Golder Associates

TECHNICAL MEMORANDUM

DATE June 5, 2013 **PROJECT No.** 12-1127-0062/4000

TO Frank Cairo
Richmond Village (South) Limited

CC Mike Green
Mattamy (Jock River) Limited

FROM Brian Byerley, P.Eng. **EMAIL** bbyerley@golder.com

UPDATED ASSESSMENT OF SUBSURFACE DRAINAGE AND ANALYSIS OF 100-YEAR STORM EVENT PROPOSED VILLAGE OF RICHMOND DEVELOPMENT, OTTAWA, ONTARIO

Golder Associates Ltd. (Golder) was retained by Richmond Village (South) Limited (RVSL) to assess the hydrogeological effects of subsurface drainage for the proposed development by RVSL and Mattamy (Jock River) Limited (Mattamy) in the Village of Richmond, in Ottawa, Ontario. The development site extends northwesterly from the Jock River, on the west side of Richmond, for a distance of about 2.5 km (See Figure 1). The overall objective of this work was to evaluate the suitability of the proposed drainage plan for the development (completed by David Schaeffer Engineering Ltd. (DSEL)) with respect to the hydrogeological conditions encountered at the site.

The current assessment was undertaken in order to estimate:

- The average long term groundwater conditions based on the proposed site drainage plan;
- The time to achieve the long term groundwater conditions;
- groundwater levels and groundwater inflows to the foundation drains during the 100-year storm event superimposed on the spring freshet; and,
- The maximum expected sump pump pumping rate (including snow melt and roof discharge).

The tasks involved in this assessment included a review of the available drainage plans, hydrogeological data, and previous work completed at the site. This technical memorandum is a revision of an initial technical memorandum on this work issued on October 3, 2013. This revision is intended to address technical comments received from Dillon Consulting (Dillon) and the City of Ottawa on the original memorandum. In response to the technical comments, the groundwater models have been modified, and additional model documentation has been included as requested by Dillon.

Data Sources

Available data that was reviewed as a part of this study is summarized as follows:

- DSEL design drawings relating to the Richmond Village storm water management ponds (DSEL project 11-486, Figures 12 and 13), surface grading plan (DSEL project 11-486, Figure 3), storm servicing plan (DSEL project 11-486, Figure 4), and storm trunk profiles (DSEL project 11-486, Figures 5, 6, and 7);

- DSEL Storm Water Management Report (DSEL Project 11-486) dated April 2012 (DSEL, 2012);
- Borehole logs (Golder 2010a) and test pit logs (Jacques-Whitford, 2007);
- Monthly groundwater elevations collected from site piezometers between April 2010 and April 2011, as summarized in Golder's August 11, 2011 technical memorandum to Susan Murphy (Golder, 2011);
- Results of hydraulic testing of the overburden and bedrock across the site, as summarized in Golder's July 16, 2010 memo to Susan Murphy (Golder, 2010a); and,
- Results of a previous modelling assessment of groundwater inflow to building foundations (Golder, 2010b).

In addition to the above, supplementary groundwater elevation data and hydraulic response testing data were collected in May 2012. Groundwater elevation data are included in Table 1 and shown in Figure 2, attached. Hydraulic response testing date are presented in the following section of this technical memorandum.

Hydraulic Testing

Single well response tests were conducted in on-site monitoring wells on May 3, 2010 as part of a previous hydrogeological investigation. Additional hydraulic testing was carried out on a subset of these monitoring wells on May 1, 2012 to confirm hydraulic conductivity (K) estimates of the overburden and shallow bedrock material within the study area. Static groundwater levels were established prior to testing. The rate of water level recovery in the monitoring well was measured following the addition (falling head test) or removal (rising head test) of a slug, displacing the water column by a known amount. Water level recovery in each monitoring well was measured manually and continuously (0.5 to 2 second interval) with a pressure transducer.

The data collected were then analyzed by using the Hvorslev Method (Hvorslev, 1951). The hydraulic conductivities estimated from the 2012 in-situ hydraulic testing are consistent with the results of the 2010 hydrogeological investigation, as summarized in the table below.

Summary of In-Situ Hydraulic Conductivity Estimates (Hvorslev Analysis)

Well ID	May 3, 2010 K (m/s)	May 1, 2012 K (m/s)	Stratigraphy at Well Screen
MW10-1A	$5x10^{-6}$	$5x10^{-6}$	Grey silty Clay
MW10-1B	$8x10^{-6}$	$6x10^{-6}$	Grey Brown silty Clay (Weathered Crust)
MW10-2	$1x10^{-6}$	--	Grey Brown silty fine Sand
MW10-3A	$2x10^{-5}$	$1x10^{-5}$	fresh Grey Dolomite
MW10-3B	$4x10^{-6}$	--	Grey Brown silty Clay (Weathered Crust)
MW10-4A	$3x10^{-6}$	--	Grey Brown fine sandy Silt
MW10-4B	$1x10^{-5}$	$1x10^{-5}$	Grey Brown silty Clay (Weathered Crust)
MW10-5A	$5x10^{-6}$	--	Grey sandy silt some gravel trace clay (Glacial Till)
MW10-5B	$2x10^{-6}$	--	Grey Brown silty fine Sand
MW10-6A	$4x10^{-6}$	$5x10^{-6}$	fresh Grey Dolomite
MW10-6B	$7x10^{-6}$	--	Grey Brown silty Sand trace Clay
MW10-7	$3x10^{-6}$	--	Grey Brown silty fine Sand
MW10-8	*	$1x10^{-4}$	Weathered to fresh Grey Dolomite

Notes: * Water level recovery too fast to measure manually, K value could not be estimated.
-- In-situ hydraulic conductivity was not completed at the monitoring well on May 1, 2012

It is noted here that hydraulic conductivity estimates from the silty clay material on the site are higher than typically encountered. Literature values of hydraulic conductivity for silty clay range from 1 x 10^{-9} to 1x10^{-7} m/s (Freeze and Cherry, 1979). It is inferred that the single well response test results are representative of the horizontal hydraulic conductivity of the material, resulting from the presence of silt lenses within the clay. Based on Golder's experience with marine clay in the area, it is expected that the vertical hydraulic conductivity of the material is much lower, and closer to literature values for silty clay.

Site Drainage Plan

Based on a review of the site drainage plan, it is Golder's understanding that the initial groundwater drainage of the site will occur through the granular backfill material within service trenches, which would be completed along the roadway centre lines. Groundwater collected by the service trenches will discharge at ground surface near surface water control features (storm water management ponds). As the proposed inverts of the service trenches are below the high groundwater elevations observed at the site, the installation and construction of the service trenches is expected to result in long term site wide lowering of groundwater elevations.

The underside of footing (USF) elevation for each dwelling will be placed above the post development normal high groundwater elevation; however, a foundation drain system will be required for short-term drainage needs during storm events. As a component of the proposed servicing plan the foundation drains of each dwelling will be connected to sumps which will discharge to the storm sewer. The granular bedding material in the service trenches will not be used as backfill around service connections to the dwellings (i.e. there will not be a direct connection from the service trenches to the foundation drains through the granular material). However, because of the open connection between the service trench bedding and the storm water management ponds, the groundwater elevations in the service trench bedding will be controlled by surface water elevations at the trench discharge point.

Model Construction

Two numerical groundwater flow (MODFLOW) models were developed to complete the hydrogeological assessment. The construction of these models was based on previously constructed models that provided estimates of groundwater inflow to basement foundation drains (Golder, 2010b). Details regarding the previous model construction and parameterization can be found in the March 2010 Technical Memorandum (Golder, 2010b). As a part of the current assessment, one model, representative of a large portion of the proposed development, was used to predict the long term groundwater levels at the Site based on the proposed drainage plan. This model is referred to herein as the "Long Term Drainage Model". A second model, representative of a single lot, was used to evaluate the short-term sump pump response to the 100-year storm and the spring freshet. This model is referred to herein as the "100-year Storm Event Model".

The construction, parameterization, and calibration of each model are summarized below.

Long Term Drainage Model

The following details summarize the long term drainage model construction and assumptions made:

- The model covers an area of 695 m x 860 m to a depth of 94 m below the simulated ground surface. The model is horizontally discretized into a 5 m x 5 m grid over the domain. The grid transitions to 1 m x 1 m blocks around the sewer trench boundaries. There are 183 rows and 193 columns in the model. The model is vertically discretized into 12 layers, ranging in thickness from 0.5 m at ground surface to 85 m at depth;
- The model domain was defined based on the centre portion of the proposed development, bounded on the east by Strachan Street and on the west by Perth Street (Shown in Figure 1). This domain was chosen to represent the conditions for the construction of the first houses (approximately 150 houses);

- Overburden thickness varies linearly from 6 metres (m) at the Perth St. Boundary to 3 m at the Ottawa Street Boundary based on test pit logs by Jacques Whitford (2007). Overburden properties were varied from silty sand in the southern portion of the model, to silty clay in the northern portion of the model based on borehole information collected by Golder, and the Jacques Whitford (2007) test pit logs. The horizontal hydraulic conductivity of all overburden units was assigned a value of 5 $x10^{-6}$ m/s based on the in-situ hydraulic conductivity estimates described above. The silty sand unit was assumed to be isotropic. The vertical hydraulic conductivity of the silty clay was assigned a value of $1x10^{-8}$ m/s based literature values (Freeze and Cherry, 1979) and model calibration to observed hydraulic heads;

- The upper 2 m of bedrock in the model was assigned a horizontal hydraulic conductivity of $5x10^{-6}$ m/s, representative of an upper weathered unit. All other bedrock was assigned a horizontal hydraulic conductivity of $5x10^{-7}$ m/s. Both bedrock units were assigned an anisotropy ratio of 10:1. The distribution of hydraulic properties within the model domain are shown on Figure 3 (in plan) and Figure 4 (in cross-section);

- The specific yield of the overburden material was 0.2. The specific storage of the overburden and rock units was $1x10^{-5}$ m^{-1}. These values are considered representative of the materials encountered during subsurface investigations at the site;

- Groundwater was assumed to flow to the north-east towards the Arbuckle Municipal Drain. Constant head boundaries were specified through all bedrock layers in the model to create an average horizontal hydraulic gradient of 0.002 m/m and an average groundwater elevation of 94.2 masl (value in the centre of the modelled portion of the development). These values correspond to groundwater elevations measured at the Site in April 2011, which were the highest recorded groundwater elevations observed during the monthly monitoring summarized in the August 2011 Technical Memorandum (Golder, 2011);

- The bottom of the model was specified as a no flow boundary at an elevation of 0 m;

- During all monitoring sessions an upward gradient between the bedrock and the silty clay overburden was observed at MW10-3. Upward vertical gradients were not observed between the bedrock and the silty sand overburden in MW10-6. It is interpreted that the upward gradients observed at MW10-3 are the result of the confinement of the bedrock aquifer by the silty clay, and the proximity of the well to the Arbuckle Drain (approximately 15 m to the north), which likely controls the groundwater elevations in the silty clay layer. To account for the presence of the vertical gradients, the Arbuckle Drain was specified in the model using a drain boundary condition. The elevation of the boundary was specified at 93 m based on topographic mapping. All boundary conditions specified in the long term model are shown on Figure 5; and,

- Recharge of 10 mm/year was applied to the silty sand, while 2 mm/year was applied to the silty clay. These values were adjusted to limit mounding of groundwater.

The Long Term Drainage Model was initially run without any of the boundaries that represent the service trenches and Storm Water Management Pond 1, to establish an initial groundwater condition approximating conditions observed in April 2011 (representative of high groundwater level conditions). The predevelopment groundwater elevations based on this simulation are shown on Figure 6. Groundwater elevations observed during the April 2011 monitoring session are also tabulated on Figure 6 for comparison.

To simulate the transient drainage of the site due to the proposed site drainage plan, the model was changed as follows:

- Storm sewer invert elevations were assigned based on drawings provided to Golder by DSEL. The depth of the sewer inverts (relative to the proposed road surface) ranged from 2.0 m at the southwest portion of the site, to 2.5 m near Pond 1. Drain boundaries were placed an additional 0.1 m below the sewer inverts to represent the bedding layer below the sewer pipe. Drain conductance was specified as 250 m^2/day. Drain boundary cells were 1 m in width;
- Storm Water Management Pond 1 was specified as a constant head boundary at an elevation of 92.35 m, which represents the normal operating level of the pond (based on information provided by DSEL). All boundary conditions specified for the long term model are shown on Figure 7;
- Drain and constant heads cells were assigned the same hydraulic conductivity as the native overburden;
- Backyard ditches and sump pumps (which could further lower the water table) were not included in the simulation;
- Infiltration to the groundwater table from surface recharge was assumed to be consistent with current conditions (i.e., reduction in recharge following placement of hard surfaces - roofs, pavement, etc. – was not considered); and,
- It was assumed that no short circuiting of flow between the service trench fill and the storm sewer pipe would occur. Equivalent porous media assumptions apply at this interface (i.e., no flow would occur along any gaps between the backfill material and the storm sewer pipe).

100-Year Storm Event Model

The following details summarize 100-year storm model construction and assumptions made:

- The model covers an area of 10 m x 14 m to a depth of 21 m below the simulated ground surface. The model is horizontally discretized into a 0.2 m x 0.2 m grid over the domain. The grid transitions to 0.1 m x 0.2 m blocks around the boundaries representing the foundation drains and service trenches. There are 48 rows and 44 columns in the model. The model is vertically discretized into 52 layers, ranging in thickness from 0.1 m to 1 m;
- The ground surface at the road centre line (post-grading) elevation was assigned as 95.15 m, and the elevation of the top of rock was assigned as 89.95 masl. These values correspond to the area (see Figure 1) that is hydraulically connected to Pond 1. The elevation of Pond 1 during the 100-year storm event is expected to be 94.11 m;
- The representative horizontal hydraulic conductivity of the overburden, weathered bedrock, and competent bedrock were 5×10^{-6} m/s, 5×10^{-5} m/s, and 5×10^{-7} m/s, respectively. The weather bedrock unit was assumed to be 2 m thick. The representative horizontal hydraulic conductivity of the overburden was increased from 1×10^{6} m/s in the original 2010 analysis (Golder, 2010b) to 5×10^{-6} m/s based on the results of hydraulic testing (described above). A horizontal to vertical anisotropy ratio of 10:1 was maintained for each hydrostratigraphic unit. The hydraulic property distributions in the model in plan and cross-section are shown on Figure 8;
- The specific yield of the overburden material was 0.2. The specific storage of the overburden and rock units was 1×10^{-5} m^{-1}. These values are considered representative of the materials encountered during subsurface investigations at the site;

- The foundation drain elevation (93.08 masl) was assigned using a drain boundary at 2.07 m below the road elevation. This assumes a 3% surface grade across the lot, an 11 m set-back from the centreline of the road, and a 2.4 m depth from ground surface to the foundation drain elevation, and is considered representative of the lowest USF depth relative to the proposed road elevations, based on drawings provided by DSEL;
- A constant head boundary, representative of the storm sewer trench was set to an elevation of 0.1 m below the foundation drain elevation (92.98 masl). This is representative of a 0.15 m separation between the storm sewer invert and the foundation drain elevations (DSEL, 2012), and assumes 0.05 m depth of water in the sewer bedding above the sewer invert;
- A service trench 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 1 m width was assumed for all trenches. The constant head boundary was assigned at the same elevation as the storm sewer trench. Boundary conditions specified in the model are shown on Figure 8;
- It was assumed that the portions of the storm sewer and foundation drain specified in the model only draw water from areas located within the model domain (i.e. it is assumed that the sump pumps, and service stubs will generate groundwater flow divides at each property boundary), and regional gradients are not considered;
- Drain and constant heads cells were assigned the same hydraulic conductivity of the native overburden;
- No backyard ditches or neighbouring basement drains were included in the simulation; and,
- It was assumed that no short circuiting of flow between the service trench fill and the storm sewer pipe would occur. Equivalent porous media assumptions apply at this interface.

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). The initial groundwater elevations for the 100-year storm simulation are shown on Figure 9. 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 94.11 masl. 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. Two additional simulations were completed in which the recharge was increased to 4000 mm/year and 8000 mm/year to evaluate the sensitivity of the model to this parameter.

Following the storm event, groundwater elevations in the storm sewer and service stub were lowered to 92.98 m, and the recharge was reduced to 90 mm/year. Groundwater elevations within the service trenches were assumed to decrease instantaneously 24 hours after the start of the storm based on information provided by DSEL. The instantaneous rise and fall of groundwater elevations is expected to generate more inflow to the foundation drains than would be generated by the expected gradual changes that are more likely to occur within the same 24 hour time period.

Results

Long Term Drainage Model

The simulated average long term (steady-state) groundwater elevation within the proposed development following installation of the storm sewer network was 93.45 masl. Long term simulated groundwater elevations varied from 93.7 masl in the southeastern corner of the site to 92.35 masl along the edge of Storm Water Management Pond 1.

These conditions were achieved approximately 400 days following the initiation of drainage by the storm sewer network. Steady-state groundwater elevations are shown on Figure 10 and hydraulic head drawdown in comparison to the initial condition (representative of the high water levels observed in April 2011) is shown on Figure 11. It is noted that the drawdown in the area simulated ranged from 1 m (at the southwest boundary of the site) to more than 1.9 m (at the edge of Pond 1) in comparison to the high groundwater elevations measured in April 2011.

Based on the simulated long term groundwater elevations, USFs placed at depths less than 1.9 m below the proposed road surface should be above the steady-state groundwater level. This maximum USF depth is based on the assumption that:

- The storm sewer bedding will permit drainage to a level that is 0.1 m below the proposed sewer invert; and,
- House foundations will be set back no more than 20 m from the road centreline.

Given the period of time required to achieve the steady-state groundwater elevations it is likely that short term dewatering activities will be required during construction.

It is noted that other areas of the site with similar hydrogeological conditions (such as the areas north of Ottawa Street and north of Perth Street) can be expected to undergo similar levels of groundwater lowering if the same site drainage plan is implemented. Hydrogeological conditions to the south of Ottawa Street are somewhat different from other areas of the site; therefore, further evaluation of the drainage plan for this portion of the development is recommended prior to finalizing the USF elevations.

100-Year Storm Event Model

Groundwater elevations following the simulated 24 hour storm event are shown on Figure 13 for each of the three recharge scenarios simulated. The simulated peak groundwater elevation in the model cell adjacent to the foundation drains (0.1 m from the drains) was 0.1 m above the USF elevation for all 3 recharge values simulated. The peak groundwater elevation at 1 m from the drains was 0.40 to 0.50 m above the USF elevation for the recharge conditions simulated. It is noted that groundwater mounding occurred in each of the simulations due to the high recharge values specified.

Simulated groundwater inflow to the foundation drains and service trenches are shown on Figure 14. The simulated peak 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 mm/year). These values represent the maximum expected sump pump pumping rate due to groundwater inflow; they do not include any potential short-circuiting of surface runoff to the drains. It is noted that a small portion of the flow out of the model domain during the storm event was through the sewer trenches, due to the high levels of recharge applied to simulate freshet conditions. Following the storm event, flow to the foundation drains ended within 12 hours as gradients were reversed towards the storm sewer. Inflow rates following the storm conditions are included on Figure 14. It is noted that the service trenches and stubs would quickly convey water away from the foundation drains. In the 12 hours following the storm event a total of 0.08 m^3 to 0.1 m^3 flows to the foundation drains, while 0.5 m^3 to 0.6 m^3 flow to the service trenches for the recharge conditions specified.

Summary and Conclusion

The results of the modelling assessment indicate the following:

- The simulated long term (steady-state) groundwater elevations were between 93.7 masl and 92.35 masl. The results indicate that USF elevations may be placed a maximum of 1.9 m below the proposed road grade. The constant head boundaries specified to simulate regional flow were selected to approximate groundwater elevations observed in April 2011 (the highest groundwater elevations measured on-site). Based on the groundwater level monitoring data (see Figure 2), lower groundwater elevations occur during dryer times of year;

- It is expected that sump pumps would be required to operate during a 100-year storm event; however, the duration of operation would not be expected to extend more than 12 hours beyond the duration of the 100-year storm event. Sump pump operation may also be required during the spring freshet, or during any period of high recharge, as is typical for dwellings with sump pumps. Based on the results of the groundwater modelling, the expected volume of water that would be pumped at each dwelling would be easily handled by standard commercially available residential-type sump pumps;

- Based on the site's subsurface conditions and the results of the groundwater modelling, it is expected that the amount of groundwater lowering that would occur through the bedding material within the service trenches and the storm water management ponds would not adversely affect water levels in water supply wells, or existing and future residential foundations; and,

- The modelling assumed that groundwater drainage of the site will occur through the bedding material within service trenches, and groundwater collected by the service trenches will discharge at ground surface near the storm water management pond. The modeling also assumed that the granular material within the service trenches will not directly connect to foundation drains. Inspection during construction, to ensure proper construction of the drainage system, is recommended.

Limitations

This report was prepared for the use of Richmond Village (South) Limited and Mattamy (Jock River) Limited. The report, which specifically includes all tables, figures and appendices, is based on data gathered by Golder Associates Ltd., and information provided to Golder Associates Ltd. by others. The information provided by others has not been independently verified or otherwise examined by Golder Associates Ltd. to determine the accuracy or completeness. Golder Associates Ltd. has relied in good faith on this information and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the information as a result of omissions, misinterpretation or fraudulent acts.

The assessment of environmental conditions and possible hazards at this site has been made using the results of physical measurements from a number of locations. The site conditions between testing locations have been inferred based on conditions observed at the testing locations. Actual conditions may deviate from the inferred values.

Hydrogeological investigations and groundwater modelling are dynamic and inexact sciences. They are dynamic in the sense that the state of any hydrological system is changing with time, and in the sense that the science is continually developing new techniques to evaluate these systems. They are inexact in the sense that groundwater systems are complicated beyond human capability to evaluate them comprehensively in detail, and we invariably do not have sufficient data to do so. A groundwater model uses the laws of science and mathematics to draw together the available data into a mathematical or computer-based representation of the essential features of an existing hydrogeological system. While the model itself obviously lacks the detailed reality of the existing hydrogeological system, the behaviour of a valid groundwater model reasonably approximates that of the real system. The validity and accuracy of the model depends on the amount of data available relative to the degree of complexity of the geologic formations and on the quality and degree of accuracy of the data entered. Therefore, every groundwater model is a simplification of a reality and the model described in this report is not an exception.

The professional groundwater modelling services performed as described in this report were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practising under similar conditions, subject to the quality and quality of available data, the time limits and financial and physical constraints applicable to the services. Unless otherwise specified, the results of previous or simultaneous work provided by sources other than Golder Associates Ltd. and quoted and/or used herein are considered as having been obtained according to recognized and accepted professional rules and practices, and therefore deemed valid. This model provides a predictive scientific tool to evaluate the impacts on a real groundwater system of specified hydrological stresses and/or to compare various scenarios in a decision-making process. However and despite the professional care taken during the construction of the model and in conducting the simulations, its accuracy is bound to the normal uncertainty associated to groundwater modelling and no warranty, express or implied, is made.

Any use which a third party makes of this report, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made, or actions taken based on this report.

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.

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LICENSED PROFESSIONAL ENGINEER
B. T. BYERLEY
June 5/13
PROVINCE OF ONTARIO

MIB/BTB/SRW/sg

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Attachments: Table 1 – Groundwater Monitoring Data
Figure 1 – Site Plan
Figure 2 – Observed Groundwater Levels
Figure 3 – Hydraulic Conductivity Distribution in Plan View (Long Term Model)
Figure 4 – Hydraulic Conductivity Distribution in Cross-Section (Long Term Model)
Figure 5 – Boundary Conditions for Predevelopment Conditions (Long Term Model)
Figure 6 – Simulated Groundwater Elevations for Predevelopment Conditions (Long Term Model)
Figure 7 – Boundary Conditions for Post Development Conditions (Long Term Model)
Figure 8 – Hydraulic Conductivity Distribution and Boundary Conditions (100-Year Storm Model)
Figure 9 – Initial Groundwater Elevations (100-Year Storm Model)
Figure 10 – Simulated Steady State Groundwater Elevations (Long Term Model)
Figure 11 – Simulated Steady State Hydraulic Head Drawdown (Long Term Model)
Figure 12 – Simulated Groundwater Elevation Contours following the 24 hour 100-Year Storm
Figure 13 – Simulated Groundwater Flows to Boundaries (100-Year Storm Model)

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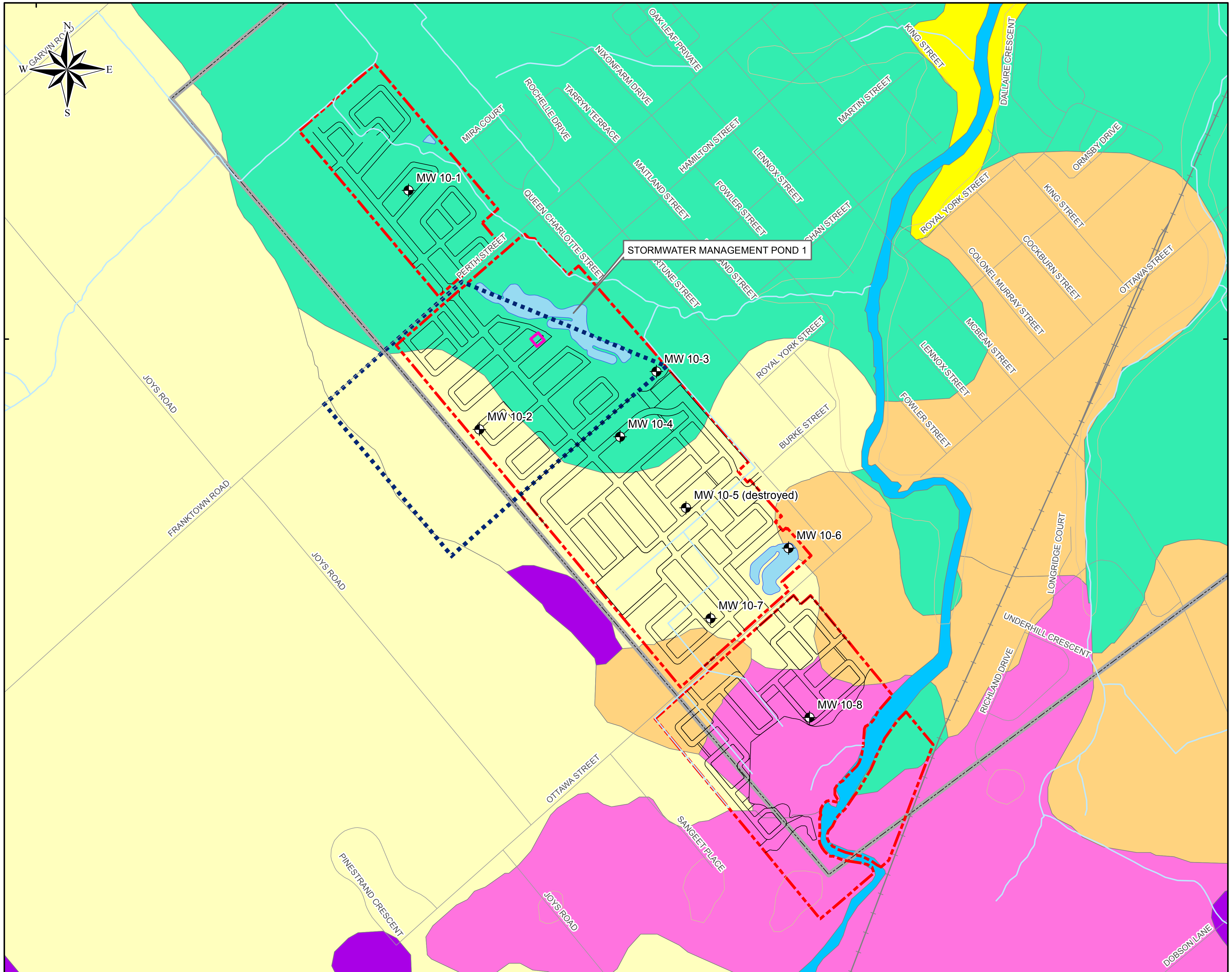
Table 1: Groundwater Monitoring Data

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, 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. 15, 2011	May 1, 2012
MW10-1A	94.55	3.15	Grey silty Clay	0.73	0.78	0.80	0.81	0.78	0.81	0.61	0.77	0.72	0.78	1.00	1.21	0.89	0.64	0.83
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	0.75	0.72	0.78	0.97	1.17	0.90	0.66	0.83
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	0.59	0.34	0.55	0.78	0.89	0.56	0.13	--[4]
MW10-3A	94.0	4.55	Fresh grey Dolomite	0.22	0.26	0.58	0.28	0.23	0.25	-0.09[1]	0.08	-0.06[1]	--[3]	--[3]	--[3]	--[3]	-0.21[1]	0.25
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	0.72	0.56	0.73	0.90	0.77	0.69	0.29	0.95
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	0.47	0.27	0.44	0.53	0.64	0.41	0.21	--[4]
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	0.46	0.25	0.44	0.56	0.65	--[3]	0.26	0.78
MW10-5A	95.65	3.03	Glacial Till	0.82	0.79	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[4]
MW10-5B	95.65	1.21	Grey brown silty find Sand	0.89	0.88	1.33	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]	--[4]
MW10-6A	95.67	4.24	Fresh grey Dolomite	1.38	1.38	1.98	1.34	1.31	1.52	0.66	0.84	0.67	0.68	1.22	1.44	0.56	0.46	1.21
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	0.62	0.26	0.50	1.16	1.42	--[3]	0.05	1.15
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	0.06	0.04	0.03	--[3]	--[3]	--[3]	0.02	--[4]
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	0.31	0.26	0.27	0.80	0.81	--[3]	0.19	0.74

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.
4 Only select wells were monitored in May 2012 as a component of a hydraulic response testing program.



N:\Active\2012\1127 - Geosciences\12-1127-0062 Richmond Village Drainage and Sump Design\Spatial_IM\GIS\MXD\Phase 4000\1211270062-4000-01_surficial.mxd

LEGEND

- MONITORING WELL LOCATION
- PROPOSED ROAD
- WATERCOURSE
- RAILWAY
- ROAD
- TOPOGRAPHIC CONTOUR (m)
- PROPOSED STORMWATER MANAGEMENT POND
- PROPOSED DEVELOPMENT LANDS
- RICHMOND VILLAGE BOUNDARY

RevisedSurficialGeology

- 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*
- APPROXIMATE BOUNDARY OF LONG-TERM DRAINAGE MODEL DOMAIN
- 100 YEAR STORM EVENT MODEL DOMAIN

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LIMITED REPORT NO. 12-1127-0062 PHASE 4000

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

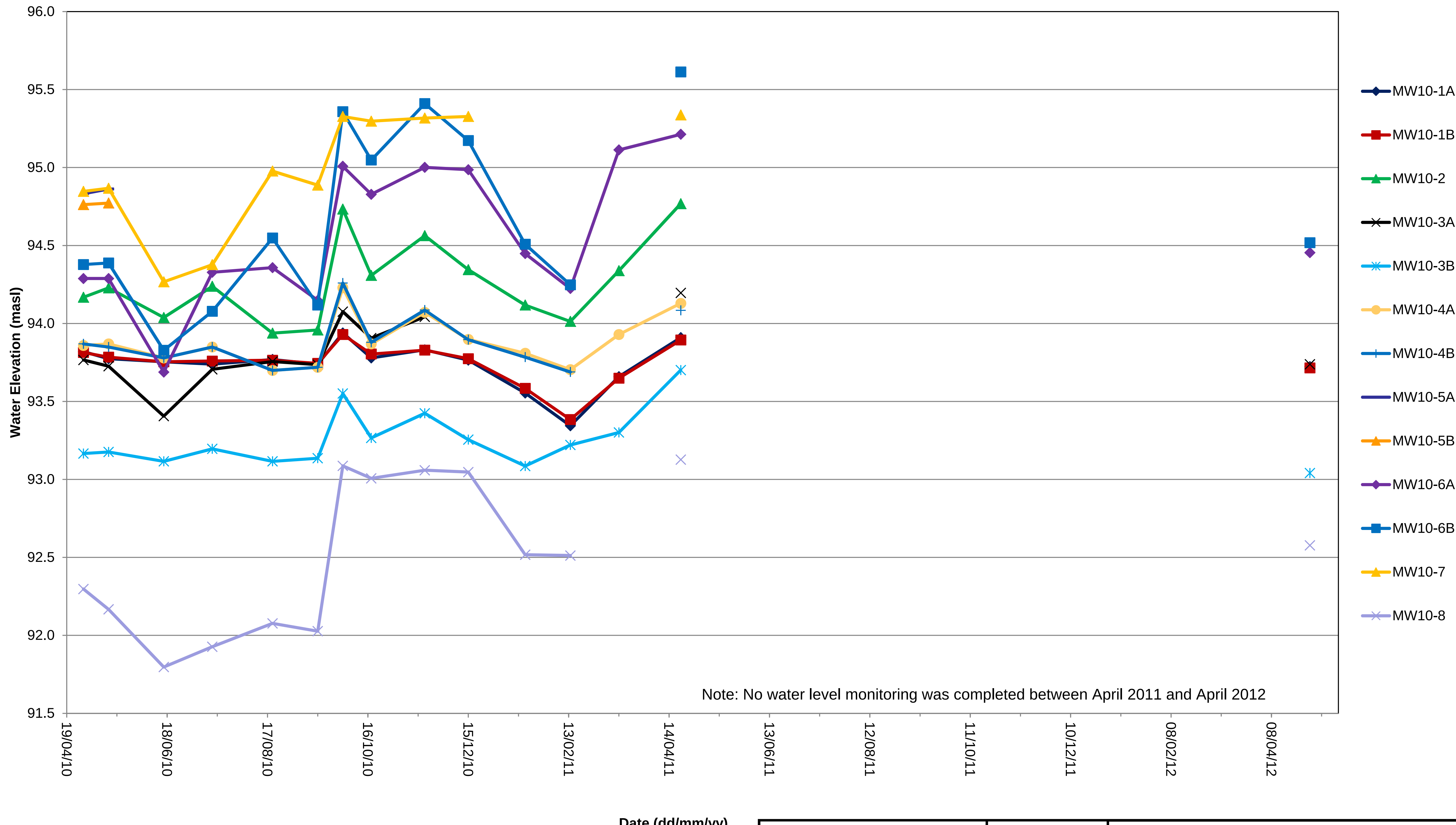
RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

TITLE

SITE PLAN

Golder Associates
Ottawa, Ontario

PROJECT No. 12-1127-0062			SCALE AS SHOWN	REV. 0.0
DESIGN	NB	May 2012	FIGURE 1	
GIS	ABD	July 2012		
CHECK	MIB	June 2013		
REVIEW	BTB	June 2013		

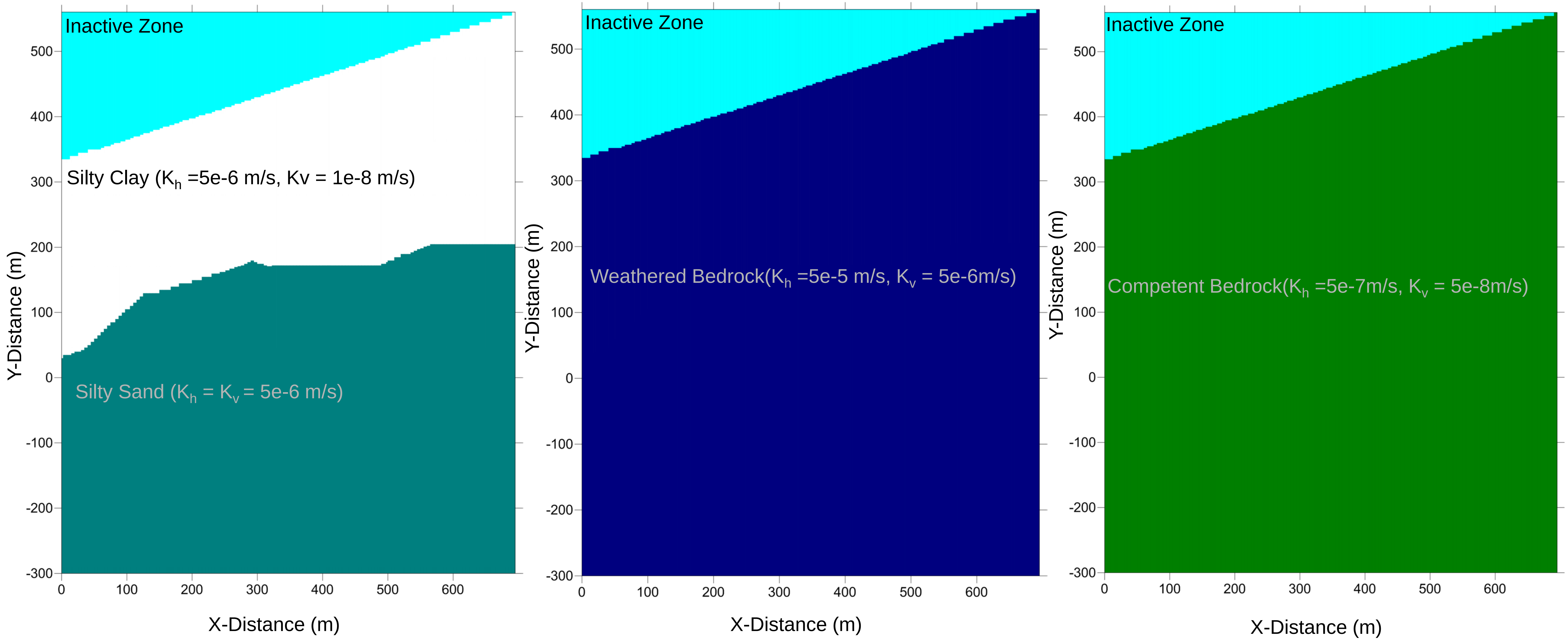


Golder Associates

SCALE	AS SHOWN
DATE	JUNE 2013
DRAWING	MIB
REVIEW	BTB
PROJECT NO.	12-1127-0062

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

Observed Groundwater Elevations **2**



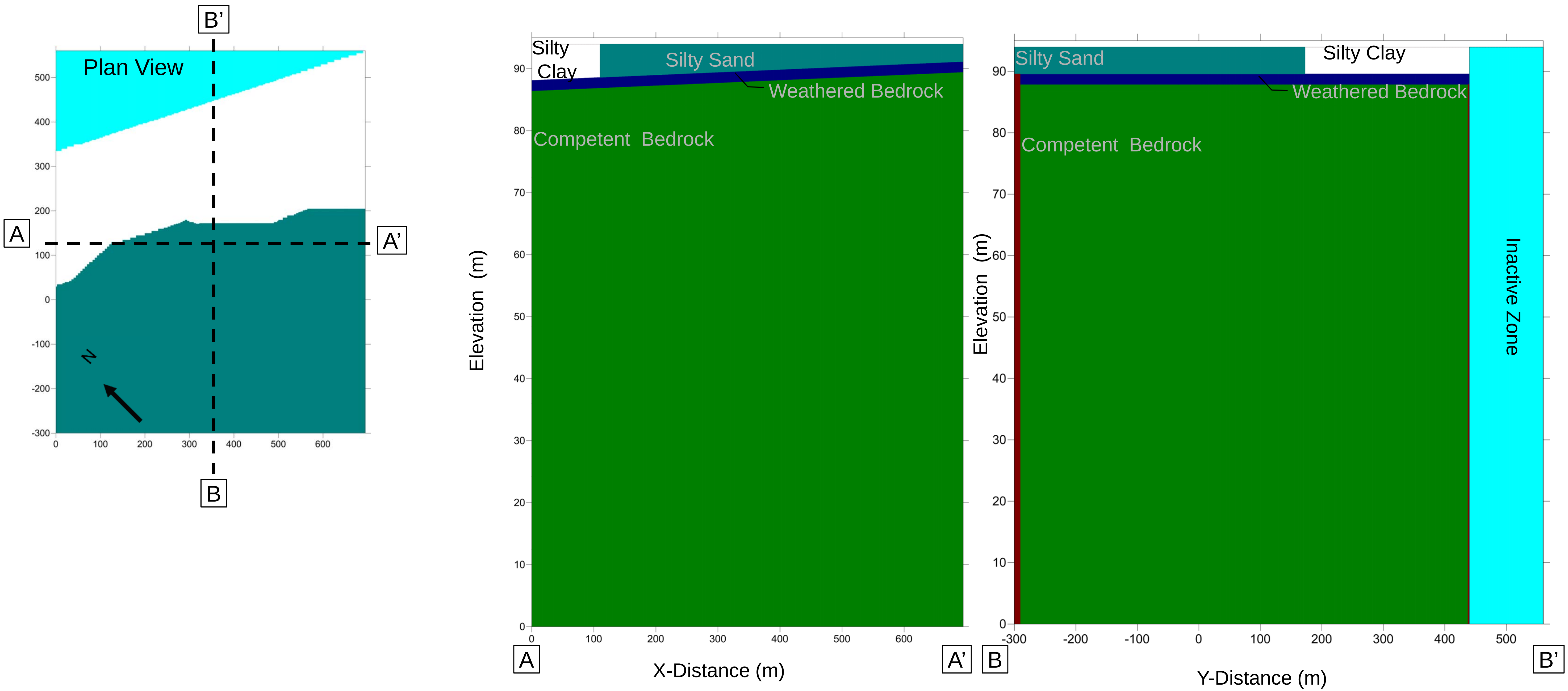
Layers 1-8 (Overburden) Layer 9 Layer 10 - 12

SCALE	AS SHOWN
DATE	JUNE 2013
DRAWING	MIB
REVIEW	BTB
PROJECT NO.	12-1127-0062

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

Hydraulic Conductivity Distribution in Plan View (Long Term Model)

3



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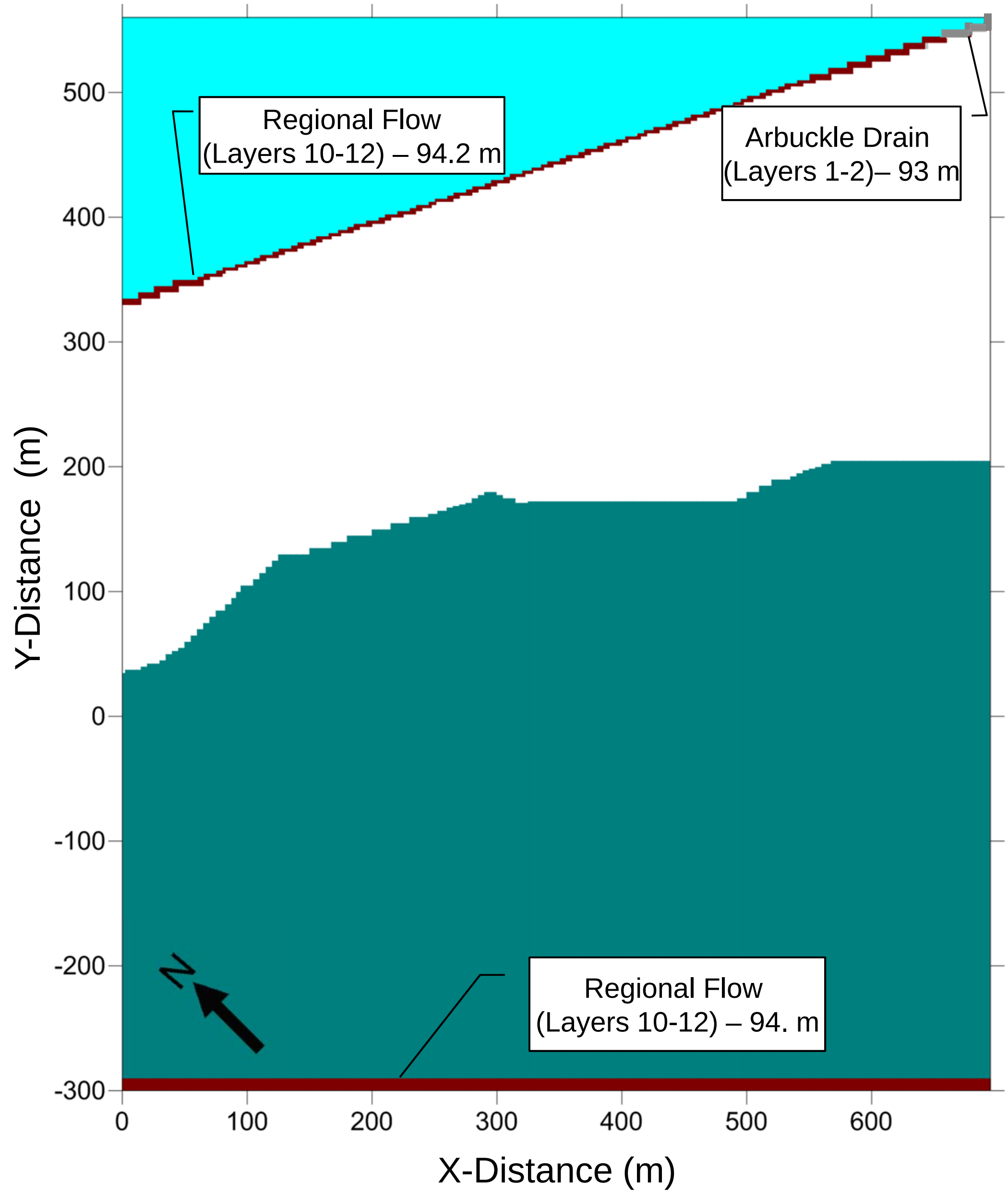
SCALE	AS SHOWN
DATE	JUNE 2013
DRAWING	MIB
REVIEW	BTB
PROJECT NO.	12-1127-0062

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

Hydraulic Conductivity Distribution in Cross-Section (Long Term Model)

4

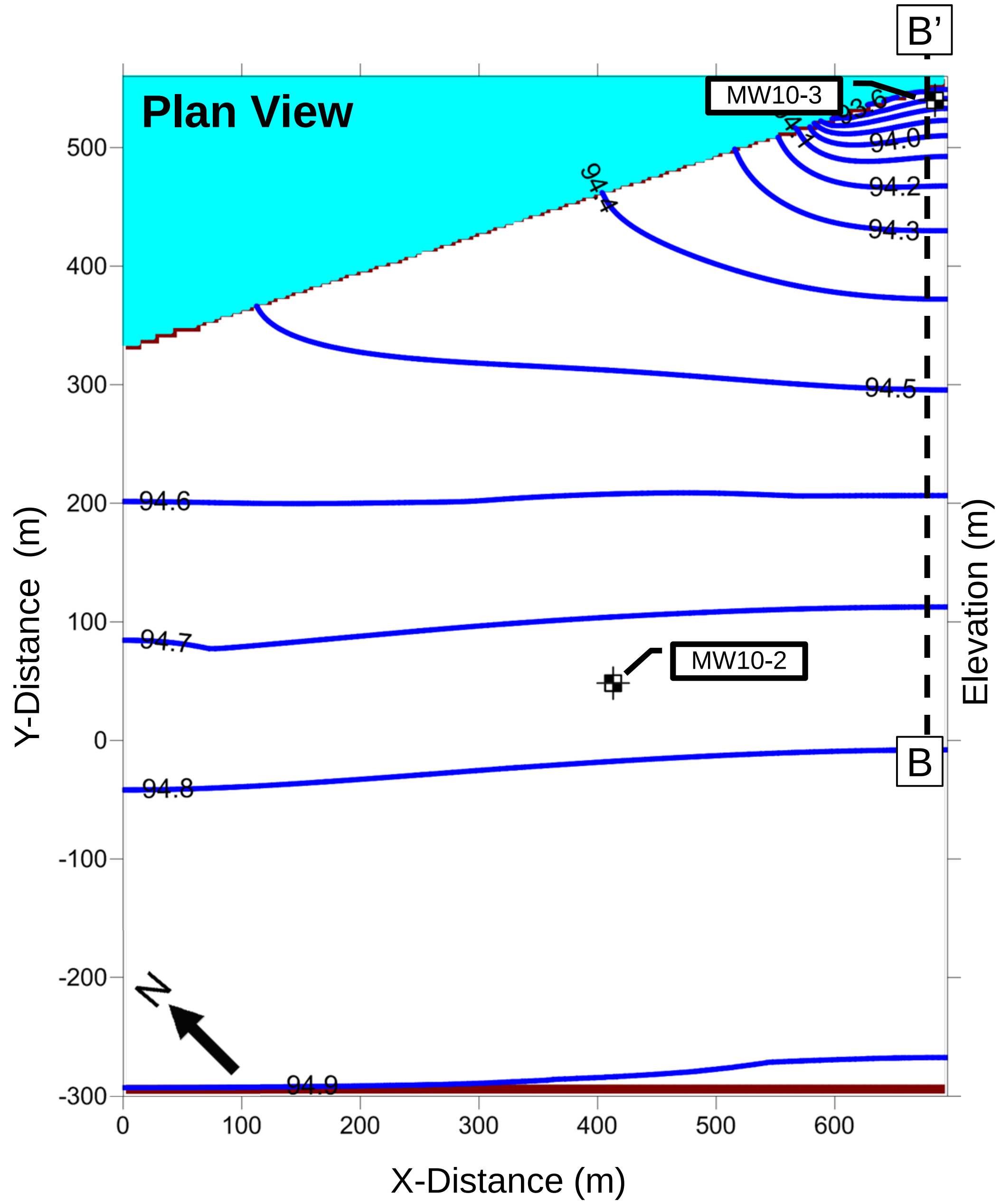


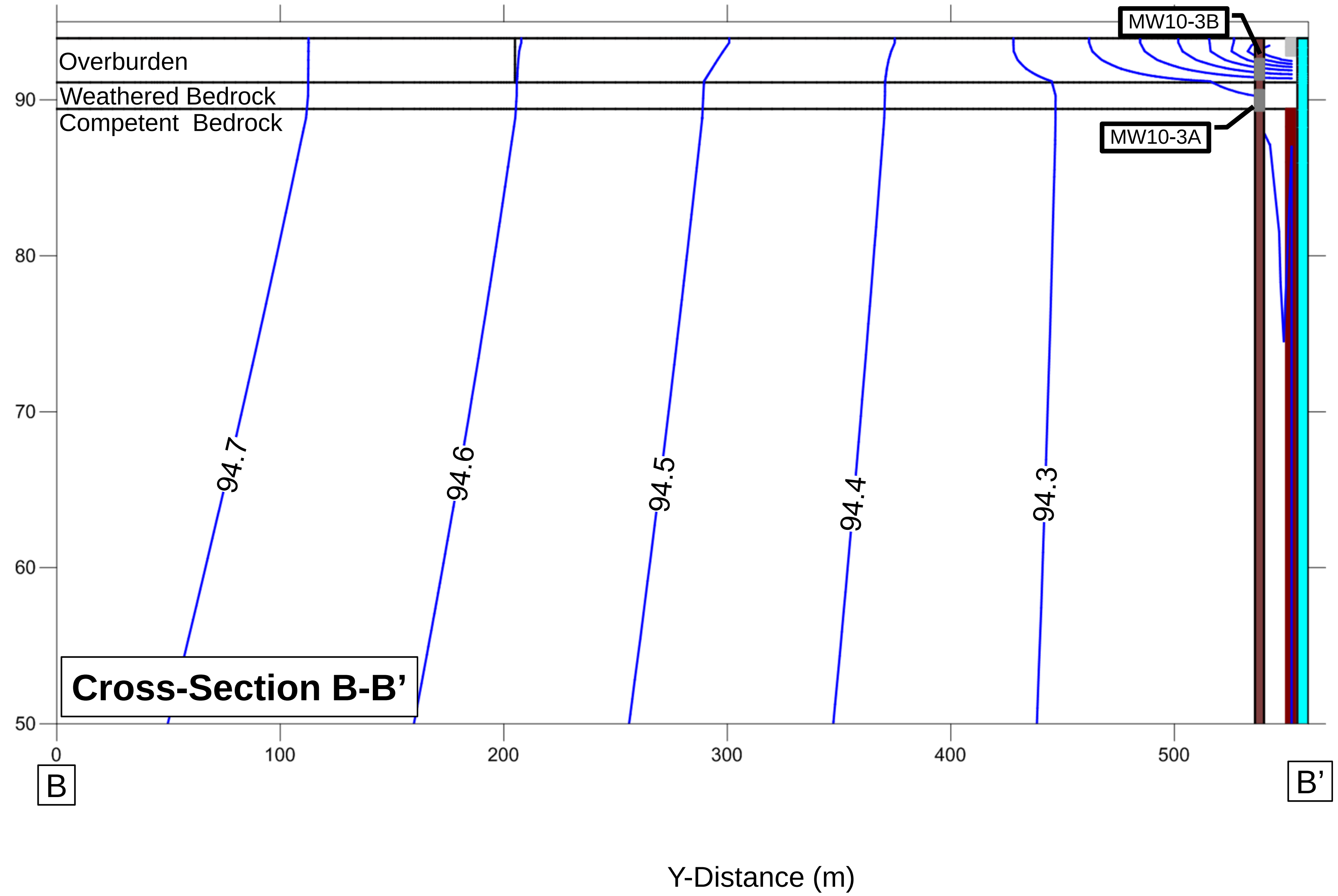
Notes:

- Constant Head Boundary
- Drain Boundary

Regional flow boundaries (constant heads) assigned through bedrock layers

Golder Associates	SCALE AS SHOWN	RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE
	DATE JUNE 2013	
	DRAWING MIB	
	REVIEW BTB	Boundary Conditions for Predevelopment Conditions (Long Term Model) — 5
	PROJECT NO. 12-1127-0062	





Monitoring Well	Observed Head (m)	Simulated Head (m)
MW10-2	94.8	94.8
MW10-3A	94.2	94.2
MW10-3B	93.7	93.9

Notes:

- Groundwater Elevation Contours (m) – 0.1 m Contour Interval
- Constant Head Boundary
- Drain Boundary

Cross-Section elevation truncated at 50 m, model bottom elevation at 0 m

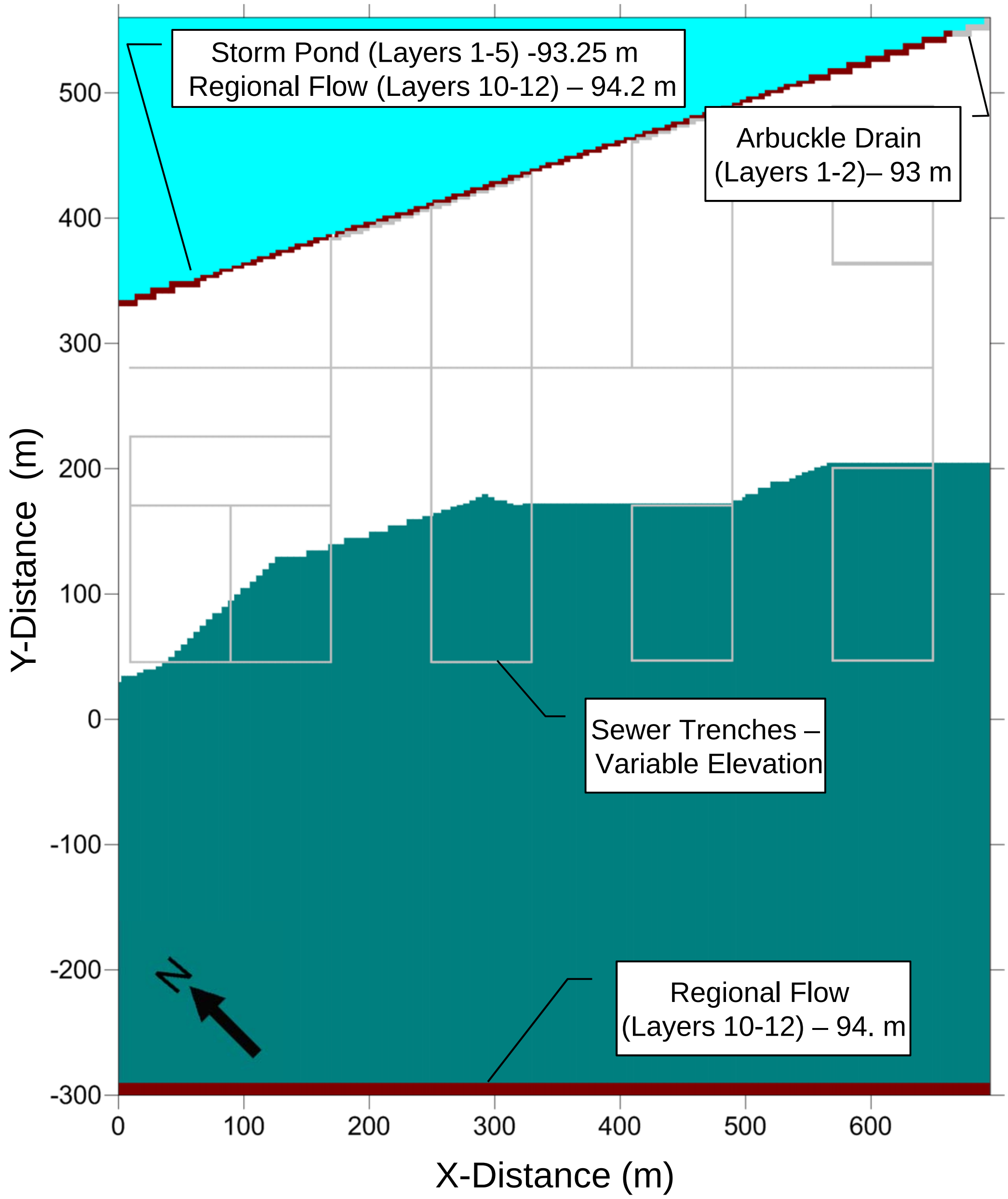
Golder Associates

SCALE	AS SHOWN
DATE	JUNE 2013
DRAWING	MIB
REVIEW	BTB
PROJECT NO.	12-1127-0062

RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE

Simulated Groundwater Elevations for Predevelopment Conditions (Long Term Model)

6

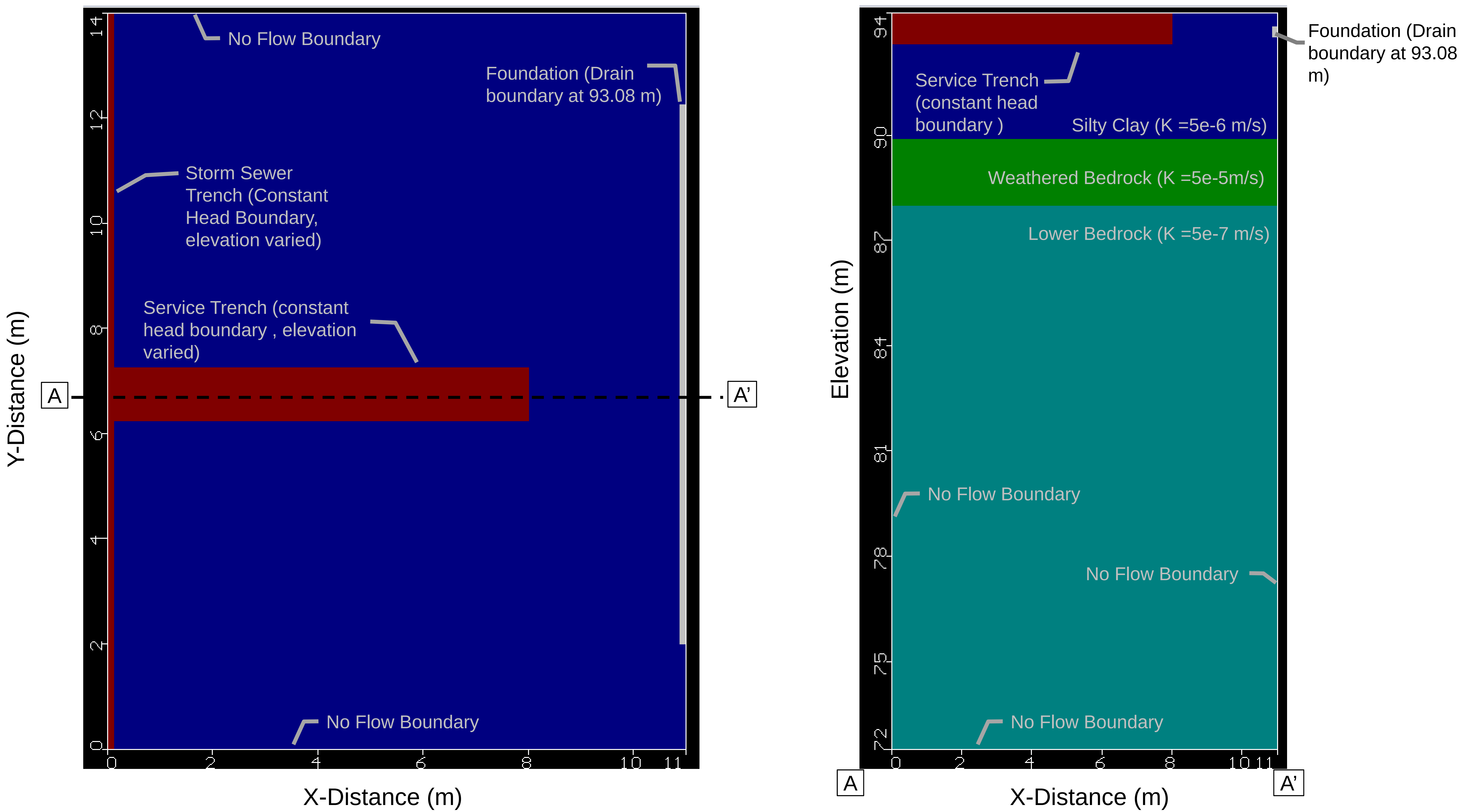


Notes:

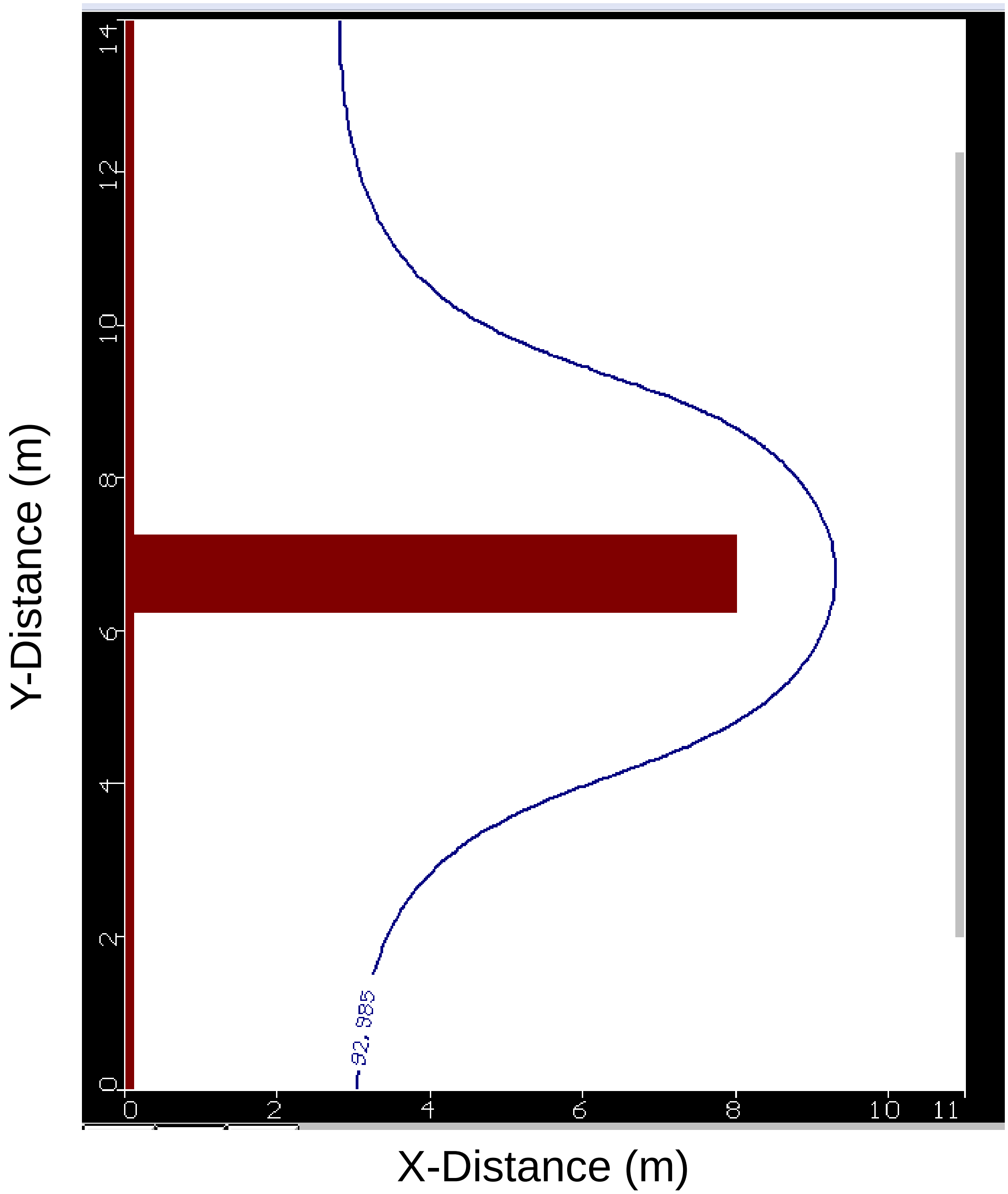
- Constant Head Boundary
- Drain Boundary

Storm sewer drain boundary elevation assigned based on proposed storm trunk profiles (DSEL project 11-486, Figures 5,6, and 7 (April, 2012))

Golder Associates	SCALE AS SHOWN	RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE
	DATE JUNE 2013	
	DRAWING MIB	
	REVIEW BTB	Boundary Conditions for Post Development Conditions (Long Term Model) — 7
	PROJECT NO. 12-1127-0062	



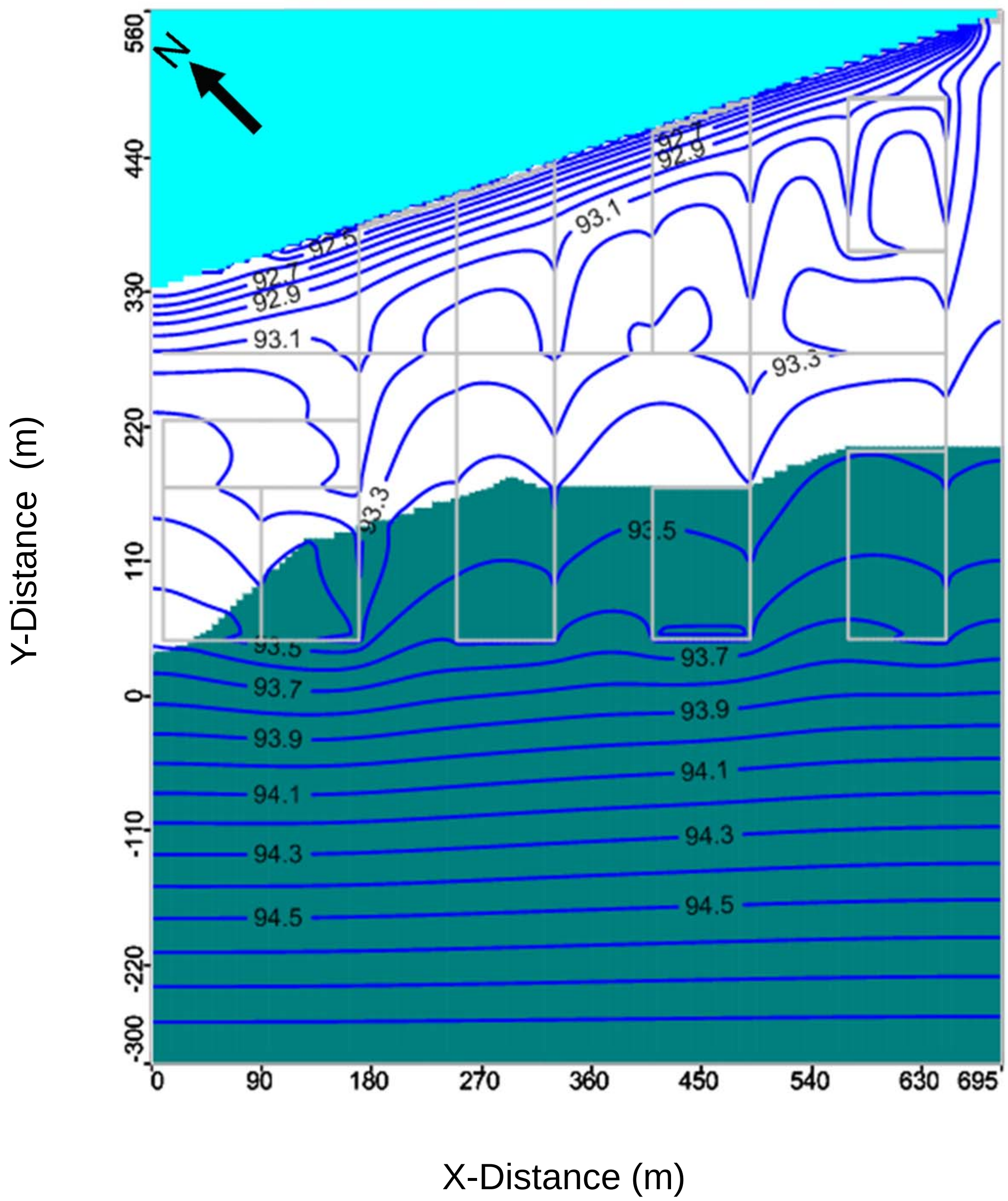
Golder Associates	SCALE AS SHOWN	**RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE**
	DATE JUNE 2013	
	DRAWING MIB	
	REVIEW BTB	Hydraulic Conductivity Distribution and Boundary Conditions (100-Year Storm Model)
	PROJECT NO. 12-1127-0062	**8**



Notes:

- Groundwater Elevation Contours (m) – 0.005 m Contour Interval
- Constant Head Boundary
- Drain Boundary

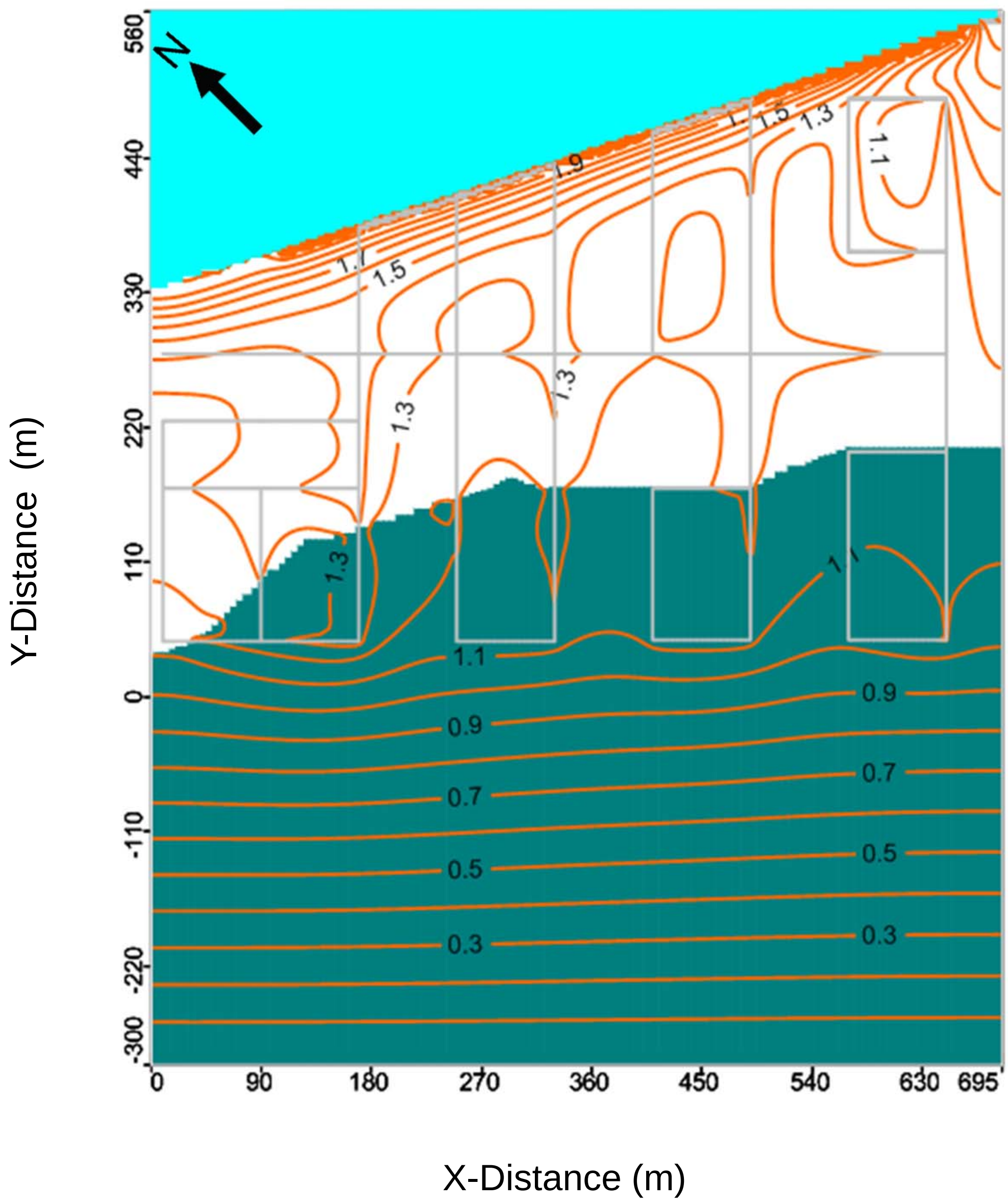
Golder Associates	SCALE AS SHOWN	**RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE**
	DATE JUNE 2013	
	DRAWING MIB	
	REVIEW BTB	Initial Groundwater Elevations (100-year Storm Model)
	PROJECT NO. 12-1127-0062	**9**



Notes:

- Groundwater Elevation Contours (m) – 0.1 m Contour Interval
- Constant Head Boundary
- Drain Boundary

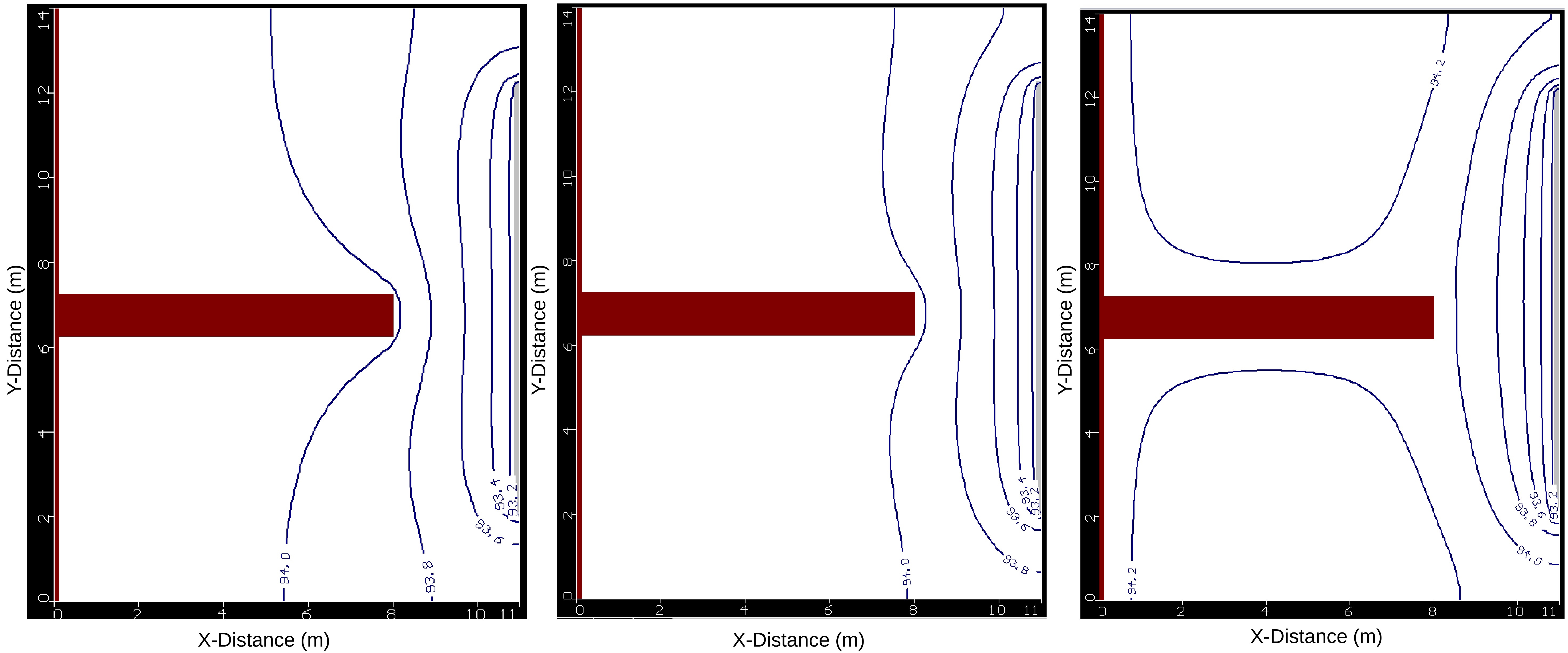
Golder Associates	SCALE AS SHOWN	RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE
	DATE JUNE 2013	
	DRAWING MIB	
	REVIEW BTB	Simulated Steady State Groundwater Elevations (Long Term Model) — 10
	PROJECT NO. 12-1127-0062	



Notes:

- Drawdown Contours (m) – 0.1 m Contour Interval
- Constant Head Boundary
- Drain Boundary

Golder Associates	SCALE AS SHOWN	RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE
	DATE JUNE 2013	
	DRAWING MIB	
	REVIEW BTB	Simulated Steady State Hydraulic Head Drawdown (Long Term Model) — 11
	PROJECT NO. 12-1127-0062	



Recharge = 2000 mm/year

Recharge = 4000 mm/year

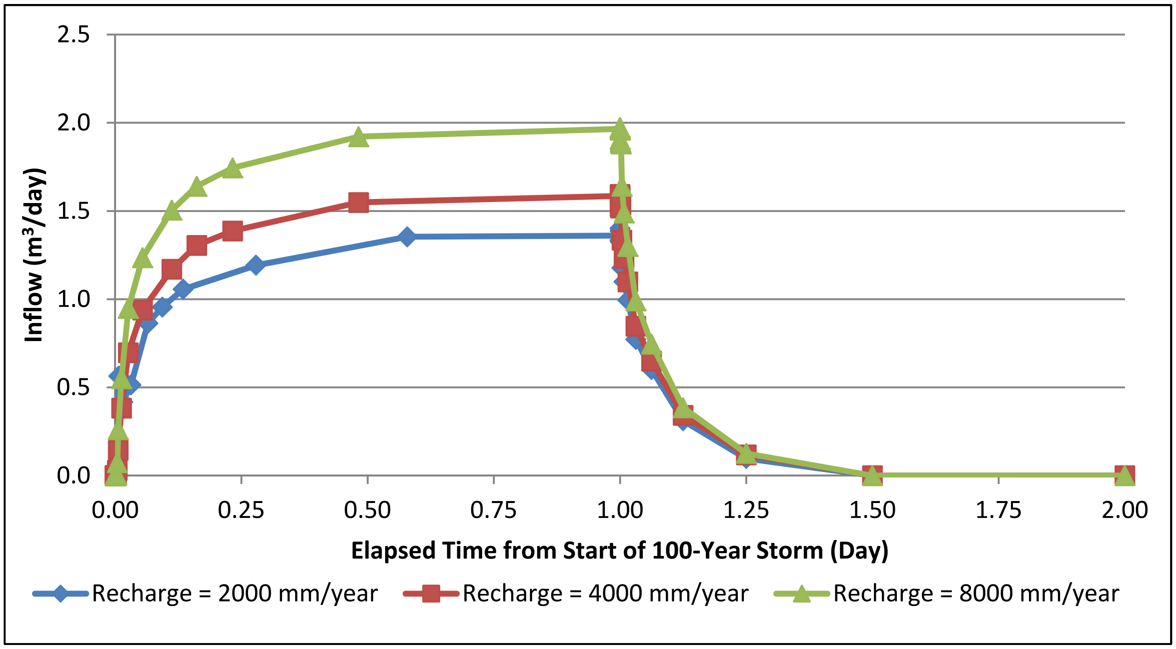
Recharge = 8000 mm/year

Notes:

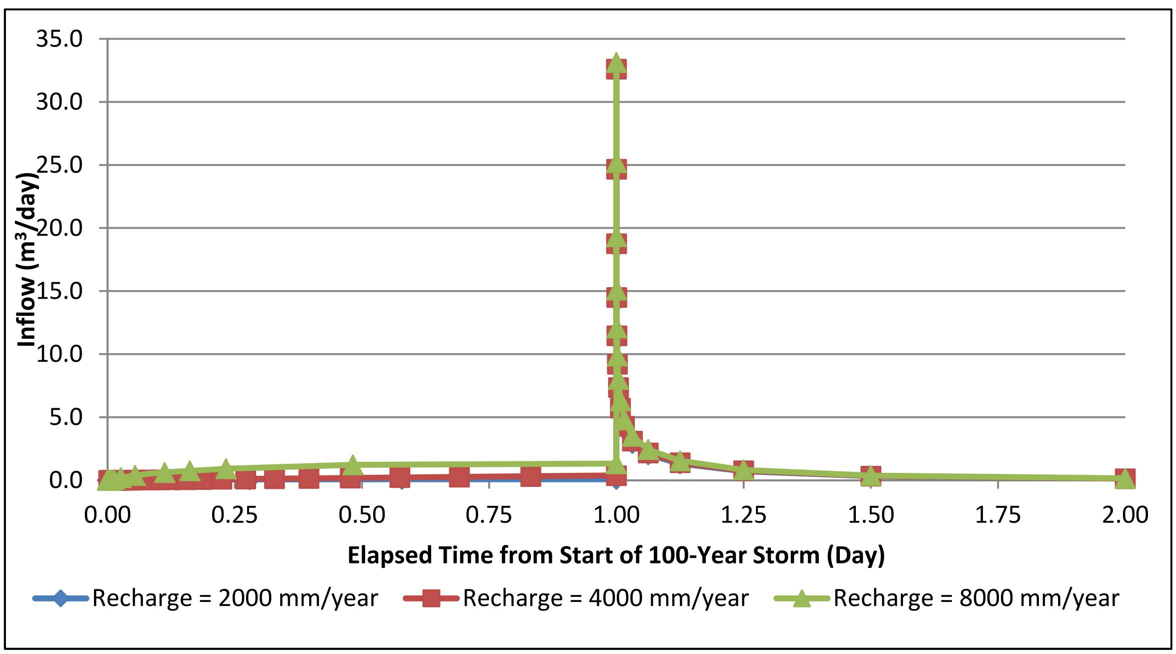
- Groundwater Elevation Contours (m) – 0.2 m Contour Intervals
- Constant Head Boundary
- Drain Boundary

Golder Associates	SCALE AS SHOWN DATE JUNE 2013 DRAWING MIB REVIEW BTB PROJECT NO. 12-1127-0062	**RICHMOND VILLAGE ASSESSMENT OF SUBSURFACE DRAINAGE** Simulated Groundwater Elevation Contours Following 24 hour 100-year Storm	**12**

Flows to Drain Boundary



Flows to Constant Head Boundary



Golder Associates

SCALE	NA
DATE	JUNE 2013
DRAWING	MIB
REVIEW	BTB
PROJECT NO.	12-1127-0062

RICHMOND VILLAGE
ASSESSMENT OF SUBSURFACE DRAINAGE

Simulated Flows to Boundaries
(100-year Storm Model)

13