mm);	a calibration coefficient that can typically vary from 0.01 to 0.3 for undeveloped and urban areas. The higher the value, the more runoff generated. To set the baseline for existing conditions, we decided to take a value in the low range.
InitGWResVol=[100](mm), GWResK=[0.9](mm/day/mm), VhydCond=[1](mm/hr);	parameters that are used to simulate both the groundwater storage and discharge to surface watercourses from undeveloped areas. Without

	adequate field measurements, these parameters were selected based on previous experience.
IaRECper=[3](hrs);	the time that it takes for the Initial Abstraction over pervious areas to recover during a dry period in urban areas.
IaRECimp=[2](hrs);	the time that it takes for the Initial Abstraction over impervious areas to recover during a dry period in urban areas.
InterEventTime=[12](hrs);	the continuous dry time required to reset the parameters in the SCS procedure to their initial values.

Note that the subject site has a relatively high Curve Number (CN) of 88 under existing conditions as per the November 2009 *Floodplain Mapping Report*. In order to best represent the slow infiltration rates of the existing soils, and to provide a consistent comparison between existing and proposed conditions, the SCS procedure was used to simulate infiltration over the subject site for both existing and proposed conditions. Continuous hydrologic simulations were also performed with a CN of 99.99, in order to simulate the runoff from the subject site if no infiltration takes place. The difference between the runoff simulated with the actual CN of 88, and the runoff simulated with the CN of 99.99, is equal to the infiltrated volume over the subject site.

Based on the existing and proposed continuous simulations, the average annual infiltration over the subject site is approximately 58.4% less under proposed conditions (89,007 m^3) than under existing conditions (214,079 m^3).

A summary of the water balance analysis results may be found in Attachment A. Digital SWMHYMO modelling input and output files are also attached.

Yours truly,
J.F. Sabourin and Associates Inc.

Laura Pipkins, P.Eng.

cc: J.F. Sabourin, M.Eng, P.Eng.
Director of Water Resources Projects

Attachment A: Simulated Annual Infiltration Volumes for the Richmond Village (South) Limited Subdivision Site

ATTACHMENT

A

Simulated Annual Infiltration Volumes for the Richmond Village (South) Limited Subdivision Site

JFSA Water Resources and Environmental Consultants

Table 1: Richmond Village (South) Limited Subdivision Infiltration Volumes Under Existing Conditions (1)

Year	Total Rainfall		Runoff (No Infiltration)		Runoff (With Infiltration)		Infiltration	
	(mm)	(m^3)	(mm)	(m^3)	(mm)	(m^3)	(mm)	(m^3)
1967	386.9	490628	253.96	322047	148.87	188782	105.09	133265
1968	592.8	751730	338.04	428669	163.51	207347	174.53	221321
1969	569.8	722563	312.40	396154	147.21	186677	165.19	209477
1970	558.9	708741	306.76	389002	143.44	181896	163.32	207106
1971	522.1	662075	258.93	328349	113.48	143904	145.45	184445
1972	784.3	994571	484.87	614864	263.87	334614	221.00	280250
1973	744.9	944608	433.84	550153	212.13	269002	221.71	281150
1974	386.2	489740	184.15	233521	78.73	99838	105.42	133683
1975	535.5	679068	301.57	382421	140.55	178231	161.02	204189
1976	492.4	624412	238.57	302531	105.76	134114	132.81	168416
1977	677.6	859265	369.37	468398	169.93	215488	199.44	252910
1978	638.8	810062	345.64	438306	142.15	180260	203.49	258046
1979	866.5	1098809	540.45	685345	281.15	356526	259.30	328818
1980	622.0	788758	328.88	417053	147.69	187286	181.19	229767
1981	936.4	1187449	562.73	713598	318.35	403700	244.38	309898
1982	596.1	755914	300.13	380595	123.91	157130	176.22	223465
1983	587.3	744755	288.49	365834	129.88	164701	158.61	201133
1984	459.4	582565	268.94	341043	128.13	162482	140.81	178561
1985	559.9	710009	316.74	401658	127.56	161759	189.18	239899
1986	849.4	1077124	509.26	645793	282.46	358188	226.80	287605
1987	639.9	811457	321.75	408011	157.93	200271	163.82	207740
1988	643.2	815642	336.59	426830	161.03	204202	175.56	222628
1989	522.5	662582	260.01	329719	112.97	143257	147.04	186461
1990	727.8	922923	392.68	497958	190.63	241738	202.05	256220
1991	555.8	704810	264.10	334905	118.36	150092	145.74	184813
1992	730.2	925967	398.37	505173	197.92	250982	200.45	254191
1993	721.1	914427	373.67	473851	168.11	213180	205.56	260671
1994	527.0	668289	306.73	388964	150.72	191128	156.01	197836
1995	321.6	407821	216.31	274303	130.16	165056	86.15	109247
1996	512.2	649521	277.39	351758	129.99	164840	147.40	186918
1997	433.2	549341	232.72	295112	92.08	116767	140.64	178346
1998	440.3	558344	231.72	293844	100.94	128002	130.78	165842
1999	424.4	538182	243.43	308694	104.27	132225	139.16	176469
2000	535.9	679575	293.17	371769	144.61	183380	148.56	188389
2002	551.5	699357	367.64	466204	205.93	261140	161.71	205064
2003	554.6	703288	326.84	414466	174.96	221867	151.88	192599
Average		747066		415191		201113		214079
Minimum		407821		233521		99838		109247
Maximum		1187449		713598		403700		328818

(1) For a 126.81 ha drainage area as per the October 2013 *Preliminary Stormwater Management Plan* memo.

Note: Average Annual Percentage Infiltration = 28.7%.

Table 2: Richmond Village (South) Limited Subdivision Infiltration Volumes Under Proposed Conditions (1)

Year	Total Rainfall		Runoff (No Infiltration)		Runoff (With Infiltration)		Infiltration	
	(mm)	(m^3)	(mm)	(m^3)	(mm)	(m^3)	(mm)	(m^3)
1967	386.9	490628	242.82	307920	196.76	249511	46.06	58409
1968	592.8	751730	347.65	440855	274.34	347891	73.31	92964
1969	569.8	722563	318.26	403586	248.93	315668	69.33	87917
1970	558.9	708741	310.40	393618	242.19	307121	68.21	86497
1971	522.1	662075	273.24	346496	213.62	270892	59.62	75604
1972	784.3	994571	487.38	618047	393.59	499111	93.79	118935
1973	744.9	944608	445.68	565167	348.72	442212	96.96	122955
1974	386.2	489740	190.32	241345	149.25	189264	41.07	52081
1975	535.5	679068	307.41	389827	240.13	304509	67.28	85318
1976	492.4	624412	242.87	307983	192.04	243526	50.83	64458
1977	677.6	859265	385.52	488878	301.73	382624	83.79	106254
1978	638.8	810062	366.36	464581	278.14	352709	88.22	111872
1979	866.5	1098809	550.21	697721	437.19	554401	113.02	143321
1980	622.0	788758	340.46	431737	264.71	335679	75.75	96059
1981	936.4	1187449	571.16	724288	469.71	595639	101.45	128649
1982	596.1	755914	323.51	410243	252.13	319726	71.38	90517
1983	587.3	744755	301.25	382015	239.28	303431	61.97	78584
1984	459.4	582565	267.20	338836	207.17	262712	60.03	76124
1985	559.9	710009	335.03	424852	250.23	317317	84.80	107535
1986	849.4	1077124	507.20	643180	412.94	523649	94.26	119531
1987	639.9	811457	325.93	413312	263.69	334385	62.24	78927
1988	643.2	815642	345.53	438167	275.46	349311	70.07	88856
1989	522.5	662582	272.43	345468	212.29	269205	60.14	76264
1990	727.8	922923	403.48	511653	320.38	406274	83.10	105379
1991	555.8	704810	278.48	353140	223.39	283281	55.09	69860
1992	730.2	925967	404.60	513073	325.65	412957	78.95	100116
1993	721.1	914427	395.96	502117	313.49	397537	82.47	104580
1994	527.0	668289	307.63	390106	243.13	308313	64.50	81792
1995	321.6	407821	211.03	267607	173.42	219914	37.61	47693
1996	512.2	649521	284.57	360863	224.69	284929	59.88	75934
1997	433.2	549341	242.73	307806	184.03	233368	58.70	74437
1998	440.3	558344	235.46	298587	181.14	229704	54.32	68883
1999	424.4	538182	244.78	310406	185.49	235220	59.29	75186
2000	535.9	679575	299.89	380291	236.95	300476	62.94	79814
2002	551.5	699357	367.44	465951	296.52	376017	70.92	89934
2003	554.6	703288	333.96	423495	268.51	340498	65.45	82997
Average		747066		425089		336083		89007
Minimum		407821		241345		189264		47693
Maximum		1187449		724288		595639		143321

(1) For a 126.81 ha drainage area as per the October 2013 *Preliminary Stormwater Management Plan* memo.

Note: Average Annual Percentage Infiltration = 11.9%;

Proposed conditions average annual infiltration volume = 58.4% less than existing conditions volume.