Appendix D

Groundwater Modelling

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

Final Report

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Executive Summary

This report documents the development and application of a three-dimensional, numerical groundwater flow model of the area influenced by pumping from the Region of Waterloo municipal supply wells in Cambridge, Ontario. The model has been calibrated for multiple sets of calibration targets and has been tested extensively. The model is applied to support the assessment of the potential effects of developing additional groundwater supplies in the eastern portion of the model area, referred to as Cambridge East. The model has been applied to provide quantitative estimates of the potential effects of alternative schemes for groundwater takings from the three well fields in eastern portion of Cambridge: the Pinebush Well Field, the Clemens Mill Well Field, and the Shade's Mills Well Field. It is anticipated that the model will also provide a framework for the interpretation of hydrogeologic data from the ongoing Region of Waterloo groundwater monitoring program and the inference of data gaps.

The Cambridge East groundwater modelling has been conducted using approaches deliberately designed to limit the uncertainties in the model predictions. However, when assessing the results of predictive simulations it is important to bear in mind that uncertainties in the analyses will always persist. Groundwater models are deliberately simplified representations of complex natural systems. Even in portions of the Cambridge study area where the characterization is relatively complete, it is not feasible to capture all the details of the surface water and groundwater systems in the numerical model. The data that have been used to constrain the analyses come from a variety of sources of differing reliability and not all of the available monitoring data can be interpreted definitively. In light of these limitations, it is more appropriate to consider the predicted potential effects of alternative schemes for groundwater takings as hypotheses regarding what changes *might* occur, rather than as definitive statements regarding what *will* occur. These hypotheses would be tested through the ongoing collection and interpretation of hydrogeologic data.

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1 INTRODUCTION

The Region of Waterloo retained S.S. Papadopulos & Associates, Inc. and Matrix Solutions to develop a three-dimensional, numerical groundwater flow model of the Cambridge area. The model development, calibration, testing and application has been conducted to support the assessment of the potential effects of developing additional groundwater supplies in the eastern portion of the model area, referred to as Cambridge East. The model has been applied to provide quantitative estimates of the potential effects of alternative schemes for groundwater takings from the three well fields in eastern portion of Cambridge: the Pinebush Well Field, the Clemens Mill Well Field, and the Shade's Mills Well Field. It is anticipated that the model will also provide a framework for the interpretation of hydrogeologic data from the ongoing Region of Waterloo groundwater monitoring program and the inference of data gaps.

The groundwater model synthesizes what is known about the overburden and bedrock groundwater systems of the Cambridge East area. Emphasis is placed on the bedrock, as this is the primary source of municipal supplies. The model represents a first analysis of the area based on the updated stratigraphic interpretation of the area developed by the Ontario Geological Survey (Brunton, 2008, 2009; Brunton and Brintnell, 2011).

The Cambridge East groundwater model is calibrated to match average conditions in 2003, and to match the results of two extensive pumping tests. The predictive capabilities of the model are assessed by comparing model results against the observed long-term effects of the increase in pumping at the Clemens Mill Well Field that occurred in the mid-1990s, and by comparing model results against the observed response to pumping at a new test well.

The groundwater model is applied to assess the potential changes in water levels and in the components of the water balance that may arise for alternative pumping scenarios. The potential effects that are the focus of the analyses are long-term declines in water levels in the shallow and deep subsurface, and changes in the components of the overall water balance within the zone of influence of pumping. The changes are assessed with respect to the average conditions observed in 2007.

This report on the development and application of the Cambridge East groundwater model is divided into five main sections:

- Development of the groundwater flow model, including the representation of the hydrostratigraphy and the specification of model properties, boundary conditions and sources and sinks;
- Calibration of the groundwater flow model;
- Testing of the groundwater flow model;
- Simulation of Cambridge East Environmental Assessment scenarios; and
- Perspective on the results of the scenario simulations.

2 DEVELOPMENT OF THE GROUNDWATER FLOW MODEL

2.1 Modelling Approach

The groundwater model developed for the Cambridge East area represents the first attempt to simulate groundwater flow within the Cambridge area based on the updated interpretation of the bedrock stratigraphy in southern Ontario developed by F.R. Brunton and colleagues of the Ontario Geological Survey (Brunton, 2008, 2009; Brunton and Brintnell, 2011). The structure of the bedrock geology in the Cambridge area is complex; however, the Brunton geologic framework provides a basis for understanding the variations in the properties of the bedrock and the controls those variations exert on groundwater flow. The development of a groundwater model based on the Brunton geologic framework is an essential step for understanding current conditions and interpreting new data.

The groundwater model of Cambridge East that has been developed is relatively complex. This complexity is warranted for the present applications, as the model must support assessments of the potential effects of changes in pumping at multiple well fields. The Cambridge well fields generally contain more than one production well, and the multiple production wells draw water from shallow and deep flow zones. The model is three-dimensional to support explicit representation of the interface zone between the overburden and the bedrock and the stratigraphic units of the Brunton geologic model. The model covers a relatively large geographical area, so that the model extends to natural hydrologic boundaries, to the extent practical, and accommodates consideration of the potential for interaction between local and more distant well fields. The model is also highly refined, to resolve hydraulic gradients within well fields and allow representation of variations in material properties relatively close to the production wells. The groundwater model is also refined to support demonstrations that the model is capable of matching the observations collected during recent long-term pumping tests conducted with extensive monitoring.

2.2 Model Selection

The comprehensive groundwater flow simulator FEFLOW (v6.2; Diersch, 2014) was selected to develop the numerical groundwater flow model for the Cambridge East EA. FEFLOW has been used in several groundwater modelling studies within or adjacent to the Study Area, including:

- The Guelph-Puslinch Groundwater Protection Study (Golder, 2006);
- The Tier Two Water Budget and Subwatershed Stress Assessment for the Grand River Watershed (AquaResource, 2009a);
- The Regional Municipality of Waterloo Groundwater Model Calibration Results (Waterloo Hydrogeologic Inc., 2008); and
- The Regional Municipality of Waterloo Tier Three Water Budget and Local Area Risk Assessment (Matrix Solutions and SSP&A 2014).

FEFLOW is a commercially available, three-dimensional finite element groundwater modelling code. FEFLOW is capable of simulating variably-saturated groundwater flow, and the transport of thermal energy and solutes. FEFLOW has been selected for the present analyses because:

- FEFLOW supports the refinement of a model around specific areas of interest such as pumping wells or rivers, to more precisely simulate observed physical features and follow naturally complex boundary conditions such as the steep slope along the Grand River;
- FEFLOW can represent a complex layer structure that conforms to pronounced variations in the thicknesses of aquifer/aquitard layers;
- FEFLOW is capable of stable simulations of the water table; and
- FEFLOW has been used widely for other applications in the Region of Waterloo. The use of FEFLOW allows for close integration with the groundwater models that are currently being developed to support Source Water Protection studies for the Region of Waterloo and the City of Guelph.

2.3 Model Limits and Discretization

2.3.1 Model Limits

The limits of the Cambridge East model are shown in **Figure 1**. The model limits have been defined to extend beyond the areas of the hydrogeological system that are likely to be affected by pumping from the municipal water supply wells in the Cambridge area. The model covers portions of the City of Cambridge and the surrounding rural areas, and extends northward into the southern portion of the City of Guelph. The model has a maximum width of approximately 26.5 km (west-east) and maximum length of 27 km (north-south); the model has an area of 591 km^2.

The model includes portions of the Central Grand River, Upper Speed River, Mill Creek and Fairchild Creek subwatersheds of the Grand River watershed (AquaResource Inc., 2009). A portion of the western boundary of the model is aligned along the Grand River. The model is bounded to the west by the Cedar Creek Spillway corridor that is occupied by Cedar Creek and Blair/Bechtel Creeks. To the north, the bedrock groundwater flow system terminates along the Niagara Escarpment. It is neither necessary nor practical to extend a model of the Cambridge area that far. As natural flow boundaries are distant to the north, east and southeast the model area, groundwater levels are specified along these boundaries. The levels have been assigned based on the regional scale interpretations of groundwater levels developed for the Regional Municipality of Waterloo Tier Three Assessment (Matrix Solutions and SSP&A, 2014) and the Guelph Tier Three Assessment (Matrix Solutions, in progress). Details regarding the specification of water levels are presented in Section 3.2.5.

The boundaries of the model have been set sufficiently far from Cambridge East municipal water supply wells that they do not influence the simulated groundwater flow conditions under current pumping or for alternative pumping scenarios.

2.3.2 Model Discretization

The first step in solving a groundwater flow problem using the finite element method is to discretize the model area. In the horizontal, the model area is overlain by a network of triangles. FEFLOW simulates groundwater flow by calculating groundwater levels at the vertices of the elements, referred to as *nodes*. The elements and nodes are referred to collectively as the finite element *mesh*. In the vertical, the model is divided into *layers*, with the vertices of the triangles forming prismatic *elements*. The network of elements that define either the top or bottom of a layer is referred to as a *slice*.

Properties are assigned to each element to specify the characteristics of the hydrostratigraphic units (for example, unit thicknesses and hydraulic conductivities). Boundary conditions are assigned to the finite element nodes around the perimeter of the model. Internal sources and sinks are assigned to nodes within the model to represent wells, lakes, streams, and recharge.

The finite element mesh of the Cambridge East model is designed to meet the requirements of the Cambridge East EA, in consideration of the Region's existing municipal supply wells, other water users, and groundwater/surface water interaction. The finite element mesh is refined within areas of the model where groundwater levels are expected to change significantly or where a high resolution of model output is required (for example, near pumping wells). The dimensions of the elements in the central portion of the model mesh range from about 100 m to 150 m. The sizes of the elements are increased gradually to approximately 450 m along the perimeter of the model. The elements are 25 m along rivers and streams. Around municipal and non-municipal pumping wells that extract large volumes of groundwater, element sizes range from 3 m to 5 m.

The model mesh may be refined in the future to examine new points of interest within the model domain. Further refinement is straightforward to implement without affecting the matches to the observations and the predictions of the model that are presented in this report.

3 OVERBURDEN AND BEDROCK HYDROSTRATIGRAPHY OF THE CAMBRIDGE AREA

3.1 Overburden Hydrostratigraphy

In 2007, the Ontario Geological Survey (OGS) delineated the hydrostratigraphy of the overburden within the Region of Waterloo (Region), including the City of Cambridge (Bajc and Shirota, 2007). The OGS hydrostratigraphic model of the overburden characterized the continuity of sands and gravels, regional and local till sheets, and fine-grained silt and clay deposits. The OGS conceptual model included 18 layers, with each layer representing a distinct hydrostratigraphic unit composed of sediments with similar characteristics.

The OGS overburden hydrostratigraphic model has been modified in the Cambridge area to incorporate additional data and characterization studies completed as part of the Region of Waterloo Tier Three Assessment (Blackport Hydrogeology Inc., 2009; Golder Associates, 2009a, 2011b, 2011c; Stantec Consulting Ltd, 2009a, 2009b, 2009c, 2009d, 2011a, 2011b). The overburden hydrostratigraphy has also been modified to incorporate additional characterization at the landfill sites within the Region (Golder Associates, 2008b), and as part of this EA and the broader Integrated Urban System Groundwater Supply Optimization and Expansion Project (IUS Project; Golder Associates, 2008a, 2009b, 2009c, 2009d). As part of these studies, new cross-sections were drawn and interpreted across the Cambridge study area. Four additional cross-sections have been developed in the vicinity of Puslinch Lake to improve the characterization of shallow conditions around and beneath the lake (AquaResource Inc., 2010a). These sections include information provided by Harden Environmental (1999) to establish the basal contact of organic sediment at the base of Puslinch Lake.

The overburden hydrostratigraphy for the Cambridge East model is adapted from AquaResource Inc. (2010a). The overburden hydrostratigraphic units are listed on **Table 3-1** and shown schematically in **Figure 2a**. Four of the units Bajc and Shirota (2007) identified are either absent in the Cambridge area or present in only relatively small areas or have minimal thicknesses. In particular, the Cambridge model excludes four OGS units: ATB2, AFB3, AFC1, ATE1 and ATG1. The overburden units that are present in the Cambridge area as discussed in more detail below.

Table 3-1: Overburden Hydrostratigraphic Units in the Cambridge area (AquaResource Inc., 2010a)

Hydrostratigraphic Unit	Hydrostratigraphic Unit Description
1	Puslinch Lake Sediments, Whittlesey Clay and Whittlesey Sand
2	Wentworth Till (ATA2; *Poor Aquitard*)
3	Grand River Outwash (AFA2; *Aquifer*)
4	Port Stanley Till (ATB1; *Aquitard*)
5	Waterloo Moraine Stratified Sediment Equivalents (AFB1, AFB2; *Aquifer*)
6	Maryhill and Catfish Creek Tills (ATC1, ATC2; *Aquitard*)
7	Pre-Catfish and Pre-Canning Outwash (AFD1, AFF1; *Aquifer*)

According to the naming convention of Bajc and Shirota (2007), the first two letters of the label of a hydrostratigraphic unit in the Region of Waterloo identify whether the unit is interpreted as an aquitard (AT) or an aquifer (AF), while the latter two characters correspond to the sequence of the units, with A (and 1) denoting the youngest grouped sequence and F (and 3) the oldest.

The uppermost hydrostratigraphic unit represents recent organic sediments in Puslinch Lake, Glacial Lake Whittlesey Clay and Sand deposits, and the Wentworth Till. Puslinch Lake sediments are only present beneath Puslinch Lake, and Glacial Lake Whittlesey sediments (ATA1 and AFA1) and Wentworth Till deposits (ATA2) are only mapped at surface in the southeastern half of the Study Area (OGS, 2010).

Puslinch Lake is a kettle lake that formed when a large buried block of glacial ice melted following ice retreat from the Study Area. Sediments deposited on top of the ice block collapsed to occupy the underlying void space, forming the lake depression that was subsequently filled with water. Since its formation, approximately 12 m of organic sediments have been deposited in the lake (Harden Environmental, 1999). The lacustrine sediments are described as an unconsolidated, viscous to gelatinous, fine-grained deposit with very low hydraulic conductivity (Denhoed, 2010).

Whittlesey Clay and Whittlesey Sand (ATA1) deposits were laid down approximately 13,000 years ago when glacial meltwater ponded in front of the Erie and Huron lobes and inundated the present day Lake Erie and Huron basins, as well as much of the southern half of southwestern Ontario, forming Glacial Lake Whittlesey (Dreimanis, 1966; Barnett, 1979). Fine-grained sediment settled from suspension in the quiet water glaciolacustrine environment (Lake Whittlesey Clay) as water levels rose in response to the advance of Halton Till ice during the Port Huron Stade (Barnett, 1979). Prior to the advance of Halton Till ice, the lake level was lower, and beach and shoreline led to the deposition of sand along the southeastern flanks of the Paris and Galt Moraines (Whittlesey Sand). Whittlesey Sand is very fine- to coarse-grained sand, with lesser silts and clays (Barnett, 1978, 1979; Bajc and Shirota, 2007).

The Wentworth Till (ATA2) originated from subglacial and debris flow conditions near the ice margin of the Erie-Ontario lobe. The deposits were reworked by meltwater along the Paris and Galt Moraines (Russell *et al*., 2009). Debris flow tills record the movement of sediment down a slope, and consist of poorly-sorted deposits that incorporate lenses of the underlying sediment (Boulton, 1972; King *et al*., 1998; Sverre Laberg and Vorren, 2008). Lenses and discontinuous beds of stratified sand and gravel lie within the Paris and Galt Moraines, suggesting the Wentworth Till was deposited at the same time as glaciofluvial activity (Golder, 2011b; Stantec, 2011a, 2011b). The Wentworth Till is described as sandy silt till, with a stone content ranging from 35% to 65% (Golder, 2011b; Russell *et al*., 2009; Stantec, 2011a, 2011b).

The Grand River Outwash (AFA2) unit consists of coarse-grained sand and gravel mapped west of, and between, the Paris and Galt Moraines, within the Speed and Grand River valleys, and beneath the Wentworth Till of the Paris Moraines (OGS, 2010; Golder, 2011b; 2011c; Russell *et al.*, 2009; Stantec, 2011a, 2011b). These sediments were laid down by proglalcial meltwater streams in a braided river system (Bajc and Karrow, 2004). Lenses and discontinuous beds of fine-grained sediment observed in the stratified sand and gravel reflect slack water conditions during low-flow periods. Deposits are thickest along the Grand River system in front (west) of the Paris and Galt Moraines.

The Port Stanley Till (ATB1) was deposited by the Erie-Ontario lobe during the Port Bruce Stade, approximately 15,000 years ago (Sharpe, 1978; Karrow, 1984; Fulton *et al*., 1986). It was deposited by a mixture of debris flows, settling of suspended sediment and ice-rafted debris at the base of a glacial lake, as well as subglacially beneath the ice lobe (Barnett, 1978, 1987, 1998; Bajc and Hunter, 2006; Russell *et al*., 2009). The Port Stanley Till consists of silty to sandy silt till with interbeds of laminated silts and clays. The Port Stanley Till is present west of, and beneath, the Paris and Galt Moraines and it reaches a maximum thickness of approximately 35 m under the Pinebush Well Field (Golder, 2011b; Stantec, 2011a, 2011b).

The Waterloo Moraine Stratified Sediment Equivalents (AFB1, AFB2) are characterized as stratified very fine to very coarse-grained sands and granule to cobble gravels, as well as minor lenses and discontinuous beds of silts and clays (Bajc and Shirota, 2007). This unit likely records a westward prograding subaquatic fan into a proglacial lake and/or glaciofluvial activity (Bajc and Karrow, 2004; Bajc and Shirota, 2007).

The Maryhill Till and Catfish Creek Till (ATC1, ATC2) are silt and sand-rich till units, respectively, that may include interbeds of fine-grained glaciolacustrine sediments associated with the deposition of the Maryhill Till. The Maryhill Till is a clast-poor (less than 2%), clay-rich till that was deposited by the Erie-Ontario lobe (Rajakaruna, 1994; Karrow, 1984; Karrow and Paloschi, 1996; Bajc and Karrow, 2004). Interbedded and overlying fine-grained material may be suspended sediments that settled under quiet water glaciolacustrine conditions (Bajc and Shirota, 2007). The Catfish Creek Till was deposited subglacially when glacial ice advanced from the northeast through the Study Area (Karrow and Paloschi, 1996; Kryszkowski and Karrow, 2001). It is an overconsolidated, stone-rich, sandy to silty till (Karrow, 1974; Karrow, 1988; Barnett, 1992). Unlike other areas in the Region of Waterloo, interbeds of coarse-grained glaciofluvial sediments (AFC1 in Bajc and Shirota (2007)) are not observed in the Catfish Creek Till in the study area.

The Basal Sands (Pre-Catfish Outwash; AFD1) unit consists of coarse-grained outwash sands and gravels deposited by glacial meltwater in front of an advancing ice lobe (Bajc and Shirota, 2007). This unit is discontinuous and rests on the weathered bedrock surface in the study area. This unit is the equivalent of the Pre-Catfish and Pre-Canning Outwash units (AFD1 and AFF1) described by Bajc and Shirota (2007).

As indicated previously, the coverage the Bajc and Shirota overburden hydrostratigraphic model is limited to the Region of Waterloo. The groundwater model extends beyond these political limits. High-resolution analyses are not critical in the areas beyond the limits of the Region of Waterloo; in these areas a simplified approach has been adopted for the present analyses. In these areas, the total thickness of the overburden (calculated as the difference between the ground surface and the top of bedrock) is divided equally into six model layers, as shown in **Figure 2b**. The hydraulic conductivity values applied to those layers is based on the lithologic information reported at borehole locations. Additional information on the assignment of hydraulic conductivity values is presented in **Section 4.1**.

3.2 Bedrock Hydrostratigraphy

The major municipal production wells in the Cambridge East area pump from the bedrock. The groundwater model developed for the Cambridge East area is a first attempt to simulate groundwater flow within the Cambridge area based on the updated interpretation of the bedrock stratigraphy in southern Ontario developed by F.R. Brunton and colleagues of the Ontario Geological Survey (Brunton, 2008, 2009; Brunton and Brintnell, 2011).

Brunton and co-workers have identified eight bedrock hydrostratigraphic units in the study area. The units are listed on **Table 3-2**, with general descriptions presented below. Details of the development of the bedrock hydrostratigraphic model in the Cambridge area are presented in a stand-alone report included here as **Appendix A** (SSP&A, 2012).

Table 3-2: Bedrock Hydrostratigraphic Units in the Cambridge area (Brunton, 2009)

Hydrostratigraphic Unit	General Hydrostratigraphic Character
Guelph Formation	Aquifer
Reformatory Quarry Member of the Eramosa Formation	Aquifer/Aquitard
Vinemount Member of the Eramosa Formation	Aquitard
Goat Island Formation	Aquifer
Upper Gasport Formation	Aquifer
Middle Gasport Formation	Major Aquifer
Lower Gasport/ Rochester/ Irondequoit/ Rockway/ Merritton Formations	Aquifer/Aquitard
Cabot Head Formation	Aquitard

The Guelph Formation consists of medium to thickly-bedded crinoidal grainstones and wackestones (limestone) with lesser reefal complexes (Brunton, 2008). This formation is the uppermost bedrock unit in the eastern portions of the Region, including the Cambridge area.

Brunton and co-workers have revised the interpretation of the Eramosa unit. In particular, they have indicated that the unit should be designated a formation (Brunton, 2008, 2009; Brunton and Brintnell, 2011). They have identified two members within the Eramosa Formation, the upper Reformatory Quarry, and the lower Vinemount Member. The Reformatory Quarry is a dolostone unit about 5 m thick, with a seismite marker bed traced from Niagara to the Bruce Peninsula (Brunton, 2008). The Vinemount Member averages 10 m in thickness in the Guelph area and pinches out beneath Cambridge. The Vinemount Member consists of thinly bedded, finely crystalline dolostone with shaley beds that give off a distinctive petroliferous odour when broken (Brunton, 2008). This dark grey to black dolostone unit has been commonly mapped in studies in southern Ontario as the "black shale" Eramosa Member of the Guelph Formation (Johnson *et al.*, 1992).

The Goat Island Formation consists of two members, the upper Ancaster Member and lower Niagara Falls Member (Brunton, 2008). The Ancaster Member is a finely crystalline dolostone and its thickness varies depending on the thickness of the underlying Gasport Formation (Brunton, 2008). The Niagara Falls Member is a finely crystalline crinoidal grainstone with small reef mounds. The Niagara Falls Member ranges in thickness from a few metres up to 10 m (Brunton, 2008). In general, areas mapped with a thicker Goat Island Formation are interpreted to have a thicker Eramosa Formation and a more prominent Vinemount Member (Brunton, 2008).

The Gasport Formation is a cross-bedded crinoidal grainstone-packstone with reef mounds (Brunton, 2008). The Gasport Formation varies in thickness in the Cambridge area from 25 m to over 70 m, depending on the accommodation space that was available to form reef mound complexes (Brunton, 2008). The reef mounds that contain crinoidal grainstones and coquina beds make some areas of the Gasport Formation highly transmissive. In the Guelph area, the middle portion of the Gasport Formation has previously been referred to as the production zone of the Amabel Formation (Brunton *et al.*, 2007).

In the Cambridge area there are four thin units between the Gasport Formation and the Cabot Head Formation. Only a small thickness of the lower portion of the Rochester Formation is present in the study area. The Rochester Formation is a mixed carbonate-shale unit that pinches out north of Hamilton and Cambridge. In the Cambridge area, the unit is about 1 m thick (Brunton, 2008). The Irondequoit Formation is approximately 3 m thick in the study area and consists of thickly to medium-bedded crinoidal dolomitic limestone (Brunton, 2008). The Rockway Formation is a thin unit (its thickness varies from 1 m to 2 m) fine crystalline argillaceous dolostone (to limestone) with shaley partings (Brunton, 2008). The Merritton Formation is a thin unit that consists of a bioturbated dolostone unit with a fine crystalline matrix and dark shaley partings. The Merritton Formation may have been previously mapped in the Region as the Fossil Hill Formation.

The Cabot Head Formation is described as grey-green noncalcareous shale with thin interbeds of sandstone and limestone (Johnson *et al.*, 1992). The thickness of the Cabot Head Formation ranges from 10 m to 39 m.

3.3 Model Layers

A total of 15 model layers are defined to represent the overburden and bedrock hydrostratigraphic units. The model layers are summarized on **Table 2-3** and are illustrated in **Figure 3**.

A 0.1 m thick layer is included at the ground surface to incorporate shallow surface water features. This layer is underlain by six model layers (slices 1 to 8) that represent the overburden hydrostratigraphic units listed on Table 3-1 and described in Section 3.1.

Model layer 8 (slices 8 and 9) represents a Contact Aquifer, which is conceived as an interface zone that comprises any permeable materials at the bottom of the overburden and the weathered top of rock. The Contact Aquifer is assigned a uniform thickness of 2 m across the model; the elevations of the top and bottom of the Contact Aquifer are determined by adding and subtracting 1 m from the bedrock surface.

Seven model layers (slices 9 through 16) represent the bedrock units from the Guelph Formation to the top of the Cabot Head Formation. Layer 9 represents the Guelph Formation. Layer 10 represents the Reformatory Quarry Member of the Eramosa Formation (the Stone Road Member is not present in the Cambridge area). A separate layer 11 is included to represent the Vinemount Member of the Eramosa Formation, as where this unit is present it acts as a regionally significant barrier to vertical flow. Layer 11 represents the two members of the Goat Island Formation. The Gasport Formation extends over three model layers. Layer 13 represents the upper portion of the Gasport Formation. Layer 14 represents the middle portion of the Gasport Formation. Where this unit is transmissive it acts as the dominant aquifer in the deep bedrock system.

The Rochester, Irondequoit, Rockway and Merritton Formations are thin (1 m to 3 m thick) in the study area and not significant with respect to groundwater flow. To simplify the analysis, these units are lumped with the lower portion of the Gasport Formation in Layer 15.

The Cabot Head Formation is interpreted to be a regional aquitard, and its surface is represented as an impermeable boundary that forms the base of the groundwater model. The Cabot Head Formation is described as grey-green noncalcareous shale with thin interbeds of sandstone and limestone (Johnson *et al.*, 1992). Support for the specification of the top of the Cabot Head Formation as a limit to active groundwater flow is presented in **Appendix A** (SSP&A, 2012).

The relations between the model layers and hydrostratigraphic units are summarized on **Table 3-3**.

Table 3-3: Model layers and corresponding hydrostratigraphic units

Model Layer	Hydrostratigraphic Unit	Hydrostratigraphic Unit Description
1	-	A thin (0.1 m) layer used to incorporate shallow surface water features
2	1 2	Puslinch Lake Sediments, Whittlesey Sand and Whittlesey Clay Wentworth Till (ATA2; *Poor Aquitard*)
3	3	Grand River Outwash (AFA2; *Aquifer*)
4	4	Port Stanley Till (ATB1; *Aquitard*)
5	5	Waterloo Moraine Stratified Sediment Equivalents (AFB1, AFB2; *Aquifer*)
6	6	Maryhill and Catfish Creek Tills (ATC1, ATC2; *Aquitard*)
7	7	Pre-Catfish and Pre-Canning Outwash (AFD1, AFF1; *Aquifer*)
8	8	Contact Aquifer at the interface between the overburden and bedrock
9	9	Guelph Formation *(Aquifer)*
10	10	Eramosa Formation - Reformatory Quarry Member (*Aquifer/Aquitard*)
11	11	Eramosa Formation - Vinemount Member (*Aquitard*)
12	12	Goat Island Formation (*Aquifer*)
13	13	Upper Gasport Formation (*Aquifer*)
14	14	Middle Gasport Formation (*High Permeability Aquifer*)
15	15	Lower Gasport/ Rochester/ Irondequoit/ Rockway/ Merritton Formations (*Aquifer/Poor Aquifer/Poor Aquitard*)
-	16	Cabot Head Formation (*Aquitard*)

The Cambridge East model includes 16 slices that define 15 model layers. Each slice contains approximately 145,000 nodes, delineating 289,000 triangular prismatic elements in each model layer. There are a total of 2.32 million nodes in the three-dimensional finite element mesh, delineating 4.33 million prismatic elements.

The layer structure of the groundwater model is illustrated on two cross-sections that run through the Puslinch Lake area. The locations of the sections are shown in Figure 1 and the sections are presented in **Figure 4** and **Figure 5**.

4 MODEL PROPERTIES, BOUNDARY CONDITIONS AND SOURCES AND SINKS

The key model properties are the layer thicknesses, hydraulic conductivities and storage parameters. The layer thicknesses are specified based on the surfaces defined in **Appendix A**. Hydraulic conductivity values quantify the relative ease with which water can move through pore spaces or fractures. Hydraulic conductivity values control the model-calculated hydraulic head distribution. The storage parameters control the timing of responses to transient stresses. The specific storage controls the timing of the responses of hydrostratigraphic units below the water table, for example, the changes in water levels caused during a pumping test in a deep bedrock unit. The timing of the decline of the water table to transient stresses is controlled by the specific yield.

4.1 Hydraulic Conductivities

In situ estimates of hydraulic conductivities are generally sparse and representative of conditions in the immediate vicinity of the test locations. Therefore, adjustments of hydraulic conductivities are required to achieve a match to the observations with the simulation results. The systematic adjustment of material properties to match the observations is referred to as model calibration. Estimates of hydraulic conductivity that are specified as the starting point for model calibration are based on the conceptual understanding of the geologic/hydrostratigraphic units and their hydrogeologic properties. Where data exist, pumping tests or slug tests constrain the hydraulic conductivity estimates within particular geologic formations. Where such data are not readily available, hydraulic conductivity values are estimated from literature values for materials with similar lithological descriptions, or from previous studies conducted in the area. In this study, both site-specific data and literature values have been used to assign initial estimates of hydraulic conductivity.

Golder Associates collected available hydraulic conductivity and transmissivity estimates as part of the Environmental Assessment; these values have been used wherever available to constrain the properties inferred during the analyses. Interpretations of pumping tests generally yield estimates of the transmissivity, which corresponds to the product of the hydraulic conductivity and thickness of a hydrostratigraphic unit. Hydraulic conductivity and transmissivity estimates for the overburden and bedrock units have been extrapolated from field-based local-scale studies. Average field measured values were initially assigned in the model and the values were altered within the range of field measured conductivities through the model calibration process. **Appendix B** contains details regarding the calibrated hydraulic conductivity values in each model layer within the model domain.

Where bedrock or overburden layers pinch out, the model layer is assigned a minimum layer thickness of 0.1 m and the hydraulic conductivity of the underlying layer. The thicknesses of the bedrock and overburden model layers were not adjusted during model calibration.

4.2 Storage Parameters

Groundwater storage is defined as the quantity of water released from an aquifer system due to a unit change in hydraulic head. Storage parameters are not used in steady-state flow solutions, but are required in simulations of transient responses to short-term changes in the groundwater system. The magnitude of the FEFLOW storage coefficient is dependent on whether the aquifer is unconfined or confined. The storage coefficient for confined conditions is referred to as specific storage (S_s). In a confined aquifer, when there is a reduction of hydraulic head, water is derived through the expansion of water and the compression of the rock or sediment. A typical value of the specific storage is 10^{-6} /m (Boonstra, 1989).

In an unconfined aquifer, the primary source of water from storage is from gravity drainage from the pores. The storage coefficient for unconfined conditions is designated the specific yield (S_y). For unconsolidated sediments, specific yield values representative of complete drainage are similar in magnitude to the porosity; values of the specific yield for granular materials typically range from 0.1 to 0.3 (Stephens *et al.*, 1998).

To simplify the model calibration, only hydraulic conductivities have been adjusted to match the observed hydrographs and the final drawdowns. Fixed values are specified for the specific storage and specific yield for two reasons:

- It is not possible to determine the storage parameters independently. Even controlled *in situ* hydraulic tests do not yield definitive estimates of specific storage values, with estimates ranging over orders of magnitude; and
- The simulation results are relatively insensitive to changes in the storage parameters. Confined storage coefficients may vary over an order of magnitude without causing significant differences in the results.

Sensitivity analyses with respect to the storage parameter have been conducted to investigate potential impacts to the simulation results due to the uncertainties in the model storage parameters. The results are presented in **Appendix F**.

4.3 Model Boundary Conditions, Sources and Sinks

Boundary conditions represent the interaction between the simulated groundwater system and the surrounding area, expressed in terms of conditions around the perimeter of the model. Two types of boundary conditions are included in the groundwater model:

- *Specified–Flux boundary conditions* are assigned at locations along the perimeter surface of the model to represent known fluxes into or out of the modeled groundwater system; and
- *Specified–Head boundary conditions* are assigned at locations along the perimeter surface of the model where the hydraulic head values are constrained.

Sources and sinks refer to features that add or withdraw water from the groundwater system. These features include infiltration to the water table (recharge), pumping wells, lakes and streams. Sources and sinks are represented with some of the same techniques as the boundary conditions, for example, recharge is represented as a specified flux across the water table. For the Cambridge East analysis, streams are represented with specified-head conditions. The Region of Waterloo municipal production wells are simulated with a specialized specified-flux condition, referred to in FEFLOW terminology as a "well" boundary condition.

4.4 Boundary Conditions

The model boundary conditions are shown in **Figure 6a**. To assign appropriate boundary conditions for the perimeter of the model, shallow and deep groundwater levels have been reviewed across the Region of Waterloo, using potentiometric surfaces developed in the Region of Waterloo Tier Three Assessment (AquaResource, 2010a). Where water level trends suggest natural divides across which there is no groundwater flow, no-flow boundary conditions are specified. In areas where water level trends indicate cross-boundary flow, water levels equivalent to the interpreted levels in those layers are specified.

To limit the extent of the model, groundwater levels are specified along the northern, eastern and south-eastern portions on the model boundary, where natural hydrologic boundaries are distant. These boundaries are sufficiently far from the municipal well fields in the Cambridge area that they have negligible influence on the model results.

Along the northern and eastern model boundaries, the model overlaps with the groundwater model that has been developed for the Guelph Tier Three Source Water Protection study. The steady-state simulation results of the Guelph model are used to assign the groundwater levels around the northern edge of the Cambridge model. For the south-eastern model boundary, the assigned heads have been interpolated from an interpretation of regional groundwater flow based primarily on water levels taken from the domestic water well records. The water well record data were first examined to remove any anomalies; the wells were then grouped according to whether they were open in the overburden or the bedrock. The water levels from the water well records were interpolated separately for the overburden and bedrock. The interpolated water levels along the boundary were examined to ensure that they did not exceed the ground surface and were consistent at the intersections with the other boundaries.

4.5 Surface Water Features

The interaction between groundwater and surface water is simulated in the model using specified-head conditions. Groundwater may discharge into the surface water body, or water may leak from the surface water feature, depending on the difference between the water levels specified for the surface water features and the model-calculated levels in the underlying groundwater system.

The rivers, streams and creeks that are simulated explicitly in the model are shown in **Figure 6b.** In general, creeks with a Strahler Order equal to 3 or greater are simulated in the model, as well as all rivers or streams that lie within 1 km of a Region of Waterloo municipal water supply well, all rivers with existing or historic streamflow observations (*i.e.*, Water Survey of Canada gauge stations), and rivers that are reported by the GRCA or MNR to host coldwater fish communities.

The surface water features are represented in the model by assigning specified-head conditions to nodes of the finite element mesh along a river or stream reach. The specified head for each node is set as the estimated surface water elevation at that location, using data from the 10 m Digital Elevation Model (DEM) for the study area. In some cases, the elevation from the DEM is not consistent with the elevation of the centerline of the stream channel. In these cases, the stream network has been inspected and the river stage specified in the model adjusted to ensure that the river stage decreases monotonically in the downstream direction. When developing the finite element mesh, the selected streams have been 'simplified' in a Geographic Information System (GIS) so the streams have segments that are a minimum length of 50 m. Finite element nodes are then assigned at the end of each segment, with a buffer row of elements assigned around each segment.

The Shade's Mills Reservoir is included in the set of surface water bodies modelled using specified-head boundary conditions. The water level for the Shade's Mills Reservoir is assigned based on a review of the GRCA reservoir elevation data (2007) and the 10 m DEM.

The approach for representing Puslinch Lake evolved during the analyses. At the start of the analyses, Puslinch Lake was also included among the surface water bodies modelled using specified-head conditions. The final approach adopted to simulate Puslinch Lake has involved representing the lake as a zone of high conductivity, over which an elevated rate of recharge corresponding to the difference between precipitation and evaporation over the lake itself is applied. This is a well-established approach for representing lakes (Anderson and others, 2002). The change in approach prevents Puslinch Lake from erroneously supplying unlimited amounts of water to the underlying aquifer when pumping is increased in the predictive scenario simulations. With this revised approach, the long-term average water level for Puslinch Lake is treated as a target to be met during model calibration. The target water level for Puslinch Lake is set as the average water level at the reported in the Region of Waterloo WRAS+ database (gauge LWL-06; object number 9202660). The approach for representing the Shade's Mills Reservoir did not require a similar revision of approach as no significant increases in pumping are considered in the vicinity of the reservoir. The specified water level for the Shade's Mills Reservoir and the target elevation for Puslinch Lake are listed on **Table 4-1**.

Table 4-1: Specified Lake Levels

Lake	Specified Lake Level (m ASL)
Puslinch Lake	304.6
Shade's Mills Reservoir	287.2

4.6 Recharge

Groundwater recharge refers to the annual or seasonal rate at which precipitation infiltrates through the unsaturated zone and ultimately reaches the water table. The rate of groundwater recharge depends on several factors, including land use and vegetation, surficial soil type, surficial geology, physiography, and ground surface topography. Recharge is enhanced in areas where the ground surface is hummocky and there are local-scale closed basins.

Recharge rates are commonly derived using surface water modelling techniques that are often undertaken on a watershed or subwatershed basis. Recharge is specified in the Cambridge groundwater model independently of model calibration. The results of the Grand River Watershed continuous streamflow-generation model (GAWSER; Guelph All-Weather Sequential-Events Runoff) are the primary basis for the assignment of recharge. The Grand River Watershed GAWSER model has been revised as part of the Region of Waterloo Tier Three Assessment and incorporates approximately 15 years of continuous improvement and advancement (AquaResource, 2009b).

The GAWSER model utilizes Quaternary geology, land cover, and topography to subdivide the model area into hydrologic response units to predict the response to a precipitation event. Precipitation, estimated using input from historic and current climate records, is partitioned into three major hydrologic components: runoff, evapotranspiration, and recharge. The model is calibrated by comparing the GAWSER-simulated hydrographs to observed streamflow at various gauge locations within the study area. The final distribution of recharge inferred through calibration of the GAWSER model is shown in **Figure 7**. The figure is reproduced from AquaResource (2009b).

The highest recharge rates within the Grand River Watershed are simulated in the hummocky outwash areas that exist along the Paris and Galt Moraines. High recharge rates are also simulated along the Grand and Speed Rivers where coarse-grained outwash sediments are mapped at surface. Relatively low recharge rates are simulated on the fine-grained till plains located north and northwest of the Cambridge. Additional details regarding the GAWSER model and how it has been calibrated and refined as part of the Region of Waterloo Tier Three Assessment are presented in AquaResource (2009b).

4.7 Pumping Wells

Two sets of water takings from pumping wells are specified in the model: municipal water takings from Region of Waterloo production wells and non-municipal pumping. The municipal and non-municipal water takings specified as pumping wells are shown in **Figure 8**.

Municipal water takings

The data for the municipal supply wells are listed on **Table 4-2**. The pumping rates listed on **Table 4-2** are the average pumping rates reported for 2003. Pumping rates from 2003 have been adopted for calibration of Region-wide models because conditions during that year provide a representative impression of average conditions over the 10-year period preceding the shutdown of the Greenbrook Well Field. The municipal wells are simulated as specified-flux conditions within the groundwater flow model using one-dimensional vertical line elements superimposed on the three-dimensional finite element mesh. For a well that is open across only one model layer, the pumping rate is assigned to that layer. For wells that are open across multiple model layers, the pumping rate is assigned to the lowermost slice intersected by the well. For these wells the specified pumping rate is partitioned automatically between the layers across which the well is open according to the relative transmissivity of the model layers. This approach ensures that a single water level is calculated for each well.

Table 4-2: Municipal Pumping Wells

Municipal Well	Easting (NAD83)	Northing (NAD83)	Formation Screened[1]	Rate assigned for the model calibration analyses (m^3/day)
G16	558336	4804721	Guelph Formation to Upper Gasport Formation	1,636
G17	556271	4804365	Reformatory Quarry Member to Upper Gasport Formation	1,364
G38	557302	4802637	Overburden	0
G39	557324	4802665	Contact Aquifer to Guelph	2,814
P16	550338	4807753	Overburden to Contact Aquifer	217
P9	555792	4806582	Guelph Formation to Goat Island Formation	1,157
H3	555314	4808183	Goat Island Formation to Upper Gasport Formation	281
P10	556951	4806839	Overburden to Guelph Formation	2,085
P11	557140	4806113	Contact Aquifer to Upper Gasport Formation	1,106
P17	557128	4806110	Reformatory Quarry Member to Middle Gasport formation	832
G7	558050	4802493	Overburden to Guelph Formation	2,199
G8	558339	4802613	Overburden to Contact Aquifer	1,042
G6	556355	4805062	Contact Aquifer to Goat Island	1,237
G18	557327	4804287	Guelph Formation to Goat Island Formation	1,347
P15	555754	4806616	Guelph Formation to Goat Island Formation	468
G5	555084	4806561	Contact Aquifer to Guelph Formation	1,225
G9	557175	4800261	Contact Aquifer to Vinemount Member	2,475
P6	554159	4804014	Guelph Formation to Upper Gasport Formation	1,386
H4	556693	4808882	Contact Aquifer to Goat Island Formation	284
H5	555327	4810826	Reformatory Quarry Member to Upper Gasport Formation	1
G4	553871	4802268	Guelph Formation	455
G1	555241	4799693	Reformatory Quarry Member to Goat Island Formation	7936
G1A	555242	4799687	Reformatory Quarry Member to Goat Island Formation	26
G2	555213	4799683	Reformatory Quarry Member	7159
G3	555183	4799708	Guelph Formation to Reformatory Quarry Member	5683
G14	555293	4799716	Guelph Formation to Goat Island Formation	0.3
G15	555170	4798053	Guelph Formation to Goat Island Formation	1077
BM1	559618	4795353	Guelph Formation	34.4
BM2	559622	4795353	Guelph Formation to Reformatory Quarry Member	0.1

[1] [1] The screened formation entries are inferred from the hydrostratigraphic model using the available screen information of the pumping wells.

Non-municipal water takings

In addition to municipal water demands, several non-municipal permitted water takers rely on groundwater supplies within the study area. Three data sources have been reviewed to obtain the best estimates of the non-municipal permitted water takings:

- Permit To Take Water (PTTW) Database (2008) – Ministry of the Environment (provided by the GRCA);
- Water Taking Reporting System (WTRS) (2008) – Ministry of the Environment (provided by the GRCA (Extraction Date: January 19, 2010)); and
- Monthly reported water takings (2002-2006) – Grand River Conservation Authority Integrated Water Budget Report (AquaResource, 2009i).

More than one source of information on non-municipal permitted water takings has been consulted because no single data source has all the required information. For example, the MOE's PTTW database does not contain information on actual water usage; rather it only includes information on the permitted rates. In contrast, the WTRS database contains information on reported water usage for individual permits and sources; however, it does not contain the spatial location (easting and northing) for permits or water sources. Details regarding the non-municipal pumping data are summarized in a technical memorandum (AquaResource, 2010b). The data for the non-municipal wells are assembled in **Appendix C**. The assigned rates represent the monthly-averaged rates between 2002 and 2006 reported in AquaResource (2010b). The non-municipal wells are simulated using the same approach as the municipal wells.

The non-municipal takings do not include domestic wells. The takings from domestic wells are not documented; however, the cumulative takings are expected to be relatively small as the Cambridge East area is serviced.

5 CALIBRATION OF THE GROUNDWATER FLOW MODEL

5.1 Overview

Model calibration is conducted to establish that the groundwater flow model can reproduce observed groundwater levels and flows. Demonstration of the ability to match observations is essential to confirm that a model can be applied to support predictions of the potential effects of changes in groundwater takings.

The Cambridge groundwater model has been calibrated by systematically adjusting the model input parameters to achieve a match to calibration targets. The targets include multiple sets of observations, including water levels and interpreted stream baseflows. The model parameters have been adjusted within physically realistic bounds consistent with the conceptual model. The ability to represent observed conditions is assessed quantitatively and qualitatively. Quantitative evaluation involves the assessment of statistical measures of goodness-of-fit between observed and calculated quantities. Qualitative evaluation involves the assessment of trends in water levels and the distribution of groundwater discharge and judgement as to whether the results of the modelling are sensible and consistent with the overall conceptual model. Although qualitative measures necessarily involve judgement, they are not inferior to quantitative measures. Rather, they should be regarded as complementary (Anderson and Woessner, 1992; p. 236).

Calibration of the Cambridge model has included the application of manual and computer-assisted techniques to match steady-state and transient targets. To test the reliability of the calibrated groundwater model, the model is applied for the simulation of an alternative set of steady-state targets, and for the simulation of a pumping test conducted at a newly-installed test well. Details of the calibration approach are presented in the next section.

5.2 Calibration Approach

For the steady-state calibration, the model input parameters and boundary conditions are specified to simulate the long-term average groundwater flow conditions. The steady-state targets have been developed from observations representative of average conditions during 2003. The calendar year 2003 is adopted by the Region of Waterloo as a consistent basis for the calibration of Region-wide models, as it corresponds to a period of relatively stable municipal pumping. Data from 2003 are considered for calibration because conditions during that year provide a representative impression of average conditions over the 10-year period preceding the shutdown of the Greenbrook Well Field. The average annual pumping rates during 2003 are specified for all municipal and non-municipal wells within the Region. During the calibration, the matches to both region-wide conditions and conditions at the individual well fields in the study area are assessed.

In conjunction with the calibration of steady-state conditions, the Cambridge model has been calibrated to match the transient observations collected during two high-quality pumping tests: a 28-day pumping test at multiple test wells in the Cambridge East area and a 6-day pumping test at Clyde Park. These are high-quality tests characterized by their relatively long durations, careful control of pumping rates and extensive three-dimensional monitoring at dedicated observation wells. The transient calibration has involved matching the observed changes in water levels at pumping wells and monitoring wells, particularly those in the production aquifers. The focus for these transient calibrations has been directed towards matching the changes in water levels in wells with clear responses to pumping or shutdown. However, the data from these tests have also been used to refine hydraulic conductivity values in areas where clear responses were not detected.

Two additional simulations have been developed to test the ability of the calibrated model to match observations that have not been included in the sets of calibration targets. These tests comprise the "validation" effort conducted to demonstrate the predictive capabilities of the model. The first validation scenario considers the long-term effects of the increase in pumping from the Clemens Mill Well Field during the mid-1990s. The second validation scenario considers a pumping test conducted at the recently installed test well G5A. To provide a reliable assessment of the predictive capabilities of the calibrated model the validation scenarios are simulated with no adjustments of the model parameters.

5.3 Calibration Targets

During calibration, the simulation results are compared with targets derived from measurements of hydraulic heads and interpreted stream baseflows. The targets for the steady-state calibration include values of hydraulic head obtained from Ontario Ministry of Environment (MOE) records of private (domestic) water wells and from high-quality monitoring wells that are currently monitored, or were monitored in the past, by the Region of Waterloo. The model has also been calibrated to a range of spot flow measurements collected by the GRCA and others at stream reaches within the study area, and to interpretations of baseflow derived from measurements at continuous stream gauges across the study area.

5.3.1 Groundwater Level Targets Derived from MOE Water Well Records

Water well records are submitted to the MOE for all wells completed in the Province of Ontario. The MOE maintains a database of information contained in each of the records, including a record of the lithology observed when drilling and the static water level elevation observed in the well after its completion. The information is contained in the MOE Water Well Information System (WWIS) database. Waterloo Hydrogeologic (now Schlumberger Water Services) compiled a database of water levels developed from the MOE water well records. The database was developed specifically to provide water level targets for the calibration of a regional-scale groundwater flow model of the Region of Waterloo (WHI, 2008). The WHI database includes information on the reliability of the geographic location and the reference elevation of the well.

Hydraulic head measurements from 1395 records in the WHI database have been included in the set of targets for the steady-state calibration. Of these 1395 wells, 632 are completed in the overburden and 763 are completed in the bedrock. The locations of the water level records used as targets are shown in **Figure 9a**. Details on the targets are provided in **Appendix D**.

The calibration targets developed from the MOE water well records are numerous and distributed regionally. Therefore, they offer the benefit of extensive spatial coverage; however, there can be uncertainty associated with individual targets. Uncertainties arise due to errors in the reported location of the wells and measurement techniques that were not designed to provide reliable scientific information. Furthermore, the water levels reported in the MOE water well records are also only representative of the time at which the wells were completed. The targets from the MOE records are not considered to be accurate indicators of an exact water level at a specific location and are designated “low-quality” targets. The targets from the MOE water well records are used here primarily to assist in the assessment of regional trends. The targets from the MOE water well records are assigned relatively small weights in the computer-assisted calibration analyses, so that undue emphasis is not placed on achieving close matches to individual targets.

5.3.2 Groundwater Level Targets Derived from Region of Waterloo Monitoring Wells

The Region of Waterloo monitors groundwater levels on a monthly basis at all of the well fields in the Cambridge area. Continuous monitoring is conducted at selected monitoring wells. The coordinates and vertical intervals of the open intervals of the Region of Waterloo monitoring wells have been established through surveying. Time series of water levels (hydrographs) are available for the Region of Waterloo wells. Therefore, the water level targets derived from the Region of Waterloo monitoring wells are designated "high-quality". The hydrographs for the monitoring wells have been reviewed, and representative average water levels for 2003 estimated for the steady-state calibration targets.

New monitoring wells were drilled and their water levels were measured as part of the IUS study in the Cambridge area (Golder Associates, 2007). Most of the new monitoring locations are instrumented for water level monitoring at multiple depths. The data from the multi-level installation reveal some important features of the groundwater flow system, in particular the water level differences across different hydrostratigraphic units. Although these wells were installed after 2003, in light of the important insights they provide regarding vertical hydraulic gradients the water levels derived from the records for these wells are included as high-quality calibration targets. In total, 205 targets for the steady-state calibration have been assembled from the Region of Waterloo monitoring wells and the IUS monitoring wells.

The Region of Waterloo also records groundwater levels in its municipal supply wells. The average water levels for the production wells have been included as calibration targets. However, the levels in the production wells are affected by processes in the wells themselves and vary significantly depending on the pumping rate at a specific instant and the rehabilitation history of the well. Therefore, the water levels at the municipal pumping wells have been included in the set of calibration targets, but are designated as low-quality targets. A total of 25 targets are derived from the production well records.

The 205 high-quality targets include data from 35 of the Region of Waterloo monitoring wells that have monitoring intervals longer than 10 m and which extend across multiple hydrostratigraphic units. These targets are assigned relatively lower weighting, to reflect the uncertainties introduced by their long lengths.

The locations of the Region of Waterloo monitoring wells, the IUS monitoring wells and the municipal supply wells are shown in **Figure 9b**. Details on the targets are included in **Appendix D**.

5.3.3 Groundwater Discharge Calibration Targets

The steady-state model calibration targets also include estimates of groundwater discharge derived from stream baseflows. The estimates of baseflows are developed from flow measurements at continuous stream gauges and from spot streamflow measurements collected by the GRCA and for the Tier Three Assessment field program. The locations of the spot measurements and the stream gauges are shown in **Figure 9c**. An estimate of groundwater discharge is developed for the two continuous stream gauges located along Aberfoyle Creek and Mill Creek, 3AQ131 and 2GAC19. Three estimates of groundwater discharge are developed from the measurements at the five spot flow locations:

- Irish Creek: from the data at IC_20;
- Ellis Creek: from the difference in spot flow measurements at CGC_10 and EC_20; and
- Blair Creek: from the difference in spot flow measurements at BBB3 and BBB4.

5.3.4 Transient Calibration Targets

The observation data from two high-quality long-term pumping tests in the Cambridge East area are used to develop targets for transient model calibration:

- Cambridge East 28-day pumping test (pumping from wells CMPW1-06, CMPW2-06 and PBPW1-06); and
- Clyde Park 6-day pumping test (pumping from well NDPW1-08).

For the Cambridge East 28-day test, the model has been calibrated to match the declines in water levels observed at the end of pumping at 77 observation wells. Attempts to match the complete records of water level changes have also been made for 49 of the observation wells that showed the clearest response to pumping (hydrographs for a subset of these wells are presented in the figures that accompany Section 5.5). For the Clyde Park 6-day test, the model has been calibrated to match the final observed drawdowns at 53 observation wells. During calibration to match the observations during the pumping tests all adjustments made to model parameters are also incorporated in the steady-state calibration.

5.4 Results of the Steady-State Calibration

5.4.1 Matches to Hydraulic Heads

The match between the model and the steady-state calibration targets is expressed in terms of the *residual* at each target location. The residual is defined as the difference between the calculated and observed water levels:

$$R_i = cal_i - obs_i$$

Here cal_i is the calculated value and obs_i is the observed (target) value at location *i*. A positive residual indicates that the model calculates a hydraulic head that exceeds the target level.

Statistical measures calculated with the residuals provide a means for determining whether adjustments of model parameters have produced a demonstrably better match to the observations. The overall match to the observations is quantified with four statistics, as no single statistic characterizes adequately the match between observations and model calculations.

(1) The *Mean Residual* (*ME*) is defined as the arithmetic average of the residuals:

$$ME = \frac{1}{N}\sum_{i=1}^{N}\left(cal_i - obs_i\right)$$

Here *N* is the number of observations.

(2) The *Mean Absolute Residual* (*MAE*) is defined as the arithmetic average of the absolute value of the residuals:

$$MAE = \frac{1}{N}\sum_{i=1}^{N}\left|cal_i - obs_i\right|$$

(3) The Root-Mean-Squared Residual (*RMSE*) is defined as:

$$RMSE = \left[\frac{1}{N}\sum_{i=1}^{N}\left(cal_i - obs_i\right)^2\right]^{1/2}$$

(4) The normalized Root Mean Squared Residual (*NRMSE*) is calculated here by dividing the Root-Mean-Squared Error (RMSE) by the range in the observed hydraulic heads:

$$NRMSE = \frac{1}{(O_{\max} - O_{\min})}\left[\frac{1}{N}\sum_{i=1}^{N}(C_i - O_i)^2\right]^{1/2}$$

Here O_{min} and O_{max} denote the minimum and maximum observed hydraulic heads, respectively.

The statistics for the residuals for the calibration for 2003 average conditions are listed on **Table 5-1**. Separate statistics are reported for the "high-quality" calibration targets developed from the wells in the Region of Waterloo long-term monitoring network and the wells drilled since 2003 as part of ongoing IUS studies. The results for the individual calibration targets are included in **Appendix D**.

Table 5-1 Cambridge Model Calibration Summary Statistics

Calibration statistic	All Calibration Targets	High-Quality Targets
Number of targets	1625	205
Mean Residual, ME (m)	0.78	-1.08
Mean Absolute Residual, MAE (m)	4.56	3.50
Minimum Residual, m	-27.94	-11.54
Maximum Residual, m	26.49	13.51
Root –Mean Squared Residual, MSE, m	6.05	4.41
Range of observations, m	100.0	58.91
Normalized Root –Mean Squared Residual, RMSE	6.05%	7.48%

The Mean Residual for all of the calibration targets is 0.78 m. The mean error for the high-quality targets indicates that, on average, the calibrated model has a slight tendency to calculate hydraulic heads less than observed. A mean error of zero indicates that the calculated water levels are at the right average level. Although a mean error that is close to 0.0 is desirable, it is not a sufficient indicator of a good match, because large negative and positive errors may cancel each other.

For the steady-state calibration, the Mean Absolute Residuals are 4.56 m for the full set of targets and 3.50 m for the high-quality targets. The Mean Absolute Error, *MAE*, is a more useful measure of overall goodness-of-fit than the mean residual, as positive and negative residuals do not cancel. This is a measure of the likely magnitude of the difference between the model and the calibration target at any particular location in the model.

The Root-Mean-Square Residual is 6.05 m for the full set of calibration targets and 4.41 m for the 205 high-quality targets. The calibration statistic for the high-quality target suggests that on average, the simulated water levels at a specific location are within ±5 m of the interpreted long-term average value.

In theory, the Normalized Root-Mean-Square Residual (NRMSE) permits relative assessments of the goodness-of-fit for models of different extents and topographic settings. The *NRMSE* statistic for the full set of targets is 6% and the *NRMSE* statistic for the high-quality targets is 8%. Spitz and Moreno (1996; corrected 2003) have suggested that for a well-calibrated model the *NRMSE* should be less than 10%. The results for the steady-state calibration are consistent with this acceptance criterion.

A scatterplot for all calibration targets is shown in **Figure 10a**. The points on a scatterplot should plot randomly about the line of perfect agreement (the line of equality). A band of ±5 m is shown straddling the line of perfect agreement to suggest the typical uncertainty associated with the calibration targets. This band is provided simply as a suggestion to guide the eye in assessing the scatter with respect to the interpretation of regional-scale conditions. The experience of the project team members in reviewing water level targets derived from MOE water well records suggests that the uncertainty in an individual target is likely in excess of 5 m. A statistical analysis of the high-quality calibration targets has also been conducted. More than one water level measurement is available at the locations of 111 of the high-quality targets. The differences between the minimum and maximum observed water levels for these targets range from 0.08 m to 10.49 m, with an average difference of 2.2 m. This average difference represents a likely minimum value of the mismatch at any individual target in the regional-scale that can be reasonably expected from a steady-state analysis.

The scatterplot for all calibration targets within the Overburden and Contact Aquifer is shown in **Figure 10b.** The corresponding scatterplot for all calibration targets within the bedrock aquifers is shown in **Figure 10c**. As shown in **Figure 10a**, **Figure 10b** and **Figure 10c** the points in the scatterplots do not exhibit patterns that would suggest areas of the model where hydraulic heads are calculated systematically either too high or too low.

The cumulative probability distributions of the residuals for all targets, and for only the high-quality targets, are shown in **Figure 10d**. Spitz and Moreno (1996, p. 244-245) and Hill (1998) suggest that the residuals from a calibration should be normally distributed, with a mean of zero. This is interpreted to mean that the largest portion of the residuals plotted on a normal probability plot should approximate a straight line, with the residual corresponding to a 50% probability that is relatively close to zero. The cumulative probability distributions for both sets of targets are consistent with this general guidance for model acceptance.

The spatial distribution of the residuals for calibration targets within the overburden and the bedrock aquifers are presented in **Figure 11a** and **Figure 11b**, respectively. The shallow groundwater system is represented by conditions in the Contact Aquifer. At the regional scale, the groundwater levels in the Contact Aquifer are representative of water table elevations. Above the Contact Aquifer, permeable materials are thin and discontinuous in the Cambridge area. The hydraulic heads in the Middle Gasport Formation are representative of conditions in the deeper bedrock groundwater system. The simulated groundwater levels in the Contact Aquifer and the Middle Gasport Formation are presented in **Figure 12a** and **Figure 12b**. The general trend of simulated groundwater flow in the Contact Aquifer is towards the southwest. The general trend is consistent with the interpreted groundwater flow direction for the area developed for the Guelph Formation (Golder, 2011b). The residuals at the locations of the high-quality calibration targets are also shown in **Figure 12a** and **Figure 12b**. The distributions of the residuals suggest that there are no areas where the groundwater model calculates groundwater levels that are systematically greater than or less than the targets.

Where present, the Vinemount Member of the Eramosa Formation is a major regional aquitard that separates the bedrock units into what are referred to here as the Upper Bedrock and Lower Bedrock groundwater flow systems. The Upper Bedrock system comprises the Contact Aquifer, the Guelph Formation and the Reformatory Quarry Member of the Eramosa Formation. The Lower Bedrock system comprises the Goat Island Formation and the Gasport Formation. A crucial aspect of the conceptual model for regional groundwater flow is the distribution of the differences in groundwater levels between the Upper and Lower Bedrock systems. As part of the Region of Waterloo Tier Three studies, Golder Associates inferred the spatial distribution of these differences (Golder, 2011b). As a qualitative check on the model, the general consistency between the model results and the Golder Associates` inferences regarding areas of upwards and downwards flow is assessed.

The analysis is conducted at the scale of the individual hydrostratigraphic units; however, as a simplification here the general tendencies for upward or downward flow are quantified in terms of the water level differences between the Upper Bedrock system (here represented by the Contact Aquifer) and the Lower Bedrock system (here represented by the Middle Gasport Formation). The simulated water level differences between the Upper and Lower Bedrock systems are presented in **Figure 12c**. Negative values, indicated by the blue contours, denote areas where the water levels in the Lower Bedrock exceed the water levels in the Upper Bedrock, so that groundwater flow is upwards. The positive differences, indicated by the orange contours, denote areas where the water levels in the Lower Bedrock are less than the water levels in the Upper Bedrock, so that groundwater flow is downwards. The results are consistent with the interpreted vertical hydraulic gradients in the bedrock aquifers reported in Golder (2011b). In particular, the simulated results are consistent with the inference of upwards flow southeast of the Shade's Mills Well Field.

5.4.2 Inferred Groundwater-Surface Water Interactions and Matches to Baseflows Estimates

The locations and magnitudes of the model-simulated groundwater discharge zones along rivers and creeks are shown in **Figure 13**. In this figure, the blue circles denote reaches along which there are groundwater discharges to the rivers and streams (*i.e.*, gaining streams). The pink circles represent the reaches along which the calibrated groundwater model predicts leakage from the rivers and streams to the groundwater system (*i.e.*, losing streams). The sizes of the circles are scaled with respect to the magnitudes of the discharges. A comparison of the results presented in Figure 13 with maps produced by the Fisheries Habitat Plan (NVCA and Fisheries and Oceans Canada, 2009) and the GRCA (2010) suggests that the locations of most cold and cool water fishery stream reaches, both of which are known to be groundwater discharge areas, are identified correctly with the calibrated model. Some of the extreme upper reaches of the streams, particularly those close to, or above, the simulated water table, are not as well represented, especially in areas where a stream is thought to be fed by perched aquifer conditions.

Simulated groundwater discharges to streams are compared with the baseflows estimated for the two continuous stream gauges, 3AQ131 and 2GAC19. The groundwater contributions to the total flow at these gauges are estimated from the streamflow records with baseflow separation techniques. Simulated groundwater discharges are also compared with changes between locations of spot flow measurements collected during low-flow conditions. The difference between the spot flow measurements at two locations along a stream provides a rough indication of the gains or losses from the groundwater system. Spot flow measurements represent the measured flow at a location at one point in time. Although care is generally taken to ensure that the spot flows are not affected by recent rainfall events, baseflow varies seasonally and there is no assurance that the single measurement is representative of long-term average conditions. Baseflow estimates derived from gauges that are monitored continuously are more likely to be representative of long-term average flows.

The simulated baseflow and baseflow estimates derived from streamflow measurements are presented on **Table 5-2**. In general, these results suggest that the model results are consistent with the baseflow targets. The results are illustrated in **Figure 14**. The blue bands indicate the ranges of the baseflow estimates inferred from the observations and the crosses denote the cumulative groundwater discharges calculated with the calibrated steady-state groundwater model. Discussions of the results for each calibration target are provided below.

Table 5-2 Simulated versus Observed Groundwater Discharge to Streams

Stations	Station Description	Interpreted Baseflow (m^3/d) or Difference in Baseflow (m^3/d)			Simulated Net Groundwater Discharge to Stream (m^3/d)
		Minimum	Maximum	Middle	
GRCA gauges					
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd 7	17107	22032	18922	19908
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	50858	63333	55447	45845
Spotflow measurements					
Irish Creek (IC_20)	Irish Creek at Townline Rd.	5357	9245	9158	6781
Ellis Creek (Difference between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	2333	8726	3370	7183
Blair Creek (Difference between BBB3 and BBB4)	Blair Creek between Dickie Settlement Road and Blair Rd	-	-	1469	1676

5.4.2.1 Aberfoyle Creek

Streamflow in Aberfoyle Creek is measured continuously at gauge 3AQ131, located north of the intersection of Hwy 401 and Hwy 6. The gauge records the accumulated flow in Aberfoyle Creek from its headwaters. The reach of Aberfoyle Creek upstream of the gauge is simulated as net-gaining, with some segments of the creek losing water. The simulated net groundwater flow to Aberfoyle Creek is consistent with the range of baseflow estimates derived from the measurements.

5.4.2.2 Mill Creek

Mill Creek starts south of Hwy 401 near Aberfoyle and flows southwesterly through the Shade's Mills Reservoir before discharging to the Grand River. Streamflow upstream of the Shade's Mills Reservoir is measured continuously at gauge 2GAC19. Along most of its length, Mill Creek is simulated to be gaining. At the Shade's Mills Reservoir and along the section close to the Grand River, the results of the calibrated model suggest that Mill Creek leaks to the groundwater system. The calibrated model predicts that the induced infiltration from the Shade's Mills Reservoir corresponds to about 10% of the total pumping. There are no data that confirm whether there is significant pumping-induced infiltration from the Shade's Mills Reservoir. Lotowater (1993) provide arguments for and against this possibility. There is evidently a hydraulic connection between the aquifer and the reservoir, as water levels in the aquifer respond quickly to changes in reservoir levels (Lotowater, 1993). The simulated net groundwater discharge to Mill Creek is within the range of baseflow estimates developed from the baseflow separation analyses of the gauge data.

5.4.2.3 Irish Creek

Irish Creek begins north of Hwy 401 and runs westward towards the Hespeler Well Field before discharging to the Speed River. The spotflow measurements at IC_20 record the cumulative flow close to the outlet of Irish Creek at the Speed River. Most of the simulated reach of Irish Creek is predicted to be gaining water, with the exception of some localized areas. The simulated net groundwater discharge to Irish Creek is within the range of estimates developed from spot flow measurements.

5.4.2.4 Ellis Creek

Ellis Creek flows south towards Hespeler before discharging to the Speed River. Groundwater discharge is inferred from the differences between spotflows measured at locations CGC_10 and EC_20, along the reach of the creek between Maple Grove Road and Kossuth Road. The calibrated model predicts that this reach is gaining water along most of its length. The simulated net groundwater discharge baseflow for the reach between is within the range of the differences in baseflow estimates developed from the spotflow measurements.

5.4.2.5 Blair Creek

Blair Creek flows north before discharging to the Grand River near Fountain Street. The groundwater discharge to the creek is inferred from the difference in the spotflow measurements made at locations BBB3 and BBB4. This reach of Blair Creek is simulated to be gaining water along most of its length. A single value of baseflow is recorded for this reach. The simulated baseflow for this reach is close to the observed value.

5.4.3 Groundwater Interactions with Puslinch Lake

Puslinch Lake is one of most prominent surface water features within the Cambridge East area. In light of its proximity to pumping centres, it is important that it be confirmed that the calibrated model matches the data that available for the lake. Only one quantitative analysis of the lake water balance is available (Harden, 1999). Based on a rough analysis, Harden suggested the leakage from Puslinch Lake was on the order of 225,000 m^3/yr, equivalent to 616 m^3/d was reported (Harden, 1999; Table 3; reported as “Estimated Deep Aquifer Recharge”).

Puslinch Lake was simulated initially as a specified-head feature in the first model layer. In this area of the model, the thickness of the first layer is specified as the depth of the lake. The average observed water level of the lake, 304.3 mASL, is assigned as the specified head. After the preliminary calibration of the model, the calculated leakage from Puslinch Lake was about 700 m^3/day. No details regarding the methods used to estimate each component of the water balance are presented in Harden (1999) and no discussion of the reliability that should be assigned to the estimate of each component is provided. Although limited information on the Harden analysis was available, the agreement between the results of the Harden water balance calculations and the calibrated model was encouraging.

At the later stages of the modeling, it was found that the specified-head nodes representing Puslinch Lake were problematic when applied for predictive scenarios. In simulations with additional municipal pumping, the specified-head nodes erroneously supplied an unlimited amount of water to the surrounding groundwater system. To avoid this, the groundwater model was modified with Puslinch Lake simulated as a zone of elevated hydraulic conductivity and the net annual precipitation (precipitation minus lake evaporation) applied over the lake surface. The hydraulic conductivity of the zone representing Puslinch Lake is set sufficiently high such that negligible hydraulic gradients are calculated within the lake. In the revised calibration, the hydraulic conductivities of the materials around the lake are adjusted systematically to match the observed average lake level, 304.3 mASL.

A leakage rate from Puslinch Lake to the groundwater flow system of 715 m^3/d is calculated with the final calibrated model. This number is again relatively close to the Harden (1999) estimate of 616 m^3/d. The recharge assigned over Puslinch Lake in the calibrated model is 1,900 m^3/d. The area of Puslinch Lake is about 1,560,000 m^2 and as a first approximation the average annual precipitation and lake evaporation rates in this area of southern Ontario are about 1.0 m/yr and 0.6 m/yr, respectively. This corresponds to a volumetric rate of 1,700 m^3/d, which is of the same order of magnitude as the value inferred through calibration.

5.5 Transient Calibration: Cambridge East 28-Day Pumping Test

A pumping test was conducted in the Cambridge East area in September-October 2006. Wells CMPW1-06, CMPW2-06 and PBPW1-06 were pumped for 28 days. The three production wells were pumped in a stepped manner according to the schedule indicated on **Table 5-5**. During the test, the drawdowns were monitored at 77 wells, including the test pumping wells. The locations of the pumping wells and monitoring wells are shown in **Figure 15**.

Table 5-3 Cambridge 28-Day pumping test - pumping schedule

Well ID	OBJ_NUM	Pumping Start Date/Time	Pumping End Date/Time	Pumping Rate (L/s)
CMPW2-06	9202641	05-Sep-2006/12:00	02-Oct-2006/12:00	38
PBPW1-06	9202645	13-Sep-2006/13:30	02-Oct-2006/12:00	39
CMPW1-06	9202640	19-Sep-2006/12:45	02-Oct-2006/12:00	38

The test pumping wells were open across the middle unit of the Gasport Formation (model layer 14). Of the 74 observation wells monitored during the test, 38 wells were located within overburden materials (model layers 1 through 7), 3 wells were located in the Contact Aquifer (layer 8), and 33 wells were located in the bedrock (layers 9 through 15).

The adjusted final drawdowns in the pumping wells ranged from 7.5 m to 9.0 m. The drawdowns for the pumping wells have been adjusted to remove the nonlinear well losses, using the results from step tests (Golder Associates, 2011b). The final drawdowns observed in the overburden ranged from 0.2 m to 1.3 m, with 17 wells indicating no response to pumping after 28 days. The final drawdowns observed in the Contact Aquifer ranged from 0.5 m to 1.4 m. The final drawdowns in the bedrock ranged 0.2 m to 7.3 m, with negligible response observed in 7 wells.

The summary calibration statistics for the match between the observed and simulated drawdowns at the end of pumping are listed on **Table 5-4**.

Table 5-4 Summary of calibration statistics for the Cambridge East 28-day pumping test

Calibration Statistic	Value
Number of targets	77
Mean Residual, ME (m)	0.21
Mean Absolute Residual, MAE (m)	0.48
Minimum Residual, m	-2.64
Maximum Residual, m	2.57
Root-Mean Square Residual, RMSE (m)	0.76
Range of observations, m	9.05
Normalized Root Mean Square Residual, nRMSE (%)	8.40

A complete listing of simulated and measured drawdowns at the end of the 28-day pumping test is presented on **Table 5-5**. The drawdowns are calculated as the difference between the water levels at the beginning and end of pumping. The column "Well type" is used to distinguish between the three pumping wells and the 74 observation wells. The layer refers the model layer of the open interval of each well.

Table 5-5 Observed and Simulated Drawdowns for 28-Day Pumping Test

Well ID	Well type[2]	Easting	Northing	Layer	Drawdown at the end of the test	
					Observed (m)	Calculated (m)
OW57-95	M	558498	4805228	6	0.70	1.50
C2	M	559342	4806841	6	0.40	0.39
CM-OW5B-94	M	558346	4806731	14	2.23	2.90
CM-OW5D-94	M	558346	4806731	9	1.86	1.34
CM-OW5E-94	M	558346	4806731	6	1.31	0.83
CMTW1-05	M	558403	4805357	15	6.71	7.31
CMTW3A-05	M	557560	4805181	14	1.23	3.12
DW-REED	M	558578	4806477	8	1.41	1.17
CMOW3A-06	M	558916	4804867	13	4.26	5.05
CMOW3B-06	M	558916	4804867	9	1.34	1.62
CMOW3C-06	M	558916	4804867	9	0.65	0.77
HPOW1A-06	M	558786	4804395	5	0.23	0.22
PBOW1A-06	M	559286	4806058	14	3.22	3.40
PBOW1B-06	M	559286	4806058	9	0.74	1.26
PBTW2-06	M	558220	4806227	15	6.21	6.35
CMOW1D-06	M	558724	4805313	7	1.06	1.00
CMOW2E-06	M	559588	4804174	5	0.24	0.09
PBOW1D-06	M	559284	4806058	8	0.53	0.55
PBOW1F-06	M	559282	4806059	3	0.50	0.06
PBOW2A-06	M	558535	4806924	14	2.29	2.64
PBOW2B-06	M	558535	4806924	9	1.82	2.02
PBOW2C-06	M	558535	4806924	9	1.45	0.96
DW-FROSCH-JR	M	558943	4805820	8	0.73	0.67
CM-OW2BC-92	M	558283	4805068	6	0.23	0.61
OW4A-92	M	558908	4804870	9	0.66	0.83
OW4B-92	M	558911	4804870	6	0.59	0.66
OW4C-92	M	558911	4804870	4	0.55	0.07
SM3-93	M	557938	4803410	13	1.33	1.52
CMPW2-06	P	558967	4804451	14	7.57	8.81
PBPW1-06	P	558217	4806229	14	7.04	7.38
CMPW1-06	P	558400	4805353	14	9.05	8.59
SMTW1A-05	M	558520	4803673	14	3.72	3.68
CM-OW2A-92	M	558285	4805067	9	0.95	3.03
DW-OLIVER	M	559078	4804839	3	0.55	0.04
CMTW2-05	M	558964	4804452	13	7.33	7.92
CM-OW5F-94	M	558346	4806731	2	0.53	0.02
DW-SOCCER_WELL	M	558371	4805241	6	0.42	1.86
PBTW1-05	M	558317	4806857	15	1.94	2.73
CMOW1A-06	M	558726	4805314	15	4.44	4.86
CMOW1B-06	M	558726	4805314	9	1.41	2.00
CMOW1C-06	M	558726	4805314	9	0.90	1.20
CMOW2A-06	M	559589	4804174	14	4.01	4.36
CMOW2C-06	M	559589	4804174	9	0.17	0.32
PBOW1C-06	M	559286	4806058	9	0.55	0.71
PLOW1A-06	M	559403	4806775	3	0.36	0.03
PLOW2A-06	M	559066	4806672	3	0.45	0.04
CMOW2B-06	M	559589	4804174	9	0.37	0.62

[2] P denotes pumping well, and M denotes monitoring well.

Well ID	Well type[2]	Easting	Northing	Layer	Drawdown at the end of the test	
					Observed (m)	Calculated (m)
PBOW1E-06	M	559283	4806059	6	0.54	0.17
PBOW2F-06	M	558534	4806922	3	0.88	0.02
DW-SMALLBONE	M	559689	4804281	5	0.21	0.07
CMOW2D-06	M	559590	4804174	6	0.24	0.27
PBOW2D-06	M	558535	4806926	6	1.18	0.84
PBOW2E-06	M	558536	4806927	6	0.94	0.42
859_AVENUE_RD	M	558681	4803844	9	0.00	0.94
44_LAKE_RD	M	559620	4808220	6	0.00	0.10
SM3B-93	M	557938	4803410	6	0.00	0.08
TW9-78	M	560267	4804378	9	0.00	0.29
MW1-S	M	562785	4806823	4	0.00	0.08
MW1-D	M	562785	4806823	4	0.00	0.08
MW1-I	M	562785	4806823	4	0.00	0.08
SMTW1B-05	M	558520	4803673	10	0.00	1.26
SMTW1C-05	M	558520	4803673	9	0.00	0.30
CMTW3C-05	M	557560	4805181	9	0.00	0.71
PLOW1B-06	M	559403	4806775	2	0.00	0.03
CMOW1B-92	M	557015	4805952	6	0.00	0.21
CMOW1C-92	M	557015	4805952	4	0.00	0.05
CMOW1A-92	M	557019	4805950	13	0.00	1.69
CMOW2C-92	M	558283	4805068	5	0.00	0.12
CMOW2D-92	M	558280	4805065	3	0.00	0.07
TW10-78	M	562405	4806913	9	0.00	0.09
PLOW2B-06	M	559066	4806672	2	0.00	0.03
PLOW2C-06	M	559066	4806672	2	0.00	0.01
CMOW1F-06	M	558725	4805314	3	0.00	0.10
CMOW2F-06	M	559590	4804173	3	0.00	0.08
855_AVENUE_RD	M	558663	4803841	3	0.00	0.06
1493_TOWNLINE_RD	M	558603	4806220	3	0.00	0.03
PTOW1	M	560024	4808548	3	0.00	0.00

[1] P denotes pumping well, and M denotes monitoring well.

The observed and simulated drawdowns at the end of 28 days of pumping are plotted in **Figure 16a**. As shown in the figure, a good match is achieved to the observed drawdowns, with relatively little scatter. On average, the model predicts drawdowns that are only about 0.2 m higher than observed.

The cumulative probability distribution of the residuals at the end of the pumping is shown in **Figure 16b.** The residuals approximate a straight line when plotted against the cumulative normal probability, suggesting that the differences between the observed and simulated drawdowns are random and that the model does not have a systematic bias in its drawdown calculations.

The simulated declines in groundwater levels in the Contact Aquifer and the Middle Gasport Formation at the end of pumping are shown in **Figure 17a** and **Figure 17b**, respectively. The residuals are also indicated at the locations of the observation wells. A relatively large zone of influence is simulated in the deep bedrock aquifer.

Selected hydrographs showing the evolution of drawdown at the observation wells during the 28-day test are presented in **Figure 18a** through **Figure 18e**. The results suggest that the transient model effectively reproduces both the magnitudes of the drawdowns and the trends observed during this major pumping test.

During the calibration of the 28-day pumping test, it was found that to match the observed responses the three pumping wells had to be located within a zone of relatively high hydraulic conductivity. This zone is indicated as the area in Figure B-27, for which the hydraulic conductivity ranges from 5.5×10^{-5} m/s to 2.2×10^{-4} m/s. The value of the hydraulic conductivity in the vicinity of the test wells inferred through calibration is 2.2×10^{-4} m/s. The value inferred through calibration is consistent with the observation that the drawdown at pumping well CMPW2-06 is essentially identical to the drawdown at the observation well CMTW2-05, 2.8 m away. The analyses conducted during the matching of the observations of the 28-day pumping test have also suggested that the three pumping wells are located within the large-scale 'tongue-shaped' zone of high transmissivity in the Gasport Formation delineated previously in the southern portion of the City of Guelph from hydraulic testing and modelling (Golder Associates, 2011).

The attempt to match the observations during the 28-day pumping test have been particularly important for constraining the estimation of the hydraulic conductivity of the Vinemount Member of the Eramosa Formation, a regionally significant aquitard. Where the Vinemount Member is present, it is observed to act as a barrier to flow between the upper bedrock aquifers and the lower bedrock aquifers. To match the observations it has been found that the Vinemount Member must have a relatively low vertical hydraulic conductivity. A value of 10^{-9} m/s is estimated for the Vinemount Member where it is present. The Vinemount Member is thin or absent west of the Clemens Mill Well Field, and where the Vinemount Member is absent the hydraulic heads in the bedrock below the Contact Aquifer have been observed to be essentially uniform (see for example Burns, 2005; Figure 1-2).

5.6 Transient Calibration: Clyde Park 6-Day Pumping Test

A second pumping test was conducted in the Cambridge East area in September 2009. Well NDPW1-08 at Clyde Park was pumped between September 24 and 30 at an average rate of 50 L/s. The locations of pumping wells and observation wells are shown in **Figure 19.**

In the analysis of the data from any pumping test is necessary to isolate from the observed changes in water levels the declines that can be attributed solely to pumping. The inference of drawdowns during the Clyde Park pumping test is not straightforward. Well NDPW1-08 is located relatively close to the Clemens Mill Well Field and during the test the effects of pumping at the Clemens Mill Well Field are evident in the responses. The Clemens Mills wells were operated "on-demand" during the test, creating fluctuations in the water levels in observation wells in the area. At the start of the constant-rate portion of the test, the aquifer was still recovering to the step testing, further complicating the interpretation. Because background conditions changed during the test, it is not possible to identify precisely the drawdowns caused only by pumping NDPW1-08. For the purposes of calibration, a range of the observed drawdown at the end of pumping was developed for each well, for comparison with the single value obtained from the transient simulation after 6 days of pumping.

A complete listing of observed and simulated drawdowns at the end of the 6-day pumping test is presented on **Table 5-6**. The column "Well type" is used to distinguish between the pumping well and the 52 observation wells. The layer refers the model layer of the open interval of the well.

The test pumping well, NDPW1-08, was open across the middle portion of the Gasport Formation. Of the 52 observation wells monitored during the test, 23 wells were open in the overburden, one observation well was open across the Contact Aquifer and 28 wells are open across various intervals in the bedrock. Only one of the observation wells in the overburden exhibited a response to pumping after 6 days, with an interpreted drawdown of about 0.1 m. The final drawdowns observed in the Contact Aquifer well were interpreted to range between 0.30 m and 0.37 m. The final drawdowns in the bedrock ranged from about 0.2 m to 6.5 m, with negligible responses to pumping observed at 14 wells.

In **Figure 20** the simulated drawdowns are plotted against the ranges of the drawdowns interpreted from the observations. The transient simulation yields a single value of the drawdown at each monitoring location after 6 days of pumping. As indicated above, there is no corresponding definitive value for the observed drawdown. The comparison is presented in terms of ranges of observed drawdowns to account for the uncertainty in the inference of drawdowns from the observations caused by the fluctuations in pumping at the Clemens Mill Well Field. As shown in the figure, the simulation results are generally within the ranges of the observed drawdowns. The calibrated model reproduces the negligible responses observed in the overburden after 6 days of pumping. None of the simulated drawdowns at wells screened in the overburden units exceed 0.1 m.

The simulated water level declines in the Contact Aquifer and the Middle Gasport Formation at the end of the test are presented in **Figure 21a** and **Figure 21b**, respectively. The simulation results indicate that the Clyde Park pumping test induced negligible drawdown (<0.1m) in the Contact Aquifer. A relatively large zone of influence is simulated in the deep bedrock aquifer.

An independent analysis of the Clyde Park pumping test was developed to constrain the adjustment of material properties during the model calibration. The drawdowns observed in the pumping well during the step test and the responses observed during the constant-rate test suggest that the pumping well is surrounded by a zone that has an elevated transmissivity relative to the surrounding formation. The Barker and Herbert (1992) analytical model was used to explore the structure of the bedrock properties in the vicinity of the test pumping well. The Barker and Herbert model assumes the aquifer consists of a zone with one set of properties surrounding the pumping well, embedded within another zone of uniform properties corresponding to the bulk formation. The match to the drawdowns at the end of the test obtained with the Barker and Herbert model is shown in **Figure 22**. The results of the analysis with Barker-Herbert model suggested that the zone surrounding the pumping well had a transmissivity of 4,500 m^2/d, and extended about 1,600 m from NDPW1-08. In contrast, the bulk-average transmissivity of the aquifer beyond 1,600 m was estimated to be about 300 m^2/d. The results from the analysis with the Barker-Herbert model were applied to the numerical model, and resulted in a relatively good match to the observed drawdowns. This zone of high transmissivity is clearly visible in the distribution of hydraulic conductivity of the Middle Gasport Formation (Figure 27 in Appendix B).

Table 5-6 Observed and Simulated Drawdowns for Clyde Park 6-Day Pumping Test (Transient)

Well ID	Well Type[3]	Easting	Northing	Layer	Observed drawdown (m)		Calculated drawdown
					Maximum drawdown	Minimum drawdown	
1275-Village-Rd.	M	562554	4804324	3	0.00	0.00	0.00
1310-Village-Rd.	M	562166	4804197	9	0.17	0.09	0.00
2-Langford-Dr.	M	562124	4803692	3	0.12	-0.01	-0.10
CMOW1A-06	M	558726	4805314	15	2.78	1.33	-1.17
CMOW1F-06	M	558725	4805314	3	0.00	-0.07	-0.07
CMOW2A-06	M	559589	4804174	14	4.22	2.35	-1.86
CMOW2C-06	M	559589	4804174	9	0.00	0.04	0.04
CMOW2F-06	M	559590	4804173	3	0.00	-0.04	-0.04
CMTW3A-05	M	557560	4805181	15	0.00	0.61	0.61
CMTW3C-05	M	557560	4805181	9	0.00	0.03	0.03
MW1A-94	M	562785	4806823	4	0.00	0.07	0.07
MW1B-94	M	562785	4806823	4	0.00	0.01	0.01
MW1C-94	M	562785	4806823	3	0.00	0.00	0.00
NDOW1A-08	M	562254	4804736	15	5.02	4.08	-0.95
NDOW1B-08	M	562254	4804736	10	0.00	0.28	0.28
NDOW1C-08	M	562254	4804736	9	0.00	0.16	0.16
NDOW1D-08	M	562255	4804733	6	0.00	0.03	0.03
NDOW1E-08	M	562255	4804732	2	0.00	0.00	0.00
NDOW2A-08	M	561220	4803776	14	4.78	3.92	-0.87
NDOW2B-08	M	561220	4803776	10	1.59	2.25	0.66
NDOW2C-08	M	561220	4803776	9	0.95	1.05	0.11
NDOW2D-08	M	561223	4803779	8	0.37	1.07	0.70
NDOW2E-08	M	561221	4803779	2	0.00	0.00	0.00
NDOW3A-08	M	562046	4803929	3	0.00	-0.01	-0.01
NDOW3B-08	M	562045	4803930	2	0.00	-0.01	-0.01
NDOW4A-08	M	562601	4803606	3	0.00	-0.01	-0.01
NDOW4B-08	M	562601	4803608	2	0.00	-0.01	-0.01
NDOW5-08	M	563380	4805019	6	0.00	0.07	0.07
NDTW1A-08	M	562367	4804125	14	5.45	4.62	-0.82
NDTW1B-08	M	562367	4804125	10	0.25	0.55	0.36
NDTW1C-08	M	562367	4804125	9	0.18	0.08	0.01
NDTW1D-08	M	562368	4804123	6	0.00	0.07	0.07
NDTW1E-08	M	562369	4804124	3	0.00	-0.01	-0.01
NDTW2-08	M	562220	4804076	14	5.74	5.69	-0.11
PBOW1A-06	M	559286	4806058	14	2.72	1.39	-1.33
PBOW1C-06	M	559286	4806058	9	0.00	0.19	0.19
PBOW1F-06	M	559282	4806059	3	0.00	-0.08	-0.08
PBOW2A-06	M	558535	4806924	15	0.00	0.75	0.75
PBOW2C-06	M	558535	4806924	9	0.00	0.17	0.17
PBOW2F-06	M	558534	4806922	3	0.00	-0.16	-0.16
PLOW1A-06	M	559403	4806775	3	0.00	-0.07	-0.07
PLOW1B-06	M	559403	4806775	2	0.00	-0.07	-0.07
PLOW2A-06	M	559066	4806672	3	0.00	-0.10	-0.10
PLOW2B-06	M	559066	4806672	2	0.00	-0.09	-0.09
SMTW1A-05	M	558520	4803673	14	2.60	1.44	-1.63
SMTW1B-05	M	558520	4803673	10	0.00	0.31	0.31
SMTW1C-05	M	558520	4803673	9	0.00	0.03	0.03
TW10-78	M	562405	4806913	9	0.00	0.07	0.07
TW9-78	M	560267	4804378	9	0.00	0.08	0.08
1629-Clyde-Rd	M	562534	4803695	10	0.00	0.17	0.17
CMPW2-06	M	558967	4804451	14	3.51	1.82	-0.72
MW09-T3-01	M	560864	4810649	14	0.00	0.72	-0.28
NDPW1-08*	P	562219	4804072	14	6.47	6.20	-0.09

[3] P represents pumping well, and M represents the monitoring well.

* The observed drawdown of NDPW1-08 was adjusted to exclude wells losses of 2.727 m inferred from the step test conducted at this well.

5.7 Transient Calibration: Pinebush 15-Day Pumping Test

A 15-day pumping test was conducted at the Pinebush Well Field in October 2011 (Stantec, 2013). This test is particularly important with respect to the groundwater modelling of the Cambridge East area; the test was conducted for a relatively long duration with extensive monitoring in the central part of the study area, with pumping from both the shallow and deeper bedrock. The observations of the test are used to further calibrate the Cambridge East groundwater model.

The 15-day Pinebush test was conducted between 1:45 PM on October 4, 2011 to 10:00 AM on October 19, 2011. The pumping wells used for the test included TW1-10, P10A and P10B. Well P10B is a replacement for well P10, pumping from the Contact Aquifer. Both TW1-10 and P10A are deeper bedrock wells. TW1-10 pumps from the Reformatory Quarry Member, Goat Island Formation and the upper portion of the Gasport Formation; P10A pumps from the Gasport Formation. Changes in groundwater levels were monitored at about 50 monitoring locations. The locations of the pumping wells and monitoring wells are shown in **Figure 23**.

The pumping histories for the three test wells are summarized on **Table 5-7a**, and shown graphically in **Figure 24**. The test consisted of four stages. The test started with pumping from TW1-10 at a rate of 40 L/s. After about 3 days, pumping from P10B commenced at a rate of 40 L/s. After about 8 days, pumping from P10A began at a rate of 70 L/s. After about 13 days, TW1-10 stopped pumping and pumping from the other two wells ceased after 15 days.

The pumping rates of the other municipal production wells in the area are listed on **Table 5-7b**. The municipal production wells closest to the test wells are G16 and P17. During the test both wells were operated "on demand" to meet municipal requirements. The variations in the pumping rate for P17 are shown in Stantec (2013; Figure 11) and the major variations in the pumping rate are specified in the transient calibration. The rate of 18.0 L/s reported for P17 on Table 5-7b represents the average pumping rate during the testing period. Stantec (2013) do not provide the corresponding detailed history of pumping for G16 and a constant pumping rate of 23 L/s is specified for the duration of the simulation, which represents the average rate over the pumping period. Average pumping rates during 2003 are specified for the remaining municipal production wells. These are the same rates as are specified for the steady-state calibration. In effect, the drawdowns for the pumping test are interpreted as the changes in water levels caused only by pumping from the three test wells, TW1-10, P10A and P10B, and from changes in pumping from P17. The rates specified for the other municipal wells should not affect the results of the present analyses, as they are more distant and only long-term changes in average water levels should give rise to changes in water levels at the observation wells included in the transient calibration data set.

Table 5-7a Pumping rates of the test wells during the 15-Day Pinebush test

Time	Elapsed time (days)	Pumping rate (L/s)			
		TW1-10	P10B	P10A	Total
2011-10-04 13:45	0.0000	40	0	0	40
2011-10-07 12:00	2.9271	35	0	0	35
2011-10-07 12:30	2.9479	35	40	0	75
2011-10-12 9:00	7.8021	0	40	0	40
2011-10-12 10:15	7.8542	35	40	0	75
2011-10-12 11:00	7.8854	35	40	70	145
2011-10-12 11:15	7.8958	30	40	70	140
2011-10-12 11:25	7.9028	25	40	70	135
2011-10-12 11:35	7.9097	20	40	70	130
2011-10-12 12:40	7.9549	15	40	70	125
2011-10-12 13:45	8.0000	20	40	70	130
2011-10-17 16:00	13.0938	0	40	70	110
2011-10-19 10:00	14.8438	0	0	0	0

Table 5-7b Pumping rates of others municipal wells in the area during the 15-day test

Well ID	Pumping rate during the test (L/s)	Rate used in the model (L/s)
G5	24	24
P9	14	14
P15	14 - 14.5	14.25
H4	15	15
H3	0	0
G6	15	15
G16	Pumped on demand, average rate specified	23
P10	0	0
P11	0	0
P17	Pumped on demand, variable rate specified	27 to 32 (average 18.0)

The calibration of the model to match the observed drawdowns during the 15-day Pinebush pumping test has been conducted through 'fine-tuning' of the hydraulic conductivity distribution in the immediate vicinity of the test wells. Care was taken to ensure that any adjustments did not cause the match to the other sets of calibration targets to deteriorate. The finite element mesh around the test wells was refined to accommodate relatively small-scale adjustments of hydraulic conductivity values. The adjustments of the aquifer hydraulic conductivities made during the transient calibration have been guided by an independent re-interpretation of the pumping test data reported in **Appendix E**. The re-interpretation of the first part of the 15-day Pinebush, during which only TW1-10 was pumping, suggest that TW1-10 is located within a zone with a transmissivity of about 520 m^2/d. The circular zone of elevated transmissivity around TW1-10 has been incorporated in the numerical model during the transient calibration. The radius of this "inner" zone is inferred to be about 200 m; beyond 200 m the transmissivity of the "outer" zone is estimated to be 150 m^2/d. Well P10A is located about 240 m from TW1-10. A transmissivity of 160 m^2/d estimated from the independent analysis of a pumping test conducted at P10A is consistent with the estimate obtained for the outer zone around TW1-10. A synthesis of the results of the independent analyses of the 15-day Pinebush test and the P10A test suggests that P10A is located in a transition zone between the TW1-10 "inner" and "outer" transmissivity zones.

The observed and simulated drawdowns at the end of each pumping stage are compared in the scatterplots presented in **Figure 25**. The statistics of the comparison between the observed and simulated drawdowns are also presented on the scatterplots. The final calibrated model matches closely the observed drawdowns at the end of all four pumping stages. The Normalized Root Mean Square (nRMSE) Residuals for the four phases range from 1.7% to 5.7%, well below the acceptance criterion of 10% suggested by Spitz and Moreno (1996).

Selected hydrographs showing the evolution of drawdowns at the observation wells screened in the deep bedrock (Gasport Formation and Goat Island Formation), upper bedrock (Reformatory Quarry Member and Guelph Formation), Contact Aquifer, and overburden during the 15-day Pinebush test are presented in **Figure 26** through **Figure 29.** The results suggest that the transient model effectively reproduces the trends observed during the 15-day Pinebush pumping test.

The calibrated model does not simulate the fluctuations at some monitoring locations, for example, TW3A, B-05 and OW3C-95. The fluctuations observed at these wells are likely caused by changes in pumping rates at municipal production wells other than the three test wells. It is likely that the fluctuations at TW3A and B-05 are due to variations in the pumping rate of G16. The abrupt decrease in the drawdown at OW3C-95 is likely due to a brief shutdown of P15. The plots of the hydrographs for observation wells in the overburden presented in **Figure 29** show that the model slightly overestimates the impact of pumping in the shallow system. This may be due to variations in climate associated with variations in precipitation and the levels of surface water features.

The simulated drawdown contours at the end of the 15-day Pinebush test in the Contact Aquifer and the Middle Gasport Formation are presented in **Figure 30** and **Figure 31**, respectively. The observed final drawdowns are also plotted in the figures. In general, the simulated and observed drawdowns are consistent spatially.

The inferred cumulative transmissivity at the inner zone around TW1-10 in the final calibrated model is about 210 m^2/d; while the transmissivity of the outer zone is about 120 m^2/d. The final inferred transmissivity of the inner zone around TW1-10 is about half of the value concluded from the re-interpretation of the pumping test, and the transmissivity of the outer zone is consistent with the result of the re-interpretation of the pumping test. During the model calibration, a new circular zone with a radius of 17.5 m has been added around P10A; this zone has been required to match the drawdowns at C-PB-TW1ABC-09. The inferred transmissivity of the circular zone around P10A is about 340 m^2/d. In general, the model inferred transmissivities around the site of the Pinebush pumping test are consistent with the independent interpretation of the pumping test.

5.8 Sensitivity Analyses for the Calibrated Model

Extensive sensitivity analyses have been conducted during the development and calibration of the Cambridge East model. The zones of hydraulic conductivity and the values assigned to those zones have been adjusted to reproduce the overall trends in the observations and to match the various sets of calibration targets. Formal sensitivity analyses have also been conducted. The formal sensitivity analyses have been conducted with the computer-assisted calibration code, PEST (Doherty, 2005). The sensitivity analyses have been conducted to assist in identifying those parameters that have the greatest influence on the match to the calibration targets. The analyses have also been valuable to identify those parameters that have the potential to degrade the performance of the parameter estimation process through lack of sensitivity to model outcomes. Particular emphasis has been placed on understanding why particular zones have the greatest impact on the calibration. These efforts assisted in identifying areas that became the focus for additional review of the conceptual model and refined calibration, and for identifying areas of the model where subsequent adjustments did not yield any significant improvement in the calibration.

Detailed results of the PEST sensitivity analyses are presented in **Appendix F**. The results of the sensitivity analysis are reported in terms of the relative changes in the sum of the squared residuals. The sum of the squared residuals statistic is the measure of the overall goodness-of-fit with the computer-assisted calibration, and is referred to as the *objective function*. The relative changes are referred to as the "sensitivities"'; hydraulic conductivities with the largest sensitivities are those that have the greatest impact on the overall match to the calibration targets.

The overall match to the steady-state calibration targets is sensitive to the properties assigned to both the overburden and the bedrock. The goodness-of-fit of the model is particularly sensitive to the horizontal and vertical hydraulic conductivities specified for the materials representing the bed of Puslinch Lake. These parameters control both the leakage between Puslinch Lake and the groundwater system and the simulated lake level. The properties of the bedrock units that most affect the overall match to the calibration targets are the vertical hydraulic conductivities of the Reformatory Quarry Member and the Vinemount Member of the Eramosa Formation. The vertical hydraulic conductivities control the differences in hydraulic heads simulated between the Upper and Lower bedrock aquifers in the southeast area of Cambridge East.

6 TESTING OF THE GROUNDWATER FLOW MODEL

The Cambridge groundwater model has been calibrated for steady-state conditions using average pumping conditions in 2003. The model has been further calibrated for transient conditions using the data from three intensively monitored pumping tests (Cambridge East 28-day test, Clyde Park 6-day test, and the Pinebush 15-day test). Additional analyses have been conducted to confirm that the calibrated model can be applied with confidence to predict the potential effects of future changes in municipal pumping. The calibrated model is applied with no change of model parameters to simulate two additional conditions: the increase in pumping from the Clemens Mill Well Field that occurred in the mid–1990s, and a pumping test conducted at the recently installed deep bedrock well G5A.

6.1 Simulation of the Mid-1990s Increase in Municipal Pumping

To confirm that the calibrated model may provide reliable predictions of the potential long-term effects of future changes in municipal pumping, the model is applied to simulate the increase in pumping from the Clemens Mill Well Field that occurred in the mid–1990s. The locations of the wells in the Clemens Mill Well Field and the surrounding area that existed in the mid-1990s are shown in **Figure 32**. The annual pumping rates at the Clemens Mill Well Field between 1986 and 2009 are plotted in **Figure 33**. The average pumping rate for G6 up to 1995 was about 800 m^3/d. Wells G16, G17 and G18 began operating as part of the IUS in 1995. The average cumulative rate for the well field was about 4,700 m^3/d between 1995 and 2009, representing an increase of about 3,900 m^3/d with respect to conditions prior to 1995.

The following assumptions are incorporated in the analysis of the increase in pumping from the Clemens Mill Well Field:

- Long-term conditions are approximately steady before, and after, the increase in pumping in the mid-1990s;
- The average recharge rate for 2003 is representative of long–term conditions; and
- The average pumping rates for 2003 are representative of long–term production from municipal wells outside of the Clemens Mill area.

The analysis is accomplished by simulating two steady-state conditions:

- Pumping only from G6, at 1,240 m^3/day, the average pumping rate for 2003; and
- Pumping from G6, G16, G17 and G18 at rates of 1,240 m^3/day, 1,640 m^3/day, 1,360 m^3/day, and 1,350 m^3/day, respectively. These are the average rates observed between 1995 and 2009.

The pumping rates for all other municipal production wells are held steady at their average rates during 2003. The long-term changes in water levels between the two conditions are estimated by subtracting the average water levels observed in 2009 from the average water levels observed in 1995. The simulated long-term declines in hydraulic heads in the Contact Aquifer and the middle Gasport Formation due to the increase in pumping are presented in **Figure 34a** and **Figure 34b,** respectively.

The long-term changes in water levels caused by the increase in pumping are assessed by examining the records of groundwater levels at four locations in the vicinity of the Clemens Mill Well Field:

- C2;
- OW4-92 (A, B/C);
- OW5-94 (B, F); and
- OW6-94 (B/C, D).

The four monitoring locations have been selected for the assessment because the wells at these locations are included in the Region of Waterloo long–term monitoring network and observations were available for the period preceding the increase in pumping. The locations of the wells are shown in **Figure 32** and data on the open intervals for these wells are presented on **Table 6-1**.

Table 6-1 Observation Wells for the Increase in Pumping from the Clemens Mill Well Field

Well ID	Screened formation
C2	Overburden
OW4-92A	Guelph Formation
OW4-92B	Overburden
OW4-92C	Overburden
OW5-94B	Middle Gasport Formation
OW5-94F	Overburden
OW6-94B	Middle Gasport Formation
OW6-94C	Upper Gasport Formation
OW6-94D	Reformatory Quarry Member

The long-term hydrographs for the observation wells are plotted in **Figure 35** through **Figure 40**. The solid red line in each figure represents the estimated average level prior to the increase in pumping. The lower solid green line in each figure is calculated by subtracting the simulated decline in the water level caused by pumping. The groundwater levels both before and after the increase in pumping exhibit seasonal fluctuations. Therefore, it is not possible to identify definitively the magnitudes of the long-term changes in water levels that can be attributed solely to the increase in pumping. Despite this reservation, the results shown in **Figure 35** through **Figure 40** suggest that there is generally good agreement between the observed and simulated changes in long-term average water levels.

As shown in **Figure 39**, the simulated drawdown at OW5F-94 is less than estimated from the observations. This observation well is located relatively close to the Pinebush Well Field and it is possible that a portion of the observed drawdown is actually due to variations in pumping at the Pinebush Well Field. As shown in **Figure 41**, the pumping rate for well P10 in the Pinebush Well Field increased by a factor of about four between 1995 and 1997. The potential effects of this increase have not been accounted for in the analysis of the increase in pumping from the Clemens Mill wells. OW5F-94 is an overburden well and P10 is open across the Contact Aquifer. The results shown in **Figure 39** may suggest that water levels at OW5-94 are affected by pumping from both the Clemens Mill and Pinebush wells.

The results from the simulation of the increase in pumping from the Clemens Mill Well Field are important for two reasons. First, the results suggest that the calibrated model is 'fit for purpose'. That is, the calibrated model can be used with confidence to predict the likely magnitudes of the long-term effects of alternative municipal schemes pumping. Second, the results suggest that pumping from one well field in the Cambridge East area is likely to affect conditions in adjacent well fields. Therefore, it will generally be appropriate to consider all of the well fields in an integrated analysis to account for the potential interference between well fields.

6.2 Simulation of G5A pumping test

The present analyses consider the effects of municipal pumping from Cambridge East wells in their regional context. Therefore, it is important to confirm that the calibrated groundwater model provides reliable results outside of the immediate focus area of the Cambridge East well fields (Pinebush, Clemens Mill and Shade's Mills). Towards this end, the calibrated model is used to conduct a simplified simulation of a pumping test conducted at test production G5A, located west of the Pinebush Well Field. No model parameters are adjusted for the simulation; therefore, this is a second independent test case for the model.

The locations of G5A and the observation wells for the test are shown in **Figure 42**. G5A was pumped at an average rate of 20 L/s between February 25 and March 3, 2009. For the simplified analysis it is assumed that with the exception of pumping from G5A, all conditions remained constant during the test period. The analysis is accomplished by simulating two conditions:

- Steady-state simulation with no pumping from G5A (2003 calibration conditions); and
- Transient simulation of pumping from G5A at a rate of 20 L/s, with all of the other wells pumping at their 2003 average rates.

To simplify the assessment of the calibrated model, only the drawdowns at the end of pumping are compared with the simulation results. The simulated drawdowns at the end of the pumping test are compared with the final drawdowns at 15 observation wells. The observed and simulated drawdowns at the end of pumping are compared in **Figure 43.** The magnitudes of the drawdowns calculated with the calibrated model are similar in magnitude to the observations. As no parameters were adjusted to achieve this match, the results of the simulation suggest that the model may be used for predictive analyses beyond the immediate vicinity of the Cambridge East well fields.

7 ENVIRONMENTAL ASSESSMENT SIMULATIONS

7.1 Overview

In total, ten scenarios have been developed and simulated: a Base Case, three steady-state scenarios (Scenarios 1-3), and six transient scenarios (Scenarios A-F). As will be discussed in Section 7.2, the Base Case is intended to represent current average conditions. The predictions for alternative pumping schemes considered in the scenarios are presented in terms of potential changes with respect to the Base Case. A consistent approach is adopted to present the potential changes for each scenario. The following results are presented for each scenario:

- Predicted changes in groundwater levels (drawdowns) in the shallow groundwater system. The results are presented for the Contact Aquifer, as groundwater levels in the Contact Aquifer are representative of the regional water table;
- Predicted changes in groundwater levels (drawdowns) in the deeper bedrock groundwater system. The results are presented for the Middle Gasport Formation, as this is the major hydrostratigraphic unit from which the deeper municipal wells obtain their supplies;
- Predicted changes in the components of the local water balances for the Upper and Lower Bedrock groundwater systems within the Zone of Influence (ZOI) of the Cambridge East municipal production wells;
- Predicted changes in the vertical flows across the top of the Guelph Formation for the individual subwatersheds within the ZOI of the Cambridge East municipal production wells;
- Predicted changes in groundwater discharge to streams; and
- Predicted changes to the components of the water balance for Puslinch Lake.

The water balance calculations are developed for an area referred to as the Zone of Influence (ZOI). The limits are the ZOI are shown in **Figure 44**. The ZOI is defined here as the area within the Middle Gasport Formation that is potentially affected by pumping from municipal production wells in the Cambridge East area. The effects of pumping propagate furthest in the Middle Gasport Formation. The ZOI shown in Figure 44 has been delineated by integrating observed responses to pumping with results obtained with the calibrated model. The perimeter of the ZOI shown in **Figure 44** is irregular; the ZOI follows the edges of elements in the model mesh to maximize the accuracy of the internal FEFLOW water balance calculations.

In the Cambridge East area, the Vinemount Member of the Eramosa Formation is a major regional aquitard that separates the shallow and deep groundwater flow systems. The results of water balance calculations within the ZOI are developed for the "Upper Bedrock Aquifer" and the "Lower Bedrock Aquifer". Here "Upper Bedrock Aquifer" refers to a lumping of the Guelph Formation (GU) and the Reformatory Quarry Member of the Eramosa Formation (RQ) and "Lower Bedrock Aquifer" refers to the Goat Island Formation (GI) and the Gasport Formation (GS). The components of the water balance calculations within the ZOI are shown schematically in **Figure 45**. The components of the water balance reported for the Base Case and each scenario are defined below.

Upper Bedrock Aquifer

- Vertical flow across the top face: Net vertical flow across the top of the Upper Bedrock Aquifer
- Horizontal flow: Net lateral flow into the Upper Bedrock Aquifer around the perimeter of the ZOI
- Vertical flow across the bottom face: Net vertical flow across the bottom of the Upper Bedrock Aquifer
- Pumping: Cumulative pumping from municipal production wells in the Upper Bedrock Aquifer

Lower Bedrock Aquifer

- Vertical flow across the top face: Net vertical flow across the top of the Lower Bedrock Aquifer
- Horizontal flow: Net lateral flow into the Lower Bedrock Aquifer around the perimeter of the ZOI
- Pumping: Cumulative pumping from municipal production wells in the Lower Bedrock Aquifer

Predicted changes in the vertical flows for the individual subwatersheds are presented in terms of the vertical flows across the top of the Guelph Formation. The subwatersheds within the ZOI are shown in **Figure 46**. The ZOI extends over portions of the watersheds of the Speed River and the middle section of the Grand River, four creeks (Irish, Mill, Moffat and Fairchild) and a small area in Halton Region.

Predicted changes in streamflows are reported in terms of the simulated groundwater discharges to the stream reaches for which data were available for the model calibration.

Potential impacts to Puslinch Lake are reported in terms of changes in the components of the water balance for the lake between any scenario and the Base Case. The net lateral flows and net vertical flows to Puslinch Lake are reported for each scenario.

7.2 Base Case Simulation (2008 average conditions)

In consultation with the Region of Waterloo, average pumping rates during 2008 have been selected to provide baseline conditions for the assessment of the predicted changes for each scenario. With the exception of municipal supply wells G38 and G39 in the Shade's Mills Well Field, the average rates in 2008 are representative of current conditions. G38 and G39 were not pumped in 2007 or 2008; for the Base Case their average rates between 2005 and 2006 are specified. The Base Case pumping rates for the municipal pumping wells in the Cambridge East area are listed on **Table 7-1**. The total pumping for the Base Case is 547 L/s, with 251 L/s from the Cambridge East well fields. The total pumping from the Cambridge East well fields is about 15% higher in 2008 than for the average 2003 conditions considered during the model calibration.

Table 7-1 Pumping Rates for the Base Case

Well ID	Well Field	Base case (2008 average)
		Average Day (L/s)
G4	Blair Rd.	11
G4A		0
G6	Clemens Mill	16
G16		19
G17		23
G18		12
Portuguese Club (CMPW1-08)		0
Cedarbrook (CMPW2-06)		0
P6	Dunbar Rd.	10
G9	Elgin St.	15
H3	Hespeler	7
H3A		0
H4		0
H4A		0
H5		4
H5A		0
G1	Middleton St.	40
G1A		46
G2		62
G3		39
G14		37
Clyde Park (NDPW1-08)	North Dumfries	0
P9	Pinebush	17
P15		11
G5		19
G5A		0
P10		34
P10B		0
P10A		0
TW1-10		0
P11		13
P17		9
Witmer Park		0
CanAmera		0
G7	Shade's Mills	27
G8		14
G38		6
G39		30
G15	Willard	25
Total		547

The contours of simulated groundwater levels in the Contact Aquifer for the Base Case are shown in **Figure 47**. The simulated groundwater levels in the Middle Gasport Formation are shown in **Figure 48**. The insets in both figures show conditions in the immediate vicinity of the Cambridge East well fields. In general, the simulated heads are similar to those obtained with the model calibrated for 2003 conditions. There are local differences at the locations where the average pumping rates are significantly different between 2003 and 2008 (wells G5, P15, P10, G17, G18, G7, G8 and G38).

The components of the water balance for the ZOI for the Base Case are assembled on **Table 7-2**. The number of significant figures in the values is not intended to suggest the accuracy of the analyses; rather, the values are reported so that they can be checked against the results extracted from the FEFLOW output. This approach is adopted for the reporting of all subsequent results of the analyses.

Table 7-2 Calculated water balance components - Base Case

Upper Bedrock aquifers			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	4998	-9005	-4007
Vertical flow across the top of the Guelph Formation	39332	-15270	24062
Vertical flow across the bottom of the RQ Member	6136	-14286	-8150
Pumping		-5541	-5541
Specified-head nodes		-6359	-6359
Lower Bedrock aquifers			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	4908	-1579	3328
Vertical flow across the top of the Goat Island Formation	11583	-4939	6645
Pumping		-9972	-9972

The predicted vertical flows across the top of the Guelph Formation for the individual subwatersheds within the ZOI are listed on **Table 7-3**. The reported net flows are calculated as the difference between the downwards flow and the upwards flow. A positive value denotes net downwards flow.

Table 7-3 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Base Case

Subwatershed	Vertical flow (m^3/d)		
	Downwards	Upwards	Net flow*
Lower Speed River(84)**	3593	2646	947
Irish Creek(235)	8149	1012	7137
Lower Speed River(229)	3974	2889	1085
Middle Grand(234)	8672	3270	5402
Middle Grand(237)	1717	753	964
Lower Mill Creek(236)	2339	1230	1109
Moffat Creek(239)	1156	373	783
Fairchild Creek(241)	6064	2804	3260
Middle Grand(242)	113	15	98
Fairchild Creek(240)	5375	2830	2545
Upper Mill Creek(90)	822	1055	-233
Outside GRCA boundary(999)	1800	466	1334

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.

The calculated groundwater discharges to streams for the Base Case are listed on **Table 7-4**.

Table 7-4 Simulated groundwater discharges to streams – Base Case

Stations	Station description	Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges		
Aberfoyle Creek (3AQ131)	GRCA gauge Aberfoyle Creek at Twp Rd. 7	19945
Mill Creek (2GAC19)	GRCA gauge Mill Creek at Sideroad 10	45739
Spotflow measurements		
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd. and Kossuth Rd.	6984
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd. and Blair Rd.	1677

The calculated variable components of the Puslinch Lake water balance are listed on **Table 7-5**.

Table 7-5 Simulation Results for Puslinch Lake – Base Case

Quantity	Value
Net lateral flow (negative for outflow)	-1088 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d

7.3 Steady-state scenarios: Long-term development of an additional 38 L/s

Three steady-state scenarios are developed and analysed. The analyses have been designed to predict the potential long-term effects of the development of 38 L/s of additional groundwater supplies from Cambridge East wells. The additional municipal pumping accommodates a predicted increase in water demand between 2008 and the projected average rates for 2031 in the Region of Waterloo Tier Three Water Budget Study. The three scenarios are designed to meet the increase in water between 2008 and 2031 from different combinations of pumping wells in Cambridge East:

- Scenario 1: Obtaining the additional 38 L/s from a net increase in the pumping from existing municipal wells at rates to match the Region's Master Water Supply Study and Tier 3 modeling;
- Scenario 2: Obtaining the additional 38 L/s from wells around P10 at the Pinebush Well Field (P10A, P10B, and TW1-10); and
- Scenario 3: Obtaining the additional 38 L/s from the Cedarbrook well (CMPW2-06) at the Clemens Mill Well Field.

7.3.1 Steady-state Scenario 1

Scenario 1 is designed to satisfy an increase of 38 L/s with respect to the Base Case (2008 average conditions) with changes in pumping that are consistent with the Region's Master Water Supply Study and the Tier 3 modeling that has been conducted for the Region. The pumping rates for Scenario 1 are listed on **Table 7-6**. The total pumping for Scenario 1 is increased from 547 L/s to 585 L/s. The changes in Scenario 1 pumping rates with respect to the Base Case (2008 average conditions) are summarized for each well field below.

Blair Rd. Well Field (total pumping increased by 9 L/s)

- Increase pumping at G4 from 11 L/s to 20 L/s, an increase of 9 L/s

Clemens Mill Well Field (total pumping increased by 14 L/s)

- Reduce pumping at G6 from 16 L/s to 10 L/s, a decrease of 6 L/s
- Increase pumping at G16 from 19 L/s to 34 L/s, an increase of 15 L/s
- Increase pumping at G17 from 23 L/s to 25 L/s, an increase of 2 L/s
- Increase pumping at G18 from 12 L/s to 15 L/s, an increase of 3 L/s

Durbar Rd. Well Field (total pumping reduced by 10 L/s)

- Stop pumping P6, which in 2008 pumped at an average rate of 10 L/s

Elgin St. Well Field (total pumping reduced by 15 L/s)

- Stop pumping G9, which in 2008 pumped at an average rate of 15 L/s

Hespeler Well Field (total pumping increased by 24.1 L/s)

- Increase pumping at H3 from 6.5 L/s to 10 L/s, an increase of 3.5 L/s
- Resume pumping at H4 at an average rate of 15 L/s
- Increase pumping at H5 from 4.4 L/s to 10 L/s, an increase of 5.6 L/s

Pinebush Well Field (total pumping reduced by 2 L/s)

- Reduce the pumping at P9 from 17 L/s to 15 L/s, a decrease of 2 L/s
- Increase the pumping at P15 from 11 L/s to 15 L/s, an increase of 4 L/s
- Replace G5 with G5A, and reduce the pumping rate from 19 L/s to 15 L/s, a reduction of 4 L/s
- Increase the pumping at P10 from 34 L/s to 36 L/s, an increase of 2 L/s
- Increase pumping at P11 from 13 L/s to 20 L/s, an increase of by 7 L/s
- Stop pumping at P17, which in 2008 pumped at an average rate of 9 L/s

Shade's Mills Well Field (total pumping reduced by 2 L/s)

- Reduce pumping at G7 from 27 L/s to 20 L/s, a decrease of 7 L/s
- Reduce pumping at G8 from 14 L/s to 10 L/s, a reduction of 4 L/s
- Increase pumping at G38 from 6 L/s to 15 L/s, an increase of 9 L/s
- Continue pumping G39 at a rate of 30 L/s

Willard Well Field (total pumping increased by 5 L/s)

- Increase the pumping rate at G15 from 25 L/s to 30 L/s, an increase of 5 L/s

Middleton Street Well Field (total pumping increased by 15 L/s)

- Increase pumping at G1 from 40.4 L/s to 60 L/s, an increase of 19.6 L/s
- Reduce pumping at G1A from 46.2 L/s to 20 L/s, a reduction of 26.2 L/s
- Increase pumping at G2 from 62.1 L/s to 80 L/s, an increase of 17.9 L/s
- Increase pumping at G3 from 39.3 L/s to 55 L/s, and increase of 15.7 L/s
- Reduce pumping at G14 from 37.1 L/s to 25 L/s, a reduction of 12.1 L/s

Table 7-6 Pumping Rates for Scenario 1

Well ID	Well Field	Base case	Scenario 1
		Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	11	20
G4A		0	0
G6	Clemens Mill	16	10
G16		19	34
G17		23	25
G18		12	15
Portuguese Club (CMPW1-08)		0	0
Cedarbrook (CMPW2-06)		0	0
P6	Dunbar Rd.	10	0
G9	Elgin St.	15	0
H3	Hespeler	7	10
H3A		0	0
H4		0	15
H4A		0	0
H5		4	10
H5A		0	0
G1	Middleton St.	40	60
G1A		46	20
G2		62	80
G3		39	55
G14		37	25
Clyde Park (NDPW1-08)	North Dumfries	0	0
P9	Pinebush	17	15
P15		11	15
G5		19	0
G5A		0	15
P10		34	36
P10B		0	0
P10A		0	0
TW1-10		0	0
P11		13	20
P17		9	0
Witmer Park		0	0
CanAmera		0	0
G7	Shade's Mills	27	20
G8		14	10
G38		6	15
G39		30	30
G15	Willard	25	30
Total		547	585

Contours of simulated drawdowns for Scenario 1 are shown in **Figure 49** and **Figure 50**. The drawdowns for each scenario are calculated as the differences between the simulated hydraulic heads for Scenario 1 and the Base Case, with positive values denoting a decline with respect to the Base Case. Contours of simulated drawdowns in the Contact Aquifer are shown in **Figure 49**. In the outer portion of the figure, the smallest drawdown contour is 1 m, which represents the likely minimum drawdown that can be distinguished from natural seasonal fluctuations in groundwater levels, adopted as the criterion for the delineation of the WHPA-Q1 for the Region of Waterloo Tier Three Water Budget analyses. In the Contact Aquifer, the predicted drawdowns exceed 1 m only in the immediate vicinities of H4, G16 and G38. The 1.0 m drawdown contour around each of these wells is shown but is similar to the size of the symbol that denotes the location of the wells. To better illustrate the extents of the drawdown cones, the drawdown contours between 0.5 m and 1.0 m are shown in the inset.

The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 50**. There are two separate drawdown cones in the Middle Gasport Formation: a cone around G5A with a maximum drawdown of 7.5 m, and a second cone around G16 with a maximum drawdown of 4 m.

To simplify the presentation of the results of the water balance calculations, the bedrock units are lumped into an Upper Bedrock system and a Lower Bedrock system. The Upper Bedrock system comprises the Contact Aquifer, the Guelph Formation and the Reformatory Quarry Member of the Eramosa Formation. The Lower Bedrock system comprises the Goat Island Formation and the Gasport Formation. These two systems are separated by the Vinemount Member of the Eramosa Formation. The separation is natural; where the Vinemount Member is present it is a regionally significant aquitard that acts as a barrier to vertical flow between the upper and lower bedrock systems. The Contact Aquifer is the dominant unit in the Upper Bedrock system. The middle portion of the Gasport Formation is the dominant unit in the Lower Bedrock system.

The components of the water balances for the Upper and Lower Bedrock systems within the ZOI are listed for Scenario 1 on **Table 7-7**. There is a net increase in cumulative municipal pumping from the Upper and Lower Bedrock systems of 4,787 m^3/d (55 L/s), which includes a shift of the pumping from G5 in the Contact Aquifer to G5A in the Gasport Formation. The increase in pumping from the Upper Bedrock system is balanced by an increase in the vertical inflow from the overburden and Contact Aquifer and a reduction in the horizontal outflow. The increase in the vertical flow from the overburden and Contact Aquifer accounts for about 90% of the increase in the cumulative pumping from the Base Case. The increase in pumping from the Lower Bedrock system induces an increase in the vertical inflow across the top of the Goat Island Formation and an increase in the horizontal inflow to the ZOI. About 70% of the increase in pumping in the Lower Bedrock system is accounted for by the increased horizontal inflow.

Table 7-7 Calculated water balance components – Scenario 1

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5011	-8641	-3630
Vertical flow across the top of the Guelph Formation	40932	-14012	26921
Vertical flow across the bottom of the RQ Member	5557	-14986	-9429
Pumping		-7778	-7778
Specified-head nodes		-6088	-6088
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	6739	-1682	5057
Vertical flow across the top of the Goat Island Formation	11794	-4330	7464
Pumping		-12522	-12522

The calculated vertical flows across the top of the Guelph Formation for the individual subwatersheds are listed on **Table 7-8** for Scenario 1. As indicated in the note beneath the table, a positive quantity denotes net downwards flow across the top of the Guelph Formation.

The reported values of "Fraction of change (%)" for each subwatershed are calculated for Scenario A according to:

$$\text{Fraction of change (\%)} = \frac{\text{Net flow [Scenario A]} - \text{Net flow [Base Case]}}{\text{Cumulative change in net flow for all subwatersheds [Scenario A]}} \times 100$$

For example, for the Lower Speed River subwatershed (#84), a net downwards flow of 2099 m^3/day is predicted for Scenario 1, which represents an increase of 1152 m^3/day over the value of 947 m^3/day calculated for the Base Case. The cumulative change in the net downwards flow for Scenario 1 is 3096 m^3/day. Therefore, the change in the downwards flow in subwatershed #84 represents 37% of the cumulative change in the downwards flow. The positive value denotes a predicted increase in the net flow downwards across the top of the Guelph Formation within subwatershed #84.

The changes with respect to the Base Case are reported relative to the predicted cumulative change for all of the subwatersheds with the water balance calculation area (ZOI). The "Fraction of change (%)"values of the groundwater flow across the top of the Guelph Formation are presented for each scenario to provide indications of the relative magnitudes of the predicted changes in the individual subwatersheds. As shown on Table 7-8, for Scenario 1 the major changes with respect to the Base Case are predicted to occur in the Lower Speed River and Irish Creek subwatersheds.

Table 7-8 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario 1

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
84 (Lower Speed River)**	4306	2207	2099	37%
235 (Irish Creek)	8884	891	7993	28%
229 (Lower Speed River)	4202	2248	1954	28%
234 (Middle Grand)	8294	2816	5478	3%
237 (Middle Grand)	1597	806	791	-6%
236 (Lower Mill Creek)	2597	1096	1501	13%
239 (Moffat Creek)	1016	406	610	-6%
241 (Fairchild Creek)	6096	2781	3315	2%
242 (Middle Grand)	112	16	96	-0.1%
240 (Fairchild Creek)	5381	2823	2558	0.4%
90 (Upper Mill Creek)	840	1042	-202	1%
Halton Region (999)	1800	466	1334	0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario 1 are listed on **Table 7-9**, and presented graphically in **Figure 51**. With the exception of Irish Creek, the predicted changes in the groundwater discharge to streams with respect to the Base Case are negligible.

Table 7-9 Simulated groundwater discharges to streams – Scenario 1

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario 1 Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19945	19940
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45639
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5626
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6984	6859
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd.	1677	1676

The calculated variable components of the Puslinch Lake water balance for Scenario 1 are listed on **Table 7-10**. The changes in the components are relatively small, on the order of 2% to 4%.

Table 7-10 Simulation Results for Puslinch Lake – Scenario 1

Quantity	Base Case value	Scenario 1 value
Net lateral flow (negative for outflow)	-1088 m^3/d	-1107 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d	-776 m^3/d

7.3.2 Steady-state Scenario 2

Scenario 2 is designed to satisfy an increase of 38 L/s with respect to the Base Case with additional pumping from wells near P10, including P10A, P10B and TW1-10. The pumping rates for Scenario 2 are listed on **Table 7-11**. As for Scenario 1, the total pumping for Scenario 2 is 585 L/s. The changes in Scenario 2 pumping with respect to the Base Case (2008 average conditions) are summarized below for each well field.

Blair Rd. Well Field (same as the Base Case)

Clemens Mill Well Field (same as the Base Case)

Durbar Rd. Well Field (same as the Base Case)

Elgin St. Well Field (same as the Base Case)

Hespeler Well Field (same as the Base Case)

Pinebush Well Field (total pumping increased by 38 L/s)

- Stop pumping at P10; (a reduction of 34 L/s with respect to the Base Case)
- Begin pumping at P10B at an average rate of 26 L/s (the well is not pumping in the Base Case)
- Begin pumping at P10A at an average rate of 29 L/s (the well is not pumping in the Base Case)
- Begin pumping at TW1-10 at an average rate of 17 L/s (the well is not pumping in the Base Case)

Shade's Mills Well Field (same as the Base Case)

Willard Well Field (same as the Base Case)

Middleton Street Well Field (same as the Base Case)

Table 7-11 Pumping Rates for the Cambridge East Municipal Wells – Scenario 2

Well ID	Well Field	Base case	Scenario 1	Scenario 2
		Average Day (L/s)	Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	11	20	11
G4A		0	0	0
G6	Clemens Mill	16	10	16
G16		19	34	19
G17		23	25	23
G18		12	15	12
Portuguese Club (CMPW1-08)		0	0	0
Cedarbrook (CMPW2-06)		0	0	0
P6	Dunbar Rd.	10	0	10
G9	Elgin St.	15	0	15
H3	Hespeler	7	10	7
H3A		0	0	0
H4		0	15	0
H4A		0	0	0
H5		4	10	4
H5A		0	0	0
G1	Middleton St.	40	60	40
G1A		46	20	46
G2		62	80	62
G3		39	55	39
G14		37	25	37
Clyde Park (NDPW1-08)	North Dumfries	0	0	0
P9	Pinebush	17	15	17
P15		11	15	11
G5		19	0	19
G5A		0	15	0
P10		34	36	0
P10B		0	0	26
P10A		0	0	29
TW1-10		0	0	17
P11		13	20	13
P17		9	0	9
Witmer Park		0	0	0
CanAmera		0	0	0
G7	Shade's Mills	27	20	27
G8		14	10	14
G38		6	15	6
G39		30	30	30
G15	Willard	25	30	25
Total		547	585	585

The contours of simulated drawdowns in the Contact Aquifer for Scenario 2 are shown in **Figure 52**. In the Contact Aquifer, the extent of the 1 m drawdown contours covers the entire Pinebush Well Field and the northern portion of the Clemens Mill Well Field. A maximum drawdown of 2.0 m is calculated around P10B. Following the approach adopted to present the predictions for Scenario 1, to better illustrate the extents of the drawdown cones, the drawdown contours between 0.5 m and 1.0 m are shown in the inset. It is important to note that these contours are presented only for visualization purposes. It is unlikely that a drawdown of 0.5 m could be distinguished from natural seasonal fluctuations in groundwater levels.

The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 53**. In the Middle Gasport Formation, the drawdown cone is centered around P10 and extends over the Pinebush Well Field. Maximum drawdowns in the aquifer of about 16.5 m and 11 m are predicted around P10A and TW1-10, respectively.

The components of the water balances for the Upper and Lower Bedrock systems within the Zone of Influence for Scenario 2 are assembled on **Table 7-12**. There is a net increase in cumulative pumping from the Upper and Lower Bedrock systems of 3,275 m^3/d, which corresponds to the increase in municipal pumping of 38 L/s. Pumping from the Upper Bedrock system is reduced with respect to the Base Case. In the Lower Bedrock system, the increase in pumping of 3,975 m^3/d (46 L/s) is balanced by an increase in the vertical flow from the overlying bedrock units and an increase in the horizontal inflow to the ZOI. The increase in pumping from the Lower Bedrock system induces an increase in the vertical inflow across the top of the Goat Island Formation and an increase in the horizontal inflow of about the same amount.

Table 7-12 Calculated water balance components – Scenario 2

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	4982	-9014	-4032
Vertical flow across the top of the Guelph Formation	41615	-15128	26487
Vertical flow across the bottom of the RQ Member	5516	-16834	-11319
Pumping		-4841	-4841
Specified-head nodes		-6303	-6303
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net= In + Out
Horizontal flow	6811	-1521	5290
Vertical flow across the top of the Goat Island Formation	12938	-4281	8657
Pumping		-13947	-13947

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario 2 are listed on **Table 7-13**. The major predicted change with respect to the Base Flow occurs in the Irish Creek subwatershed.

Table 7-13 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario 2

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	3895	2696	1199	10%
Irish Creek(235)	9328	790	8538	54%
Lower Speed River(229)	4543	3140	1403	12%
Middle Grand(234)	8707	3108	5599	8%
Middle Grand(237)	1708	728	980	0.6%
Lower Mill Creek(236)	2521	1162	1359	10%
Moffat Creek(239)	1211	361	850	3%
Fairchild Creek(241)	6094	2775	3319	2%
Middle Grand(242)	112	15	97	0%
Fairchild Creek(240)	5382	2821	2561	0.6%
Upper Mill Creek(90)	846	1037	-191	2%
Halton Region (999)	1800	466	1334	0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario 2 are listed on **Table 7-14**, and illustrated in **Figure 54**. With the exception of Irish Creek, the predicted changes in the groundwater discharge to streams are small and nearly identical to the results for Scenario 1.

Table 7-14 Simulated groundwater discharges to streams – Scenario 2

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario 2 Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19945	19938
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45596
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5432
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6984	6928
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1677	1676

The calculated variable components of the Puslinch Lake water balance for Scenario 2 are listed on **Table 7-15**. The similarities in the results of the water balance calculations for Scenarios 1 and 2 are consistent with the fact that the cumulative pumping rates for the Cambridge well fields are the same for both scenarios.

Table 7-15 Simulation Results for Puslinch Lake – Scenario 2

Quantity	Base Case value	Scenario 2 value
Net lateral flow (negative for outflow)	-1088 m^3/d	-1139 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d	-805 m^3/d

7.3.3 Steady-state Scenario 3

Scenarios 3 is designed to satisfy an increase of 38 L/s with respect to the Base Case with pumping from the Cedarbrook well (CMPW2-06). The Cedarbrook well is located east of the Clemens Mill Well Field. The pumping rates for Scenario 3 are listed on **Table 7-16**. The changes in Scenario 3 pumping with respect to the Base Case (2008 average conditions) are summarized below for each well field.

Blair Rd. Well Field (same as the Base Case)

Clemens Mill Well Field (total pumping is increased by 38 L/s)

Pumping from the Cedarbrook well at an average rate of 38 L/s is added (the well is not pumping in the Base Case).

Durbar Rd. Well Field (same as the Base Case)

Elgin St. Well Field (same as the Base Case)

Hespeler Well Field (same as the Base Case)

Pinebush Well Field (same as the Base Case)

Shade's Mills Well Field (same as the Base Case)

Willard Well Field (same as the Base Case)

Middleton Street Well Field (same as the Base Case)

Table 7-16 Pumping Rates for the Cambridge East Municipal Wells – Scenario 3

Well ID	Well Field	Base case	Scenario 1	Scenario 2	Scenario 3
		Average Day (L/s)	Average Day (L/s)	Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	11	20	11	11
G4A		0	0	0	0
G6	Clemens Mill	16	10	16	16
G16		19	34	19	19
G17		23	25	23	23
G18		12	15	12	12
Portuguese Club (CMPW1-08)		0	0	0	0
Cedarbrook (CMPW2-06)		0	0	0	38
P6	Dunbar Rd.	10	0	10	10
G9	Elgin St.	15	0	15	15
H3	Hespeler	7	10	7	7
H3A		0	0	0	0
H4		0	15	0	0
H4A		0	0	0	0
H5		4	10	4	4
H5A		0	0	0	0
G1	Middleton St.	40	60	40	40
G1A		46	20	46	46
G2		62	80	62	62
G3		39	55	39	39
G14		37	25	37	37
Clyde Park (NDPW1-08)	North Dumfries	0	0	0	0
P9	Pinebush	17	15	17	17
P15		11	15	11	11
G5		19	0	19	19
G5A		0	15	0	0
P10		34	36	0	34
P10B		0	0	26	0
P10A		0	0	29	0
TW1-10		0	0	17	0
P11		13	20	13	13
P17		9	0	9	9
Witmer Park		0	0	0	0
CanAmera		0	0	0	0
G7	Shade's Mills	27	20	27	27
G8		14	10	14	14
G38		6	15	6	6
G39		30	30	30	30
G15	Willard	25	30	25	25
Total		547	585	585	585

The contours of simulated drawdowns in the Contact Aquifer for Scenario 3 are shown in **Figure 55**. In the Contact Aquifer, the extent of the 1 m drawdown contour is limited to the Clemens Mill Well Field. To better illustrate the extents of the drawdown cones, the drawdown contours between 0.5 m and 1.0 m are shown in the inset. These are presented only for visualization purposes and not to suggest that a drawdown of 0.5 m could be distinguished from natural seasonal fluctuations in groundwater levels.

The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 56**. In the Middle Gasport Formation, the drawdown cone is centered on the Cedarbrook well and covers the eastern portion of the Clemens Mill Well Field extends across the Pinebush Well Field. A maximum drawdown of about 7 m in the Middle Gasport Formation is predicted.

The components of the water balances for the Upper and Lower Bedrock aquifers within the Zone of Influence for Scenario 3 are assembled on **Table 7-17**. There is no change in the pumping from the Upper Bedrock system. The change in the pumping from the Lower Bedrock system is 3,283 m^3/d, which corresponds exactly to the increase in municipal pumping of 38 L/s. This is consistent with the assignment of all of the additional pumping to the Cedarbrook well, which is open from the Goat Island Formation to the Middle Gasport Formation. In the Upper Bedrock system, the increase in the inflow from the overburden and Contact Aquifer is balanced by the increased vertical outflow from the Reformatory Quarry Member. In the Lower Bedrock system, 40% of the increase in pumping is balanced by an increase in the vertical flow across the top of the Goat Island Formation. The remaining 60% of the increase in pumping is balanced by an increase in the horizontal inflow into the ZOI.

Table 7-17 Calculated water balance components – Scenario 3

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net= In + Out
Horizontal flow	4982	-8988	-4007
Vertical flow across the top of Guelph Formation	41416	-14897	26519
Vertical flow across the bottom of RQ Member	5241	-15899	-10658
Pumping		-5541	-5541
Specified-head nodes		-6317	-6317
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	6796	-1499	5297
Vertical flow across the top of Goat Island Formation	11953	-3994	7959
Pumping		-13255	-13255

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario 3 are listed on **Table 7-18**. The major changes with respect to the Base Case occur in the Lower Speed River, Irish Creek and the Middle Grand subwatersheds.

Table 7-18 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario 3

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	3715	-2648	6363	34.0%
Irish Creek(235)	9241	-845	10086	18.5%
Lower Speed River(229)	4055	2998	1057	-0.2%
Middle Grand(234)	8665	-3159	11824	40.3%
Middle Grand(237)	1720	714	1006	0.3%
Lower Mill Creek(236)	2870	1088	1782	4.2%
Moffat Creek(239)	1322	343	979	1.2%
Fairchild Creek(241)	6150	2734	3416	1.0%
Middle Grand(242)	113	15	98	0.0%
Fairchild Creek(240)	5393	2809	2584	0.2%
Upper Mill Creek(90)	865	1025	-160	0.5%
Halton Region (999)	1801	466	1335	0.0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario 3 are listed on **Table 7-19** and shown graphically in **Figure 57**. The only appreciable difference between the Base Case and Scenario 3 is the reduction in the net groundwater discharge to Irish Creek. In the present context, "appreciable" means "likely to be detected in a long-term streamflow record".

Table 7-19 Simulated groundwater discharges to streams – Scenario 3

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario 3 Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19944	19935
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45520
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5415
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6984	6949
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1677	1676

The calculated variable components of the Puslinch Lake water balance for Scenario 3 are listed on **Table 7-20**. The similarities in the results of the water balance calculations for Scenarios 1, 2 and 3 and 2 are consistent with the fact that the cumulative pumping rates for the Cambridge well fields are the same for all three scenarios.

Table 7-20 Simulation Results for Puslinch Lake – Scenario 3

Quantity	Base Case value	Scenario 3 value
Net lateral flow (negative for outflow)	-1088 m^3/d	-1137 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d	-808 m^3/d

7.4 Transient scenarios

Six transient scenarios have been simulated. These scenarios have been developed to assess the redundancy of supply for the Integrated Urban System (IUS) in the Cambridge area. In particular, the scenarios are developed to predict the potential changes arising after two years of additional pumping from Cambridge East wells to make up for a shutdown of Middleton Street well G2. G2 has the highest pumping rate of all wells in the Cambridge East model area and thus a shut-down of G2 represents a "worse-case" scenario. A duration of two years has been specified as representative of the time required to restore the pumping at the Middleton Street Well Field. There are two sets of transient simulations, incorporating different assumptions regarding the time when pumping from well G2 stops.

Redundancy Scenarios A-C

For the transient scenarios A, B and C, pumping from G2 is assumed to stop while all of the municipal supply wells are pumping at their 2031 projected rates. The 2031 projected pumping rate for G2 is 80 L/s. The replacement 80 L/s is accommodated as follows:

- Scenario A: To the extent possible, the shutdown of G2 is replaced with additional pumping from the Pinebush Well Field, with the remainder from the Clemens Mill Well Field;
- Scenario B: To the extent possible, the shutdown of G2 is replaced with additional pumping from the Clemens Mill Well Field, with the remainder from the Pinebush Well Field, and
- Scenario C: To the extent possible, the shutdown of G2 is replaced with additional pumping from the Clemens Mill Well Field but with well G18 inactive, with the remainder from the Pinebush Well Field.

The initial conditions for transient scenarios A, B and C are specified as the results of the steady-state simulation with the Tier 3 2031 pumping rates; these conditions correspond to the results from Scenario 1 (Section 7.3.1). The transient simulations are conducted for 2 years and the predicted changes after 2 years are reported with respect to the results for a Base Case that corresponds to Scenario 1.

Redundancy Scenarios D-F

For the transient scenarios D, E and F, pumping from G2 is assumed to stop while all of the municipal supply wells are pumping at their reported average rates in 2008. The 2008 pumping rate for G2 is 62 L/s. The replacement 62 L/s is accommodated as follows:

- Scenario D: To the extent possible, the shutdown of G2 is replaced with additional pumping from the Pinebush Well Field, with the remainder from the Clemens Mill Well Field;
- Scenario E: To the extent possible, the shutdown of G2 is replaced with additional pumping from the Clemens Mill Well Field, with the remainder from the Pinebush Well Field; and
- Scenario F: To the extent possible, the shutdown of G2 is replaced with additional pumping from the Clemens Mill Well Field but with well G18 inactive, with the remainder from the Pinebush Well Field.

The initial conditions for the transient scenarios D, E and F are specified as the results of the steady-state simulation with the 2008 average pumping rates. The transient simulations are conducted for 2 years and the predicted changes after 2 years are reported with respect to the results of the simulation of average conditions for 2008.

7.4.1 Transient Scenario A

Scenario A is developed to assess the potential short-term effects of replacing pumping from G2 at its projected 2031 rate (80 L/s) to the extent possible with additional pumping from the Pinebush Well Field, with any remainder from the Clemens Mill Well Field. The pumping rates for Scenario A are listed on **Table 7-21.** The total municipal pumping for Scenario A is 585 L/s, the same as for the 2031 projected average conditions (Scenario 1). The changes in Scenario A pumping with respect to 2031 average conditions are summarized by well field.

Blair Rd. Well Field (pumping unchanged with respect to 2031 average rates)

Clemens Mill Well Field (total pumping increased by 16 L/s with respect to 2031 average rates)

- Start pumping at Cedarbrook well (CMPW2-06) at a rate of 16 L/s (the well was not pumping in Scenario 1)

Durbar Rd. Well Field (pumping unchanged with respect to 2031 average rates)

Elgin St. Well Field (pumping unchanged with respect to 2031 average rates)

Hespeler Well Field (pumping unchanged with respect to 2031 average rates)

Pinebush Well Field (total pumping increased by 64 L/s with respect to 2031 average rates)

- Stop pumping at P10 (P10 is pumping at 36 L/s in Scenario 1)
- Start pumping at P10B at 30 L/s
- Start pumping at P10A at 50 L/s
- Start pumping at TW1-10 at 20 L/s
- Stop pumping at P11 (P11 is pumping at 20 L/s in Scenario 1)
- Start pumping at P17 at 20 L/s

Shade's Mills Well Field (pumping unchanged with respect to 2031 average rates)

Willard Well Field (pumping unchanged with respect to 2031 average rates)

Middleton Street Well Field (total pumping reduced by 80 L/s)

- Pumping stopped at G2 for two years (G2 is pumping at 80 L/s in Scenario 1)

Table 7-21 Pumping Rates for the Cambridge East Municipal Wells – Transient Scenario A

Well ID	Well Field	Base Case [2031 avg; Scenario 1]	Scenario A
		Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	20	20
G4A		0	0
G6	Clemens Mill	10	10
G16		34	34
G17		25	25
G18		15	15
Portuguese Club (CMPW1-08)		0	0
Cedarbrook (CMPW2-06)		0	16
P6	Dunbar Rd.	0	0
G9	Elgin St.	0	0
H3	Hespeler	10	10
H3A		0	0
H4		15	15
H4A		0	0
H5		10	10
H5A		0	0
G1	Middleton St.	60	60
G1A		20	20
G2		80	0
G3		55	55
G14		25	25
Clyde Park (NDPW1-08)	North Dumfries	0	0
P9	Pinebush	15	15
P15		15	15
G5		0	0
G5A		15	15
P10		36	0
P10B		0	30
P10A		0	50
TW1-10		0	20
P11		20	0
P17		0	20
Witmer Park		0	0
CanAmera		0	0
G7	Shade's Mills	20	20
G8		10	10
G38		15	15
G39		30	30
G15	Willard	30	30
Total		585	585

The contours of simulated drawdowns in the Contact Aquifer and the Middle Gasport Formation for Scenario A are shown in **Figure 58** and in **Figure 59**. The drawdowns are calculated by subtracting the simulated groundwater levels for Scenario A at the end of the 2-year simulation period from the groundwater levels simulated for the Base Case (Scenario 1; average projected pumping rates for 2031). Positive values of the drawdown denote declines in the water level with respect to 2031 average conditions.

As shown in **Figure 58**, in the Contact Aquifer the 1 m drawdown contour extends over the Pinebush and the northern part of the Clemens Mill Well Fields. The maximum drawdown in the aquifer is centered on P10 (more precisely its replacement P10B), and is about 2 m. The maximum drawdown in the Middle Gasport Formation as shown in **Figure 59** is about 27 m, and is predicted to occur around P10A. The drawdowns around the Cedarbrook well are about 4 m.

The components of the water balance within the ZOI are listed on **Table 7-22**. As described in Section 7.3.1, the Upper Bedrock system comprises the Contact Aquifer, the Guelph Formation and the Reformatory Quarry Member of the Eramosa Formation. The Lower Bedrock system comprises the Goat Island and Gasport Formations.

Table 7-22 Calculated water balance components – Scenario A

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5038	-8702	-3664
Vertical flow across the top slice	46517	-13934	32583
Vertical flow across the bottom slice	4585	-20164	-15579
Pumping		-7260	-7260
Specified-head nodes		-6080	-6080
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	7497	-1575	5922
Vertical flow across the top slice	17319	-3289	14030
Pumping		-19952	-19952

The vertical flows across the top of the Guelph Formation after two years for the individual subwatersheds within the ZOI for Scenario A are listed on **Table 7-23**.

Table 7-23 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario A

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	4727	2293	2434	6%
Irish Creek(235)	11959	646	11313	55%
Lower Speed River(229)	5101	2498	2603	11%
Middle Grand(234)	8767	2693	6074	10%
Middle Grand(237)	1638	773	865	1%
Lower Mill Creek(236)	3128	988	2140	11%
Moffat Creek(239)	1144	383	761	3%
Fairchild Creek(241)	6176	2731	3445	2%
Middle Grand(242)	110	16	94	0%
Fairchild Creek(240)	5396	2807	2589	0.5%
Upper Mill Creek(90)	893	1021	-128	1%
Halton Region (999)	1801	466	1335	0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario A are listed on **Table 7-24** and are plotted in **Figure 60**. The only appreciable difference between the Base Case and Scenario A is the reduction in the net groundwater discharge to Irish Creek.

Table 7-24 Simulated groundwater discharges to streams – Scenario A

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario A Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19940	19936
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45639	45515
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	5626	4853
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove and Kossuth Rd.	6859	6808
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd. and Blair Rd.	1676	1676

The calculated variable components of the Puslinch Lake water balance after two years of pumping are listed on **Table 7-25**.

Table 7-25 Simulation Results for Puslinch Lake – Scenario A

Quantity	Base Case	Scenario A
Net lateral flow (negative for outflow)	-1107 m^3/d	-974 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-776 m^3/d	-869 m^3/d

7.4.2 Transient Scenario B

Scenario B is developed to assess the potential short-term effects of replacing pumping from G2 at its projected 2031 rate (80 L/s) to the extent possible with additional pumping from the Clemens Mill Well Field, with any remainder from the Pinebush Well Field. The pumping rates for Scenario B are listed on **Table 7-26.** The total municipal pumping for Scenario B is 585 L/s. This is the same as the total pumping for the Base Case (projected 2031 average rates). The changes in Scenario B pumping with respect to the Base Case are summarized by well field.

Blair Rd. Well Field (pumping unchanged with respect to 2031 average rates)

Clemens Mill Well Field (total pumping increased by 36 L/s)

- Start pumping at Cedarbrook well (CMPW2-06) at a rate of 36 L/s

Durbar Rd. Well Field (pumping unchanged with respect to 2031 average rates)

Elgin St. Well Field (pumping unchanged with respect to 2031 average rates)

Hespeler Well Field (pumping unchanged with respect to 2031 average rates)

Pinebush Well Field (total pumping increased by 44 L/s)

- Stop pumping at P10, which is pumping at 36 L/s in the 2031 scenario
- Start pumping at P10B at an average rate of 25L/s
- Start pumping at P10A at an average rate of 40 L/s
- Start pumping at TW1-10 at an average rate of 15 L/s
- Stop pumping at P11, which is pumping at 20 L/s in the 2031 scenario
- Start pumping at P17 at an average rate of 20 L/s

Shade's Mills Well Field (pumping unchanged with respect to 2031 average rates)

Willard Well Field (pumping unchanged with respect to 2031 average rates)

Middleton Street Well Field (total pumping reduced by 80 L/s)

- Pumping stopped at G2 for two years (G2 is pumping at 80 L/s in Scenario 1)

Table 7-26 Pumping Rates for the Cambridge East Municipal Wells – Scenario B

Well ID	Well Field	Base Case (2031 avg rates)	Scenario A	Scenario B
		Average Day (L/s)	Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	20	20	20
G4A		0	0	0
G6	Clemens Mill	10	10	10
G16		34	34	34
G17		25	25	25
G18		15	15	15
Portuguese Club (CMPW1-08)		0	0	0
Cedarbrook (CMPW2-06)		0	16	36
P6	Dunbar Rd.	0	0	0
G9	Elgin St.	0	0	0
H3	Hespeler	10	10	10
H3A		0	0	0
H4		15	15	15
H4A		0	0	0
H5		10	10	10
H5A		0	0	0
G1	Middleton St.	60	60	60
G1A		20	20	20
G2		80	0	0
G3		55	55	55
G14		25	25	25
Clyde Park (NDPW1-08)	North Dumfries	0	0	0
P9	Pinebush	15	15	15
P15		15	15	15
G5		0	0	0
G5A		15	15	15
P10		36	0	0
P10B		0	30	25
P10A		0	50	40
TW1-10		0	20	15
P11		20	0	0
P17		0	20	20
Witmer Park		0	0	0
CanAmera		0	0	0
G7	Shade's Mills	20	20	15
G8		10	10	30
G38		15	15	20
G39		30	30	10
G15	Willard	30	30	30
Total		585	585	585

The contours of simulated drawdowns in the Contact Aquifer for Scenario B are shown in **Figure 61**. The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 62.** In the Contact Aquifer, the 1 m drawdown contour extends over the Pinebush and northern part of the Clemens Mill Well Fields. The maximum drawdowns in the aquifer are about 2 m, centered around P10B and TW1-10. For the Middle Gasport Formation, there are two distinct drawdown cones: around P10A with a maximum drawdown of 22 m, and around the Cedarbrook well with a maximum drawdown of about 8 m.

The components of the water balances for the Upper and Lower Bedrock systems within the Zone of Influence for Scenario B are listed on **Table 7-27**.

Table 7-27 Calculated water balance components – Scenario B

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5038	-8703	-3666
Vertical flow across the top slice	46249	-13752	32497
Vertical flow across the bottom slice	4402	-20325	-15923
Pumping		-6828	-6828
Specified-head nodes		-6081	-6081
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	7539	-1554	5985
Vertical flow across the top slice	17500	-3101	14399
Pumping		-20384	-20384

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario B are listed on **Table 7-28**.

Table 7-28 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario B

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	4650	2256	2394	5%
Irish Creek(235)	11567	599	10968	50%
Lower Speed River(229)	4932	2418	2514	9%
Middle Grand(234)	8742	2702	6040	10%
Middle Grand(237)	1645	762	883	2%
Lower Mill Creek(236)	3355	953	2402	15%
Moffat Creek(239)	1210	373	837	4%
Fairchild Creek(241)	6213	2710	3503	3%
Middle Grand(242)	110	16	94	0%
Fairchild Creek(240)	5403	2800	2603	1%
Upper Mill Creek(90)	906	1015	-109	2%
Halton Region (999)	1801	466	1335	0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario B are listed on **Table 7-29** and plotted in **Figure 63**. The only appreciable difference between the predictions for the Base Case and Scenario B is the reduction in the net groundwater discharge to Irish Creek.

Table 7-29 Simulated groundwater discharges to streams – Scenario B

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario B Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19940	19932
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45639	45481
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	5626	4808
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6859	6813
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1676	1677

The calculated variable components of the Puslinch Lake water balance for Scenario B are listed on **Table 7-30**.

Table 7-30 Simulation Results for Puslinch Lake – Scenario B

Quantity	Base Case value	Scenario B value
Net lateral flow (negative for outflow)	-1107m^3/d	-967 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-776 m^3/d	-876 m^3/d

7.4.3 Transient Scenario C

Scenario C is developed to assess the potential short-term effects of replacing pumping from G2 at its projected 2031 rate (80 L/s) to the extent possible with additional pumping from the Clemens Mill Well Field but with G18 inactive, with any remainder from the Pinebush Well Field. The pumping rates for Scenario C are listed on **Table 7-31.** The total municipal pumping for Scenario C is 585 L/s. This is the same as the total pumping for the Base Case (projected 2031 average rates). The changes in Scenario C pumping with respect to the Base Case are summarized by well field.

Blair Rd. Well Field (pumping unchanged with respect to 2031 average rates)

Clemens Mill Well Field (total pumping increased by 36 L/s with respect to 2031 average rates)

- Pumping at G18 is stopped (the well is pumping at 15 L/s in the Base Case)
- Start pumping at Portuguese Club well (CMPW1-08) at a rate of 15 L/s
- Start pumping at Cedarbrook well (CMPW2-06) at a rate of 36 L/s

Durbar Rd. Well Field (pumping unchanged with respect to 2031 average rates)

Elgin St. Well Field (pumping unchanged with respect to 2031 average rates)

Hespeler Well Field (pumping unchanged with respect to 2031 average rates)

Pinebush Well Field (total pumping increased by 44 L/s)

- Stop pumping at P10 (the well is pumping at 36 L/s in the Base Case)
- Start pumping at P10B at an average rate of 25 L/s
- Start pumping at P10A at an average rate of 40 L/s
- Start pumping at TW1-10 at an average rate of 15 L/s
- Stop pumping at P11 (the well is pumping at 36 L/s in the Base Case)
- Start pumping at P17 at an average rate of 20 L/s

Shade's Mills Well Field (pumping unchanged with respect to 2031 average rates)

Willard Well Field (pumping unchanged with respect to 2031 average rates)

Middleton Street Well Field (total pumping reduced by 80 L/s)

- Pumping stopped at G2 for two years (G2 is pumping at 80 L/s in Scenario 1)

Table 7-31 Pumping Rates for the Cambridge East Municipal Wells – Scenario C

Well ID	Well Field	Base case (Tier 3 2013 rates)	Scenario A	Scenario B	Scenario C
		Average Day (L/s)	Average Day (L/s)	Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	20	20	20	20
G4A		0	0	0	0
G6	Clemens Mill	10	10	10	10
G16		34	34	34	34
G17		25	25	25	25
G18		15	15	15	0
Portuguese Club (CMPW1-08)		0	0	0	15
Cedarbrook (CMPW2-06)		0	16	36	36
P6	Dunbar Rd.	0	0	0	0
G9	Elgin St.	0	0	0	0
H3	Hespeler	10	10	10	10
H3A		0	0	0	0
H4		15	15	15	15
H4A		0	0	0	0
H5		10	10	10	10
H5A		0	0	0	0
G1	Middleton St.	60	60	60	60
G1A		20	20	20	20
G2		80	0.0	0	0
G3		55	55	55	55
G14		25	25	25	25
Clyde Park (NDPW1-08)	North Dumfries	0	0	0	0
P9	Pinebush	15	15	15	15
P15		15	15	15	15
G5		0	0	0	0
G5A		15	15	15	15
P10		36	0	0	0
P10B		0	30	25	25
P10A		0	50	40	40
TW1-10		0	20	15	15
P11		20	0	0	0
P17		0	20	20	20
Witmer Park		0	0	0	0
CanAmera		0	0	0	0
G7	Shade's Mills	20	20	15	15
G8		10	10	30	30
G38		15	15	20	20
G39		30	30	10	10
G15	Willard	30	30	30	30
Total		585	585	585	585

The contours of simulated drawdowns in the Contact Aquifer for Scenario C are shown in **Figure 64**. The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 65.** In the Contact Aquifer, the 1 m drawdown contours extends over the Pinebush Well Field and the northern portion of the Clemens Mill Well Field. For the Middle Gasport Formation, the drawdowns contours are similar to those predicted for Scenario B. There are two distinct drawdown cones: around P10A, with a maximum drawdown of 22 m and around the Cedarbrook well with a maximum drawdown of about 8 m.

The components of the water balance for the Upper and Lower Bedrock systems within the Zone of Influence for Scenario C are listed on **Table 7-32**.

Table 7-32 Calculated water balance components – Scenario C

Upper Bedrock aquifers			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5036	-8700	-3664
Vertical flow across the top slice	46202	-13774	32428
Vertical flow across the bottom slice	4094	-19939	-15845
Pumping		-6828	-6828
Specified-head nodes		-6091	-6091
Lower Bedrock aquifers			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	7619	-1538	6081
Vertical flow across the top slice	18330	-2731	15599
Pumping		-21680	-21680

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario C are listed on **Table 7-33**.

Table 7-33 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario C

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	4669	2256	2413	5.4%
Irish Creek(235)	11941	560	11381	57.7%
Lower Speed River(229)	4931	2424	2507	9.4%
Middle Grand(234)	8445	2686	5759	4.8%
Middle Grand(237)	1589	795	794	0.1%
Lower Mill Creek(236)	3231	1020	2211	12.1%
Moffat Creek(239)	1230	370	860	4.3%
Fairchild Creek(241)	6228	2701	3527	3.6%
Middle Grand(242)	110	16	94	0.0%
Fairchild Creek(240)	5406	2797	2609	0.9%
Upper Mill Creek(90)	915	1011	-96	1.8%
Halton Region (999)	1801	466	1335	0.0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario C are listed on **Table 7-34** and plotted in **Figure 66**. The only appreciable difference between the Base Case and Scenario C is the reduction in the net groundwater discharge to Irish Creek.

Table 7-34 Simulated groundwater discharges to streams – Scenario C

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario C Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd 7	19940	19930
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45639	45458
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	5626	4707
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6859	6809
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1676	1677

The calculated variable components of the Puslinch Lake water balance for Scenario C are listed on **Table 7-35**.

Table 7-35 Simulation Results for Puslinch Lake – Scenario C

Quantity	Base Case value	Scenario C value
Net lateral flow (negative for outflow)	-1107 m^3/d	-955 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-776 m^3/d	-888 m^3/d

7.4.4 Transient Scenario D

Scenario D is developed to assess the potential short-term effects of replacing pumping from G2 at its average 2008 rate (62.1 L/s) to the extent possible with additional pumping from the Pinebush Well Field, with any remainder from the Clemens Mill Well Field. The pumping rates for Scenario D are listed on **Table 7-36.** The total municipal pumping for Scenario D is 547 L/s, the same as the total pumping for 2008 average rates. The changes in pumping for Scenario D with respect to 2008 average conditions are summarized by well field.

Blair Rd. Well Field (pumping unchanged with respect to 2008 average rates)

Clemens Mill Well Field (pumping unchanged with respect to 2008 average rates)

Durbar Rd. Well Field (pumping unchanged with respect to 2008 average rates)

Elgin St. Well Field (pumping unchanged with respect to 2008 average rates)

Hespeler Well Field (pumping unchanged with respect to 2008 average rates)

Pinebush Well Field (total pumping increased by 62 L/s)

- Pumping stopped at P10 (P10 was pumped at an average rate of 34 L/s in 2008)
- Pumping started at P10B at an average rate of 30 L/s
- Pumping started at P10A at an average rate of 47 L/s
- Pumping started at TW1-10 at an average rate of 20 L/s
- Pumping at P17 reduced by 1 L/s, from 9 L/s to 8 L/s

Shade's Mills Well Field (pumping unchanged with respect to 2008 average rates)

Willard Well Field (pumping unchanged with respect to 2008 average rates)

Middleton Street Well Field (total pumping reduced by 62 L/s)

- Pumping stopped at G2 for two years (G2 was pumping at an average rate of 62 L/s in 2008)

Table 7-36 Pumping Rates for the Cambridge East Municipal Wells – Scenario D

Well ID	Well Field	Base Case (2008 average)	Scenario D
		Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	11	11
G4A		0	0
G6	Clemens Mill	16	16
G16		19	19
G17		23	23
G18		12	12
Portuguese Club (CMPW1-08)		0	0
Cedarbrook (CMPW2-06)		0	0
P6	Dunbar Rd.	10	10
G9	Elgin St.	15	15
H3	Hespeler	6	6
H3A		0	0
H4		0	0
H4A		0	0
H5		4	4
H5A		0	0
G1	Middleton St.	40	40
G1A		46	46
G2		62	0
G3		39	39
G14		37	37
Clyde Park (NDPW1-08)	North Dumfries	0	0
P9	Pinebush	17	17
P15		11	11
G5		19	19
G5A		0	0
P10		34	0
P10B		0	30
P10A		0	47
TW1-10		0	20
P11		13	13
P17		9	8
Witmer Park		0	0
CanAmera		0	0
G7	Shade's Mills	27	27
G8		14	14
G38		6	6
G39		30	30
G15	Willard	25	25
Total		547	547

The contours of simulated drawdowns in the Contact Aquifer and the Middle Gasport Formation for Scenario D are shown in **Figure 67** and in **Figure 68**. The drawdowns are calculated by subtracting the simulated groundwater levels for Scenario D at the end of the 2-year simulation period from the groundwater levels simulated for the Base Case (2008 average conditions). Positive values of the drawdown denote declines in the water level with respect to 2008 average conditions.

As shown in **Figure 67**, for Scenario D the predicted extent of the 1 m drawdown contour in the Contact Aquifer extends over the Pinebush Well Field. The maximum drawdown in the aquifer is about 2 m, in the vicinity of well P10B. As shown in **Figure 68**, the predicted drawdown cone in the Middle Gasport Formation is centered around P10 and extends across the Pinebush Well Field. The maximum drawdowns in the aquifer at P10A and TW1-10 are about 25 m and 11 m, respectively.

The components of the water balance within the ZOI for Scenario D are listed on **Table 7-37**. The Upper Bedrock system comprises the Contact Aquifer and the Guelph Formation. The Lower Bedrock system comprises the Goat Island and Gasport Formations.

Table 7-37 Calculated water balance components – Scenario D

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5005	-9044	-4039
Vertical flow across the top slice	43565	-15110	28455
Vertical flow across the bottom slice	5353	-18240	-12887
Pumping		-5186	-5186
Specified-head nodes		-6343	-6343
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	6877	-1510	5367
Vertical flow across the top slice	14440	-4099	10341
Pumping		-15709	-15709

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario D are listed on **Table 7-38**.

Table 7-38 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario D

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	3755	2714	1041	2.1%
Irish Creek(235)	10644	766	9878	61.2%
Lower Speed River(229)	4730	3038	1692	13.5%
Middle Grand(234)	8969	3089	5880	10.7%
Middle Grand(237)	1732	731	1001	0.8%
Lower Mill Creek(236)	2598	1157	1441	7.4%
Moffat Creek(239)	1208	360	848	1.5%
Fairchild Creek(241)	6103	2778	3325	1.5%
Middle Grand(242)	111	15	96	0.0%
Fairchild Creek(240)	5383	2821	2562	0.4%
Upper Mill Creek(90)	854	1041	-187	1.0%
Halton Region (999)	1800	466	1334	0.0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario D are listed on **Table 7-39** and plotted in **Figure 69**. The only appreciable difference between the Base Case and Scenario D is the reduction in the net groundwater discharge to Irish Creek.

Table 7-39 Simulated groundwater discharges to streams – Scenario D

Stations	Station description	Base Case [2008] Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario D Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19944	19940
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45662
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5703
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6984	6942
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1677	1677

The calculated variable components of the Puslinch Lake water balance for Scenario D are listed on **Table 7-40**.

Table 7-40 Simulation Results for Puslinch Lake – Scenario D

Quantity	Base Case value	Scenario D value
Net lateral flow (negative for outflow)	-1088 m^3/d	-1022 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d	-820 m^3/d

7.4.5 Transient Scenario E

Scenario E is developed to assess the potential short-term effects of replacing pumping from G2 at its average 2008 rate (62.1 L/s) to the extent possible with additional pumping from the Clemens Mill Well Field, with any remainder from the Pinebush Well Field. The pumping rates for Scenario E are listed on **Table 7-41.** The total municipal pumping for Scenario E is 547 L/s. This is the same as the total pumping for the 2008 average rates. The changes in Scenario E pumping with respect to the Base Case (2008 average conditions) are summarized by well field.

Blair Rd. Well Field (pumping unchanged with respect to 2008 average rates)

Clemens Mill Well Field (total pumping increased by 50 L/s)

- Pumping started at the Cedarbrook well (CMPW2-06) at a rate of 50 L/s

Durbar Rd. Well Field (pumping unchanged with respect to 2008 average rates)

Elgin St. Well Field (pumping unchanged with respect to 2008 average rates)

Hespeler Well Field (pumping unchanged with respect to 2008 average rates)

Pinebush Well Field (total pumping increased by 12 L/s)

- Pumping at P10 stopped (P10 was pumped at an average rate of 34 L/s in 2008)
- Pumping started at P10B at an average rate of 16 L/s
- Pumping started at P10A at an average rate of 29 L/s
- Pumping started at TW1-10 at an average rate of 10 L/s
- Pumping at P17 stopped (P17 was pumped at an average rate of 9 L/s in 2008).

Shade's Mills Well Field (pumping unchanged with respect to 2008 average rates)

Willard Well Field (pumping unchanged with respect to 2008 average rates)

Middleton Street Well Field (total pumping reduced by 62 L/s)

- Pumping stopped at G2 for two years (G2 was pumping at an average rate of 62 L/s in 2008)

Table 7-41 Pumping Rates for the Cambridge East Municipal Wells – Scenario E

Well ID	Well Field	Base Case (2008 average)	Scenario D	Scenario E
		Average Day (L/s)	Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	11	11	11
G4A		0	0	0
G6	Clemens Mill	16	16	16
G16		19	19	19
G17		23	23	23
G18		12	12	12
Portuguese Club (CMPW1-08)		0	0	0
Cedarbrook (CMPW2-06)		0	0	50
P6	Dunbar Rd.	10	10	10
G9	Elgin St.	15	15	15
H3	Hespeler	6	6	6
H3A		0	0	0
H4		0	0	0
H4A		0	0	0
H5		4	4	4
H5A		0	0	0
G1	Middleton St.	40	40	40
G1A		46	46	46
G2		62	0	0
G3		39	39	39
G14		37	37	37
Clyde Park (NDPW1-08)	North Dumfries	0	0	0
P9	Pinebush	17	17	17
P15		11	11	11
G5		19	19	19
G5A		0	0	0
P10		34	0	0
P10B		0	30	16
P10A		0	47	29
TW1-10		0	20	10
P11		13	13	13
P17		9	8	0
Witmer Park		0	0	0
CanAmera		0	0	0
G7	Shade's Mills	27	27	27
G8		14	14	14
G38		6	6	6
G39		30	30	30
G15	Willard	25	25	25
Total		547	545	547

The contours of simulated drawdowns in the Contact Aquifer for Scenario E are shown in **Figure 70**. The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 71**. In the Contact Aquifer, the 1 m drawdown contour is limited to the northern portion of the Clemens Mill Well Field. In the Middle Gasport Formation, there are two distinct drawdown cones: the cone around TW1-10 and P10A with a maximum drawdown in the aquifer of 15 m, and the cone around the Cedarbrook well with a maximum drawdown of about 10 m.

The components of the water balances for the Upper and Lower Bedrock systems within the ZOI for Scenario E are listed on **Table 7-42**.

Table 7-42 Calculated water balance components – Scenario E

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5003	-9050	-4046
Vertical flow across the top slice	42985	-14753	28232
Vertical flow across the bottom slice	4808	-18671	-13863
Pumping		-3977	-3977
Specified-head nodes		-6346	-6346
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	7003	-1472	5531
Vertical flow across the top slice	14921	-3534	11387
Pumping		-16919	-16919

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario E are listed on **Table 7-43**.

Table 7-43 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario E

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	3791	2633	1158	4.6%
Irish Creek(235)	9652	663	8989	40.7%
Lower Speed River(229)	4439	2917	1522	9.6%
Middle Grand(234)	8941	3145	5796	8.7%
Middle Grand(237)	1748	704	1044	1.8%
Lower Mill Creek(236)	3128	1051	2077	21.3%
Moffat Creek(239)	1371	334	1037	5.6%
Fairchild Creek(241)	6192	2723	3469	4.6%
Middle Grand(242)	111	15	96	0.0%
Fairchild Creek(240)	5399	2804	2595	1.1%
Upper Mill Creek(90)	886	1026	-140	2.0%
Halton Region (999)	1801	466	1335	0.0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario E are listed on **Table 7-44** and plotted in **Figure 72**. The only appreciable difference between the Base Case and Scenario E is the reduction in the net groundwater discharge to Irish Creek.

Table 7-44 Simulated groundwater discharges to streams – Scenario E

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario E Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19944	19937
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45577
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5588
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6984	6849
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1677	1677

The calculated variable components of the Puslinch Lake water balance for Scenario E are listed on **Table 7-45**.

Table 7-45 Simulation Results for Puslinch Lake – Scenario E

Quantity	Base Case value	Scenario E value
Net lateral flow (negative for outflow)	-1088 m^3/d	-1009 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d	-834 m^3/d

7.4.6 Transient Scenario F

Scenario F is developed to assess the potential short-term effects of replacing pumping from G2 at its average 2008 rate (61.2 L/s) to the extent possible with additional pumping from the Clemens Mill Well Field but with G18 inactive, with any remainder from the Pinebush Well Field. The pumping rates for Scenario F are listed on **Table 7-46.** The total municipal pumping for Scenario C is 547 L/s. This is the same as the total pumping for the 2008 average rates. The changes in Scenario F pumping with respect to the Base Case (2008 average conditions) are summarized by well field.

Blair Rd. Well Field (pumping unchanged with respect to 2008 average rates)

Clemens Mill Well Field (total pumping increased by 50 L/s)

- Pumping at G18 stopped (G18 was pumped at an average rate of 12 L/s in 2008)
- Pumping started at the Portuguese Club well (CMPW1-08) at a rate of 12 L/s
- Pumping started at the Cedarbrook well (CMPW2-06) at a rate of 50 L/s

Durbar Rd. Well Field (pumping unchanged with respect to 2008 average rates)

Elgin St. Well Field (pumping unchanged with respect to 2008 average rates)

Hespeler Well Field (pumping unchanged with respect to 2008 average rates)

Pinebush Well Field (total pumping increased by 12 L/s)

- Pumping stopped at P10 (P10 was pumped at an average rate of 34 L/s in 2008)
- Pumping started at P10B at an average rate of 16 L/s
- Pumping started at P10A at an average rate of 29 L/s
- Pumping started at TW1-10 at an average rate of 10 L/s
- Pumping stopped at P17 (P17 was pumped at an average rate of 9 L/s in 2008)

Shade's Mills Well Field (pumping unchanged with respect to 2008 average rates)

Willard Well Field (pumping unchanged with respect to 2008 average rates)

Middleton Street Well Field (total pumping reduced by 62 L/s)

- Pumping stopped at G2 for two years (G2 was pumping at an average rate of 62 L/s in 2008)

Table 7-46 Pumping Rates for the Cambridge East Municipal Wells – Scenario F

Well ID	Well Field	Base case (2008 average)	Scenario D	Scenario E	Scenario F
		Average Day (L/s)	Average Day (L/s)	Average Day (L/s)	Average Day (L/s)
G4	Blair Rd.	11	11	11	11
G4A		0	0	0	0
G6	Clemens Mill	16	16	16	16
G16		19	19	19	19
G17		23	23	23	23
G18		12	12	12	0
Portuguese Club (CMPW1-08)		0	0	0	12
Cedarbrook (CMPW2-06)		0	0	50	50
P6	Dunbar Rd.	10	10	10	10
G9	Elgin St.	15	15	15	15
H3	Hespeler	6	6	6	6
H3A		0	0	0	0
H4		0	0	0	0
H4A		0	0	0	0
H5		4	4	4	4
H5A		0	0	0	0
G1	Middleton St.	40	40	40	40
G1A		46	46	46	46
G2		62	0	0	0
G3		39	39	39	39
G14		37	37	37	37
Clyde Park (NDPW1-08)	North Dumfries	0	0	0	0
P9	Pinebush	17	17	17	17
P15		11	11	11	11
G5		19	19	19	19
G5A		0	0	0	0
P10		34	0	0	0
P10B		0	30	16	16
P10A		0	47	29	29
TW1-10		0	20	10	10
P11		13	13	13	13
P17		9	8	0	0
Witmer Park		0	0	0	0
CanAmera		0	0	0	0
G7	Shade's Mills	27	27	27	27
G8		14	14	14	14
G38		6	6	6	6
G39		30	30	30	30
G15	Willard	25	25	25	25
Total		547	547	547	547

The contours of simulated drawdowns in the Contact Aquifer for Scenario F are shown in **Figure 73**. In the Contact Aquifer, the 1 m drawdown contour is limited to area around the Portuguese Club well (CMPW1-06). The simulated drawdowns in the Middle Gasport Formation are shown in **Figure 74**. In the Middle Gasport Formation, there are two distinct drawdown cones: around P10A with a maximum drawdown in the aquifer of 15 m, and around the Cedarbrook well with a maximum drawdown of 9 m. The drawdowns around G6, G17 and G18 are less than 1 m, as a result of the cessation of pumping at G18.

The components of the water balances for the Upper Bedrock and Lower Bedrock systems within the ZOI for Scenario F are listed on **Table 7-47**.

Table 7-47 Calculated water balance components – Scenario F

Upper Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	5002	-9046	-4044
Vertical flow across the top slice	42952	-14774	28178
Vertical flow across the bottom slice	4537	-18340	-13803
Pumping		-3977	-3977
Specified-head nodes		-6354	-6354
Lower Bedrock system			
Item	Flow rate (m^3/d)		
	In	Out	Net = In + Out
Horizontal flow	7072	-1464	5608
Vertical flow across the top slice	15559	-3213	12346
Pumping		-17955	-17955

The vertical flows across the top of the Guelph Formation for the individual subwatersheds for Scenario F are listed on **Table 7-48**.

Table 7-48 Vertical groundwater flow across the top of the Guelph Formation (m^3/d) – Scenario F

Subwatershed	Vertical flow (m^3/d)			Fraction of change (%)***
	Downwards	Upwards	Net flow*	
Lower Speed River(84)**	3803	2631	1172	5%
Irish Creek(235)	9949	630	9319	49%
Lower Speed River(229)	4438	2919	1519	10%
Middle Grand(234)	8710	3160	5550	3%
Middle Grand(237)	1704	731	973	0.2%
Lower Mill Creek(236)	3039	1114	1925	18%
Moffat Creek(239)	1387	331	1056	6%
Fairchild Creek(241)	6204	2716	3488	5%
Middle Grand(242)	111	15	96	0%
Fairchild Creek(240)	5402	2802	2600	1%
Upper Mill Creek(90)	893	1023	-130	2%
Halton Region (999)	1801	466	1335	0%

*Sign convention for the net vertical flow: positive denotes net downwards flow.
**The numbers in the brackets are the subwatershed numbers.
***The fraction of change (%) is calculated as the change in the net flow in each subwatershed with respect to the Base Case divided by the total change in the net flow for all the subwatersheds.

The calculated groundwater discharges to streams for Scenario F are listed on **Table 7-49** and plotted in Figure 75. The only appreciable difference between the Base Case and Scenario F is the reduction in the net groundwater discharge to Irish Creek.

Table 7-49 Simulated groundwater discharges to streams – Scenario F

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario F Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd 7	19945	19936
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45558
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd	6288	5506
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove Rd and Kossuth Rd.	6984	6947
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd and Blair Rd	1677	1677

The calculated variable components of the Puslinch Lake water balance for Scenario F are listed on **Table 7-50**.

Table 7-50 Simulation Results for Puslinch Lake – Scenario F

Quantity	Base Case value	Scenario F value
Net lateral flow (negative for outflow)	-1088 m^3/d	-998 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-747 m^3/d	-845 m^3/d

8 PERSPECTIVE ON THE RESULTS OF THE SCENARIO SIMULATIONS

The following sections include additional discussion to provide an appropriate context for assessing the predictions of long-term changes in groundwater levels and streamflows.

The Cambridge East groundwater modelling has been conducted using approaches deliberately designed to limit the uncertainties in the model predictions. These approaches include the following:

- Specification of recharge rates independently of the calibration of the groundwater model. The recharge rates have been specified on the basis of analyses conducted with the streamflow generation program GAWSER. The GAWSER analyses are constrained by available streamflow data;
- The calibration of the model has involved matching distinct sets of targets representative of steady state conditions and transient responses observed during extended pumping tests with intensive monitoring;
- The model has been calibrated to match interpreted rates of groundwater discharge to streams and to the available interpretations of the component of the water balance of Puslinch Lake; and
- The predictive capabilities of the model have been tested by applying the calibrated model to simulate the long term effects of an increase in pumping from the Clemens Mill Well Field and the effects of pumping at a new well. The results of these tests have confirmed that the model is capable of yielding results that are consistent with observations, without adjustment of model parameters.

The Cambridge East model has been calibrated and tested extensively. However, when assessing the results of predictive simulations it is important to bear in mind always that the model is a deliberately simplified representation of a complex natural system. The simplifications in the analyses introduce uncertainties in the analyses. Some of these uncertainties are associated with the impossibility of characterizing in detail all aspects of the subsurface in the Cambridge area. Even in portions of the study area where the characterization is relatively complete, it is not feasible to capture all the details of the surface water and groundwater systems in the numerical model. Other uncertainties arise because not all of available monitoring data can be interpreted definitively. The data used to calibrate the model come from a variety of sources of differing reliability. With these uncertainties in mind, it is more appropriate to consider the predicted changes as suggestions of what changes *might* occur, rather than as definitive statements regarding what *will* occur. In other words, the results of the modelling are hypotheses that will be tested through the ongoing collection of hydrogeologic data.

In the remainder of this section, additional discussion is provided on the predictions of long-term changes in groundwater levels, groundwater discharge to streams, and changes in the components of the water balance and the average level of Puslinch Lake.

8.1 Predictions of long-term changes in groundwater levels

The focus here is on the long-term changes in water levels predicted with the steady-state analyses. The transient scenarios have been developed to examine short-term security of supply. Predicted long-term changes in groundwater levels are presented in terms of drawdowns in the uppermost regional groundwater flow system, the Contact Aquifer, and the primary lower bedrock aquifer, the Middle Gasport Formation, are presented for each scenario. Drawdowns for each scenario are calculated as the differences between the simulated hydraulic heads for each scenario and the Base Case. The 1 m drawdown contour is interpreted as the outer limit of declines in groundwater levels that can be distinguished from natural seasonal fluctuations in groundwater levels. This criterion was adopted for the delineation of the WHPA Q1 for the Region of Waterloo Tier Three analyses and is based on an assessment of water level records from observation wells (Matrix Solutions and SSP&A, 2012).

Maximum drawdowns in the Contact Aquifer are predicted for Scenario 2, about 2 m focused around well P10B. The Contact Aquifer drawdowns are smallest for Scenario 1, about 1 m limited to the immediate vicinity of the municipal production wells. Larger drawdowns in the Middle Gasport Formation are predicted for the scenarios, with maximum drawdowns ranging from about 4 m to 17 m.

The predicted long-term changes in groundwater levels in the shallow units are less certain, reflecting the complexity of the shallow subsurface and the relatively small magnitudes of the changes observed in the events used to calibrate the groundwater model. The overburden units are discontinuous over the model area and groundwater levels in the Contact Aquifer are considered to be most representative of the shallow regional water table. Wetlands are present over large portions of Cambridge East and relatively little is known about the hydrogeology of these features. The wetlands are not currently represented explicitly in the groundwater model; rather, these features are incorporated indirectly through the specification of recharge rates and the assignment of the hydraulic properties of the overburden materials. The simplified representation of the wetlands does not account for their role as reservoirs that attenuate the effects of pumping.

The transient calibration involved matching observed changes in water levels in pumping wells and monitoring wells, with the primary focus directed towards matching the clearest responses to pumping, that is, the largest observed drawdowns. The propagation of the effects of deep pumping to the shallow groundwater system is attenuated by intervening aquifers in the overburden and bedrock. As the magnitudes of the observed changes in water levels in the shallow groundwater system are relatively small, predictions of potential changes in shallow groundwater levels are necessarily less certain. Considered from this perspective, there is likely to be more uncertainty in the predictions of drawdowns in the Contact Aquifer than the Middle Gasport Formation.

Observations during the 28-day pumping test highlight the differences in the responses between the shallow and deep groundwater systems. During the 28-day pumping test, changes in hydraulic heads were monitored in three production wells and 74 observation wells. Clear responses to pumping were observed in 80% of the bedrock observation wells (29 of 36 wells), with drawdowns at the end of pumping ranging from 0.2 m to 9.0 m. In contrast, clear responses to pumping were observed in about 50% of the overburden wells (21 of 38 wells), with final drawdowns ranging from 0.2 m to 1.3 m.

Drawdowns at observation well CMOW1-06 during the 28-day pumping test are shown in **Figure 76**. Well CMOW1-06 is located 330 m from pumping well CMPW1-06. CMOW1A-06 is open across the Lower portion of the Gasport Formation and CMOW1D-06 is open across the Contact Aquifer. The final drawdown in the "A" interval, the dominant contributing interval for the production wells, is about 5 m. In contrast, the drawdown in the shallower "D" interval is about 1 m. The trends in the drawdowns are as important as their final magnitudes. Drawdowns stabilize relatively quickly in the deeper bedrock during each increment of pumping. With the much smaller magnitudes of the shallow drawdown it is not clear whether water levels stabilize in the overburden by the end of the test. These observations have important implications with respect to the relative certainty that can be attributed to the predictions of water levels in the deep bedrock and shallow groundwater flow systems.

Additional uncertainty in the prediction of long-term changes in shallow groundwater levels arises because of the highly variable propagation of the effects of pumping from deeper to shallow strata. Long-term hydrographs at two locations, OW4-92 and OW5-94, are plotted in **Figure 77** and **Figure 78**, respectively. The "A" monitoring interval at OW4-92 is open across the Guelph Formation, while monitoring intervals "B" and "C" are screened in the overburden, with OW4B-92 open across Maryhill Till equivalent sediments (ATB3, ATC1 and ATC2) and OW4C-92 is open across the Port Stanley Till (ATB1). As shown in Figure 77, nearly identical responses are observed in the bedrock (A) and overburden (B and C) monitoring intervals. The "B" monitoring interval of OW5-94 is open across the Middle Gasport Formation and the "F" monitoring interval is screened across the Wentworth Till (ATA2). The responses in the deep (B) and shallow (F) monitoring intervals at OW5-94 appear to track each other closely; however, the absolute water levels in the two intervals differ consistently by between 1 m and 3 m. The differences between the hydrographs shown in Figures 77 and 78 reflect the subtle effects of small scale variations in the vertical properties of the bedrock and the discontinuous aquitards in the overburden.

The scenarios for the Cambridge East analyses have been developed to predict long-term changes in water levels arising from changes in the time-averaged pumping rates of municipal production wells. The data shown in Figures 77 and 78 provide important insight into the challenges inherent in checking these predictions. The data from the OW4-92 and OW5-95 well clusters have been used to infer the long-term effects of the increase in pumping at the Clemens Mill Well Field in the mid-1990s, as described in Section 4.2 Although it is possible to infer long-term differences between the periods of 1995-1997 and 1999-2008, significant short-term fluctuations are evident in both sets of responses. The short-term fluctuations are similar in magnitude to the inferred magnitudes of the long-term drawdowns. In the absence of long-term continuous monitoring, the short-term fluctuations in groundwater levels may obscure the effects of changes in pumping.

8.2 Predictions of long-term changes of groundwater discharge to streams

For all of the scenarios, the only changes in the groundwater discharges to streams that are likely to be detectible in long-term streamflow records are predicted to occur in the Irish Creek subwatershed.

Predicted reductions in the net groundwater discharge to Irish Creek vary between 9% and 16%.

- The simulated average groundwater discharge to Irish Creek for 2008 pumping rates is 6290 m^3/d. The predicted average groundwater discharges for the long-term simulations of 2031 conditions, Scenarios 1, 2 and 3, are 5630, 5430 and 5420 m^3/d, respectively. These correspond to reductions of 10% to 14% with respect to 2008 conditions.

- The simulated average groundwater discharge to Irish Creek for 2031 pumping rates is 5626 m^3/d. For the transient simulations of the changes in 2031 pumping rates, Scenarios A, B and C, are 4858, 4808 and 4707 m^3/d, respectively. These correspond to reductions of between 14% and 16% with respect to average conditions predicted for 2031 pumping rates.

- For the transient simulations of the changes in 2008 conditions, Scenarios D, E and F, are 5703, 5588 and 5506 m^3/d, respectively. These correspond to reductions between 9% and 12% with respect to 2008 conditions for average pumping rates.

The predicted reductions for Irish Creek are on the order of 10%, which has been suggested in Ontario Source Water Protection Water Budget studies as the level of concern for predicted reductions in streamflow (AquaResource, 2011; p. 58). The 10% criterion was adopted based on the assumption that reductions in streamflow of less than 10% cannot be inferred reliably from streamflow measurements. It is important to note that the 10% criterion was developed for coldwater creeks and is not strictly applicable in the present context, as Irish Creek is a warm water feature.

The predicted changes in the groundwater discharge to Irish Creek should be assigned a relatively high uncertainty. Limited data are currently available to characterize conditions in Irish Creek. Data on the properties of the streambed of Irish Creek and continuous streamflow data are not available. The only data available to constrain the analyses are three spot flow measurements made in 2008, at the point marked IC_20 in **Figure 9c**. The location of the spot flow measurements is close to the outlet of Irish Creek at the Speed River. Spot flow measurements are typically taken under dry weather conditions, with the implicit assumption that the measurements of streamflow are representative of baseflow. The baseflow is in turn assumed to be representative of the accumulated groundwater discharge to Irish Creek. As shown in **Figure 57**, the groundwater discharge to Irish Creek estimated with the calibrated groundwater flow model falls within the range of the observations. However, as shown in **Figure 79** the range in the observations is relatively wide, from 5,350 m^3/d to 9,250 m^3/d. It is not clear whether the spot values are sufficient to be representative of long-term average conditions.

The predicted changes in groundwater discharge provide important insights into the likely effects of pumping. Over the long-term, the additional pumping in Cambridge East will be supplied by a combination of induced stream leakage and the interception of water that would otherwise discharge to streams. The simulation results suggest that the greatest potential for inducing stream leakage is likely to occur for those scenarios that involve increased pumping rates from shallow wells closest to streams. In essence, larger potential impacts are predicted for increased pumping from those wells closest to Irish Creek that are open across the Contact Aquifer and Upper Bedrock Aquifers. Shallow pumping is likely to have a more concentrated effect on local streams, while the effects of pumping from the Lower Bedrock Aquifer are likely to be more diffuse.

Relative reductions in the groundwater discharge to Mill Creek are predicted to range from 0.2% and 0.5%, much smaller than for Irish Creek. In contrast, more extensive declines in groundwater levels beneath Mill Creek are predicted for all of the scenarios. This apparent paradox is resolved by considering two factors. First, the absolute magnitude of the flow in Mill Creek is significantly larger than in Irish Creek. The Base Case analysis yields groundwater discharges to Irish Creek and Mill Creek (between gauges 3AQ131 and 2GAC19) of about 6,300 m^3/d and 26,000 m^3/d, respectively. Therefore, reductions of similar absolute magnitude are reported as larger *relative* changes for Irish Creek. Second, additional groundwater withdrawals are predicted to be satisfied not by reductions in groundwater discharge to Mill Creek between 3AQ131 and 2GAC19, but by reductions along the reach downstream of 2GAC19.

8.3 Predictions of potential changes at Puslinch Lake

For each scenario, the groundwater model has been used to predict changes in the components of the water balance for Puslinch Lake.

The water balance for Puslinch Lake presented in Harden (1999) suggests that there is net leakage from Puslinch Lake to the groundwater system. The results of the calibrated model are consistent with the Harden analysis. For the simulated scenarios, the predicted rate of leakage increases by between 4% and 19%. The magnitudes of the predicted changes in the leakage from the lake should be assigned a relatively high uncertainty. No data were available to constrain the Harden (1999) analysis; therefore, the reported rate of leakage must be considered as a rough approximation. It is also important to note that conditions around the lake itself are not well understood. A portion of the lake is adjacent to a bog; a hydrogeologist with the Ontario Ministry of the Environment has suggested “the hydraulic connection between the bog and groundwater flow system is still not well understood.” (P. Grande, Ontario MOE; memorandum dated October 7, 2010). In response, Harden (2010) noted that the “bog” may in fact be perched above the groundwater system. This implies that the shallow subsurface may not be connected directly to the groundwater system that is the focus of the present analyses.

The groundwater model also calculates changes in groundwater levels in the aquifer that underlies Puslinch Lake. The results of the analysis of the 28-day pumping test suggest that the calibrated groundwater model may be capable of providing reliable estimates of potential changes in groundwater levels. The results for monitoring well C2 are reproduced in **Figure 80**. C2 is an overburden well located close to the southern bank of the lake. As shown in the figure, a clear drawdown of about 0.4 m is observed during the 28-day pumping test, and it has been possible to match the observations closely with the groundwater model.

To provide context for assessing the significance of the potential changes in groundwater levels in the aquifer underlying Puslinch Lake, the water levels available in the lake itself reported in the Region of Waterloo WRAS+ database are plotted in **Figure 81**. The reference level adopted for studies of the lake conducted by Harden (2010) is also indicated in **Figure 81**. The reference level is consistent with the value indicated on the 1:50,000 topographic map of the Cambridge area (Energy Mines and Resources Canada, 1984; Map 40 P/8, Edition 7). The data from the gauge at the north shore of the lake, Object Number 1000935, suggest that levels in Puslinch Lake vary seasonally, with a range of about 0.3 m. The predicted changes in the groundwater levels in the aquifer are similar in magnitude to the range of the natural fluctuations in the lake level. The data plotted in **Figure 81** also indicate a long-term trend in the lake level between 2000 and 2007, with the average water level increasing from about 303.9 m ASL to 304.3 m ASL. This suggests that there may be long-term changes in climate that are not captured with the steady-state groundwater model. The data plotted in the figure also appear to suggest that the levels at the south side of the lake (Object Number 9202660) may be higher than at the north side (Object Number 1000935).

9 REFERENCES

Anderson, M.P., and W.W. Woessner, 1992. **Applied Groundwater Modeling**, Academic Press, Inc., San Diego, California.

Anderson, M.P., R.J. Hunt, J.T. Krohelski, and K. Chung, 2002. Using high hydraulic conductivity nodes to simulate seepage lakes. *Ground Water*, 40(2), pp. 117-122.

American Society for Testing and Materials (ASTM), 1993. Standard Guide for Comparing Ground-Water Flow Model Simulations to Site-Specific Information, ASTM Standard Guide D5490-93, American Society for Testing and Materials, Philadelphia, Pennsylvania.

AquaResource Inc., 2009a. Grand River Conservation Authority Integrated Water Budget Report. Unpublished report submitted to the Grand River Conservation Authority. 194p.

AquaResource Inc., 2009b. Tier Three Water Budget GAWSER Modeling Report. Unpublished report submitted to Region of Waterloo, 2009.

AquaResource Inc., 2010a. Technical Memorandum: Numerical Model Surface Updates, Region of Waterloo. Submitted to the Regional Municipality of Waterloo.

AquaResource Inc., 2010b. Technical Memorandum: Permit to Take Water Review, Region of Waterloo. Submitted to the Regional Municipality of Waterloo.

AquaResource Inc., 2011. Water Budget & Water Quantity Risk Assessment Guide, Drinking Water Source Protection Program. Prepared for the Ontario Ministry of Natural Resources and the Ontario Ministry of the Environment, October 2011.

Bajc, A.F. and J.A. Hunter, 2006. Results of 2003-2004 overburden drilling programs in the region of Waterloo, southwestern Ontario; Ontario Geological Survey, Miscellaneous Release-Data 205.

Bajc, A.F. and J. Shirota, 2007. Three-dimensional mapping surficial deposits in the Regional Municipality of Waterloo, southwestern Ontario, Ontario Geological Survey, Groundwater Resources Study 3, 42p.

Barnett, P.J., 1978. Quaternary geology of the Simcoe area, southern Ontario. Ontario Division of Mines, Geological Report 162, 174p.

Barnett, P.J., 1979. Glacial Lake Whittlesey: The probable ice frontal position in the eastern end of the Erie Basin. *Canadian Journal of Earth Sciences*, 16, pp. 568-574.

Barnett, P.J., 1992. Quaternary Geology of Ontario. in *Geology of Ontario*, P. C. Thurston, H. R. Williams, R. H. Sutcliffe, G. M. Stott (editors), Ontario Geological Survey, Toronto, pp. 1011-1090.

Barnett, P.J., 1998. Quaternary Geology, Long Point – Port Burwell area; Ontario Geological Survey, Report 298, 143p.

Beak Consultants Limited, Raven Beck Environmental Limited, Waterloo Hydrogeologic Software, 1995. Delineation of the Capture Zone for the Middleton Street Well Field, Cambridge, Ontario, 2 volumes, prepared for the Regional Municipality of Waterloo, October 1995.

Blackport Hydrogeology Inc., 2009. Tier Three Water Budget and Local Area Risk Assessment: Waterloo North, William Street and Lancaster Well Fields Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Boonstra, J., 1989. **SATEM: Selected Aquifer Test Evaluation Methods**, ILRI Publication 48, International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands.

Boulton, G.S., 1972. Modern arctic glaciers as depositional models for former ice sheets. *Journal of the Geological Society of London*, 128, pp. 361-393.

Brunton, F.R., J.E.P. Dodge, and J. Shirota, 2005. Karst in southern Ontario; in *Summary of Field Work and Other Activities, 2005*, Ontario Geological Survey, Open File Report 6172, pp. 27-1 to 27-7.

Brunton F.R., D. Belanger, S. DiBiase, G. Yungwirth, and G. Boonstra, 2007. Caprock carbonate stratigraphy and bedrock aquifer character of the Niagara Escarpment - City of Guelph region, southern Ontario; in *Diamond Jubilee Canadian Geotechnical Conference and the 8th Joint CGS/IAH-CNC Groundwater Conference*, Ottawa, Ontario, Oct. 21-24th, 2007, pp. 371-377.

Brunton, F.R., 2008. Preliminary revisions to the early Silurian stratigraphy of Niagara Escarpment: Integration of sequence stratigraphy, sedimentology and hydrogeology to delineate hydrogeologic units, in *Summary of Field Work and Other Activities 2008*, Ontario Geological Survey, Open File Report 6226, pp. 31-1 to 31-18.

Brunton, F.R., 2009. Update of revisions to the early Silurian stratigraphy of the Niagara Escarpment: Integration of sequence stratigraphy, sedimentology and hydrogeology to delineate hydrogeologic units, in *Summary of Field Work and Other Activities 2009*, Ontario Geological Survey, Open File Report 6240, pp. 25-1 to 25-20.

Brunton, F.R., and C. Brintnell, 2011. Final update of early Silurian stratigraphy of the Niagara Escarpment and correlation with subsurface units across southwestern Ontario and the Great Lakes Basin, in *Summary of Field Work and Other Activities 2011*, Ontario Geological Survey, Open File Report 6270, pp. 30-1 to 30-11.

Burns, L.S., 2005. Fracture Network Characteristics and Velocities of Groundwater, Heat and Contaminants in a Dolostone Aquifer in Cambridge, Ontario, Master of Science thesis, Department of Earth Sciences, University of Waterloo.

Cooper, H.H., Jr., and C.E. Jacob, 1946. A generalized graphical method for evaluating formation constants and summarizing well-field history, *Trans. American Geophysical Union*, 27(4), pp. 526-534.

Denhoed, S., 2010. Personal Communication [teleconference]. May 28, 2010.

Diersch, H.-J.G., 2014. **FEFLOW: Finite Element Modeling of Flow, Mass and Heat Transport in Porous and Fractured Media**, Springer-Verlag, Berlin, Germany.

Doherty, J., 2005. PEST: Model-Independent Parameter Estimation, User Manual: 5th Edition, Watermark Numerical Computing.

Dreimanis, A., 1966. Lake Arkona-Whittlesey and Post-Warren radiocarbon dates from "Ridgetown Island" in southwestern Ontario. *The Ohio Journal of Science*, 66(6), pp. 582-586.

Duke Engineering & Services, 1998a. Cambridge Groundwater Modeling – Model Preparation and Code Documentation, prepared for the Regional Municipality of Waterloo, Water Services Division, July 7, 1998.

Duke Engineering & Services, 1998b. Numerical and Analytical Modeling to Guide Selection of Bedrock Transport Porosity for Use in Cambridge Capture Zone Modeling, prepared for the Regional Municipality of Waterloo, Water Services Division, September 24, 1998.

Duke Engineering & Services, 1998c. Cambridge Capture Zone Modeling Project – Final Report, Report prepared for the Regional Municipality of Waterloo, Water Services Division, November 27, 1998.

Fulton, R.J., P.F. Karrow, P. La Salle, and D.R. Grant, 1986. Summary of Quaternary stratigraphy and history, Eastern Canada, in Quaternary glaciations in the Northern Hemisphere, V. Sibrava, D.Q. Bowen, and G.M. Richmond (eds.), *Quaternary Science Reviews*, 5, pp. 211-228.

Golder Associates Ltd., 1991. Cambridge Groundwater Study, prepared for the Regional Municipality of Waterloo, April 1991.

Golder Associates, 2006. Guelph-Puslinch Groundwater Protection Study. Prepared for the Grand River Conservation Authority.

Golder Associates, 2007. Draft Interim Report on IUS Groundwater Supply Optimization and Expansion Project: Task B3 – Detailed Hydrogeological Investigation of the Cambridge East Area.

Golder Associates, 2008a. Report on IUS Groundwater Supply Optimization and Expansion Project: Task C2 – Assessment of Wells K41/42A. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2008b. Report on Hydrogeologic Data Compilation and Interpretation for the Waterloo, Kitchener and Cambridge Landfills. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2009a. Tier Three Water Budget and Local Area Risk Assessment: Mannheim Well Fields Characterization. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2009b. Report on IUS Groundwater Supply Optimization and Expansion Project: Task C3 – Assessment of Well P16 and the Fountain Street Aquifer. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2009c. Report on IUS Groundwater Supply Optimization and Expansion Project: Tasks C4 - Hydrogeological Assessment Well W14 Area. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2009d. Report on IUS Groundwater Supply Optimization and Expansion Project: Tasks C1, D1 and D4 - Hydrogeological Assessment of W5 and Erbsville Wells and the Waterloo North Aquifer System. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2011a. City of Guelph Southwest Quadrant Water Supply Class Environmental Assessment: draft Hydrogeological and Natural Environment Report. Report submitted to the Corporation of the City of Guelph, July 2011.

Golder Associates, 2011b. Tier Three Water Budget and Local Area Risk Assessment: Cambridge East Well Field Characterization. Report submitted to the Regional Municipality of Waterloo.

Golder Associates, 2011c. Tier Three Water Budget and Local Area Risk Assessment: Fountain Street Well Field Characterization. Report submitted to the Regional Municipality of Waterloo.

Grande, P., 2010. Memorandum: Review of letter report dated August 10, 2010 for the Puslinch Lake Dredgate Impoundment C of A Amendment prepared by Harden Environmental Services Ltd., Ontario Ministry of the Environment, October 7, 2010.

Harden Environmental Services Ltd., 1999. Puslinch Lake Characterization and Rehabilitation Options Report. (draft). Harden Environmental Services Ltd., Moffat, Ontario. 30p.

Harden Environmental Services Ltd., 2010. Response to MOE memorandum dated Oct 7th, 2010 for Puslinch Lake Dredgate Impoundment C of A Amendment, File WE PU 09 09, IDS# 8165-90DHP9, submitted to the Ontario Ministry of the Environment, October 25, 2010.

Hill, M.C., 1998. Methods and Guidelines for Effective Model Calibration, U.S. Geological Survey, Water-Resources Investigation Report 98-4005.

Hill, M.C., and C.R. Tiedeman, 2002. Weighting observations in the context of calibrating groundwater models, in *Calibration and Reliability in Groundwater Modeling: A few steps closer to reality*, ModelCARE'2002, Prague, Czech Republic, June 17-20, 2002, International Association of Hydrologic Sciences, Publication No. 277, pp. 196-203.

Intera Consultants Ltd., 1997. Conceptual hydrogeologic model, Cambridge groundwater modeling project, letter report submitted to Mr. James Robinson, Regional Municipality of Waterloo, December 11, 1997.

Intera Engineering Ltd., 2002. Road salt dilution zones, Middleton St. Well Field, Cambridge, letter report submitted to Mr. James Robinson, Regional Municipality of Waterloo, April 2, 2002.

International Water Supply, Ltd., 1974. Groundwater Resources Study, Cambridge Area, prepared for the Regional Municipality of Waterloo, June 1974.

International Water Supply, Ltd., 1993. Middleton St. Well Field Well and Aquifer Performance Assessment, prepared for the Regional Municipality of Waterloo, June 1993.

Johnson, M.D., D.K. Armstrong, B.V. Sanford, P.G. Telford, and M.A. Rutka, 1992. Paleozoic and Mesozoic Geology of Ontario; in *Geology of Ontario*, Ontario Geological Survey, Special Volume 4, Part 2, pp. 907-1008.

Karrow, P.F., 1984. Quaternary stratigraphy and history, Great Lakes – St. Lawrence region, in *Quaternary Stratigraphy of Canada – A Canadian Contribution to IGCP Project 24*, R. J. Fulton (ed.),. Geological Survey of Canada, Paper 84-100, pp. 137-153.

Karrow, P.F., 1987. Quaternary Geology of the Hamilton-Cambridge Area, Southern Ontario, Ontario Geological Survey Report 255, Ministry of Northern Development and Mines, Mines and Minerals Division.

Karrow, P.F., 1988. Catfish Creek Till: An important glacial deposit in southwestern Ontario, in *Proceedings of the 41st Canadian Geotechnical Conference*. Kitchener, Ontario, Preprints, pp. 186-192.

King, E.L., H. Haflidason, H.P. Sejrup, and R. Løvlie, 1998. Glacigenic debris flows on the North Sea Trough Mouth Fan during ice stream maxima. *Marine Geology*, 152(1-3), pp. 217-246.

Krzyszkowski, D., and P.F. Karrow, 2001. Wisconsinan inter-lobal stratigraphy in three quarries near Woodstock, Ontario. *Géographie physique et Quaternaire*, 55, pp. 3-22.

Lohman, S.W., 1972: Ground-Water Hydraulics, U.S. Geological Survey, Professional Paper 708.

Matrix Solutions Inc. and S.S. Papadopulos & Associates Inc., 2012. Region of Waterloo Tier Three Water Budget and Local Area Risk Assessment: Model Calibration and Water Budget Report, prepared for the Regional Municipality of Waterloo, draft dated November 2012.

Lotowater Ltd., 1997. Study of the Hydrogeology of the Cambridge Area, prepared for the Regional Municipality of Waterloo, May 1997.

Neuman, S.P., and P.A. Witherspoon, 1969. Theory of flow in a confined two aquifer system, *Water Resources Research*, 5(4), pp. 803-816.

Ontario Geological Survey, 2010. Surficial geology of southern Ontario; Miscellaneous Release Data 128; Scale of 1:50,000. Accessible at: http://www.geologyontario.mndm.gov.on.ca/mndmaccess/mndm_dir.asp?type=pub&id=MRD128-REV

Panday, S., and P.S. Huyakorn, 2008. MODFLOW SURFACT: A state-of-the-art use of vadose zone flow and transport equations and numerical techniques for environmental evaluations, *Vadose Zone Journal*, 7(2), pp. 610-631.

Rajakaruna, N., 1994. The Waterloo Moraine Project Phase 1: Subsurface stratigraphy of Western Kitchener-Waterloo, Ontario. Department of Earth Sciences. Waterloo, University of Waterloo. Master of Science. 41p.

R.J. Burnside & Associates Ltd., 2008. 2007 Biennial Groundwater Monitoring Report, prepared for the Regional Municipality of Waterloo, July 2008.

R.J. Burnside & Associates Ltd., 2009. Groundwater Monitoring Network Assessment, prepared for the Regional Municipality of Waterloo, April 2009.

Russell, H.A.J, D.R. Sharpe, and A.F. Bajc., 2007. Sedimentary signatures of the Waterloo Moraine, Ontario, Canada. In: *Glacial Sedimentary Processes and Products*, M.J. Hambrey, P. Christoffersen, N.F. Glasser, B. Hubbard (eds.), International Association of Sedimentologists Special Publication 39, Blackwell Publishing, pp. 85-108.

Sharpe, D.R., 1978. A revised correlation of the Port Huron moraine system in southern Ontario. Geological Society of America, Abstract with Programs, 10: 284.

Spitz, K., and J. Moreno, 1996. **A Practical Guide to Groundwater and Solute Transport Modeling**, John Wiley & Sons, Inc., New York, New York.

Stantec Consulting Ltd., 2007. Middleton Water Supply Study: Hydrogeologic Assessment, prepared for the Regional Municipality of Waterloo, Transportation and Environmental Services Department, Water Resources Protection Administration, March 2007.

Stantec Consulting Ltd., 2009a. Tier Three Water Budget and Water Quantity Risk Assessment: Erb Street Well Field Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Stantec Consulting Ltd., 2009b. Tier Three Water Budget and Local Area Risk Assessment: Greenbrook Well Field Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Stantec Consulting Ltd., 2009c. Tier Three Water Budget and Local Area Risk Assessment: Parkway and Strasburg Well Fields Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Stantec Consulting Ltd., 2009d. Tier Three Water Budget and Water Quantity Risk Assessment: Strange Street Well Field Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Stantec Consulting Ltd., 2011a. Tier Three Water Budget and Local Area Risk Assessment: Cambridge Northwest Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Stantec Consulting Ltd., 2011b. Tier Three Water Budget and Local Area Risk Assessment: Cambridge Southwest Characterization Study. Report submitted to the Regional Municipality of Waterloo.

Stephens, D.B., K.-C. Hsu, M.A. Prieksat, M.D. Ankeny, N. Blandford, T.L. Roth, J.A. Kelsey, J.R. Whitworth, 1998. A comparison of estimated and calculated effective porosity, *Hydrogeology Journal*, 6, pp. 156-165.

Sverre LaBerg, J. and T. O. Vorren, 2008. Flow behavior of submarine glacigenic debris flows on the Bear Island Trough Mouth Fan, western Barents Sea. *Sedimentology*, 47(6), pp. 1105-1117.

Theis, C.V., 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using ground-water storage, *Trans. American Geophysical Union*, 16th Annual Meeting, Part 2, pp. 519-524.

Waterloo Hydrogeologic Inc. (WHI), 2008. Regional Municipality of Waterloo Groundwater Model Calibration Results. Unpublished Technical Memorandum submitted to the Regional Municipality of Waterloo.

Yager, R.M., 1996. Simulated Three-Dimensional Ground-Water Flow in the Lockport Group, a Fractured-Dolomite Aquifer Near Niagara Falls, United States Geological Survey, Water Supply Paper 2487.

FIGURES

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- Cross Section Locations
- Expressway / Highway
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Scale 1:125,000

2.5 0 2.5 5

Kilometres

REFERENCES:
Base Data - Region of Waterloo, 2008; GRCA, 2008
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Projection: UTM Zone 17N, NAD 83
Map Version: 1; Map Date: 10-06-2014; Created By: ccurry

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Figure 1
Study Area, Model Domain and Cross-Section Locations

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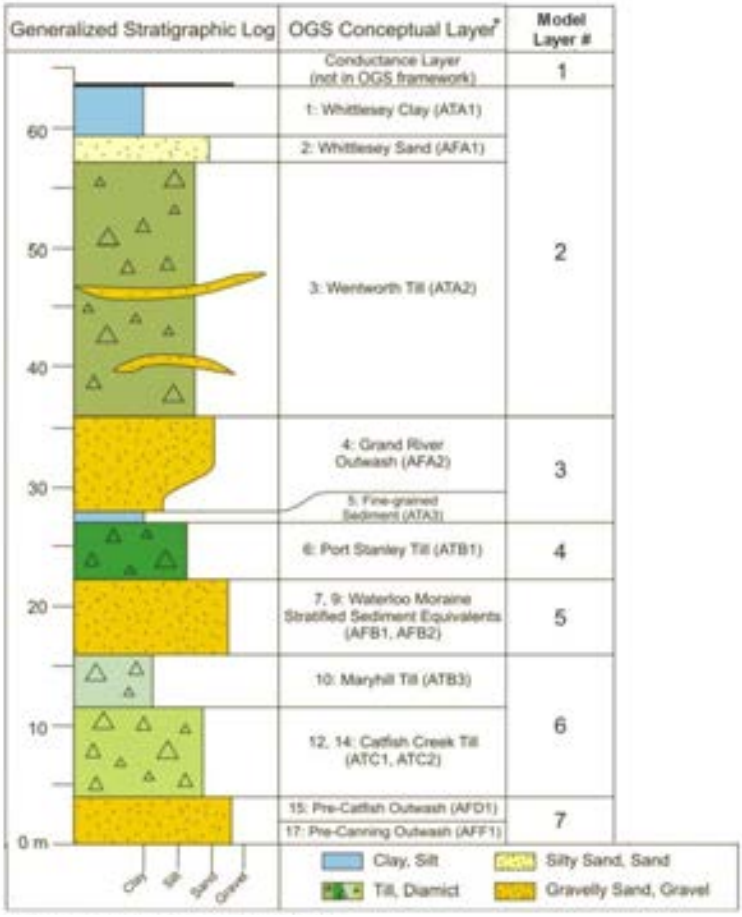
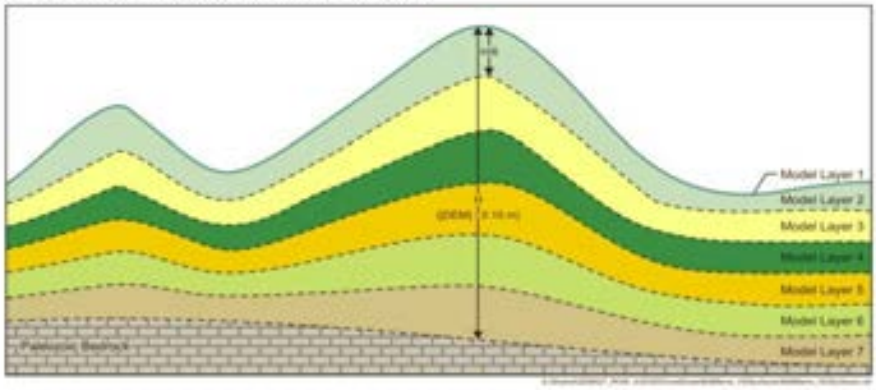


A) Overburden model layers within the Region of Waterloo



B) Overburden model layers outside the Region of Waterloo

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA : GROUNDWATER MODELLING	**Figure 2** Overburden Model Layers	Matrix Solutions Inc.

Bedrock Stratigraphy

Formation	Member	
Guelph	Hanlon	Upper bedrock units
	Wellington	
Eramosa	Stone Road	
	Reformatory Quarry	
	Vinemount	
Goat Island	Ancaster	Lower bedrock units
	Niagara Falls	
Gasport	Gothic Hill	
Rochester		
Irondequoit		
Rockway		
Merritton		
Cabot Head		

Cambridge East Model Layers

Contact Aquifer — 8

Layer 9

10

11

12

13

14

15

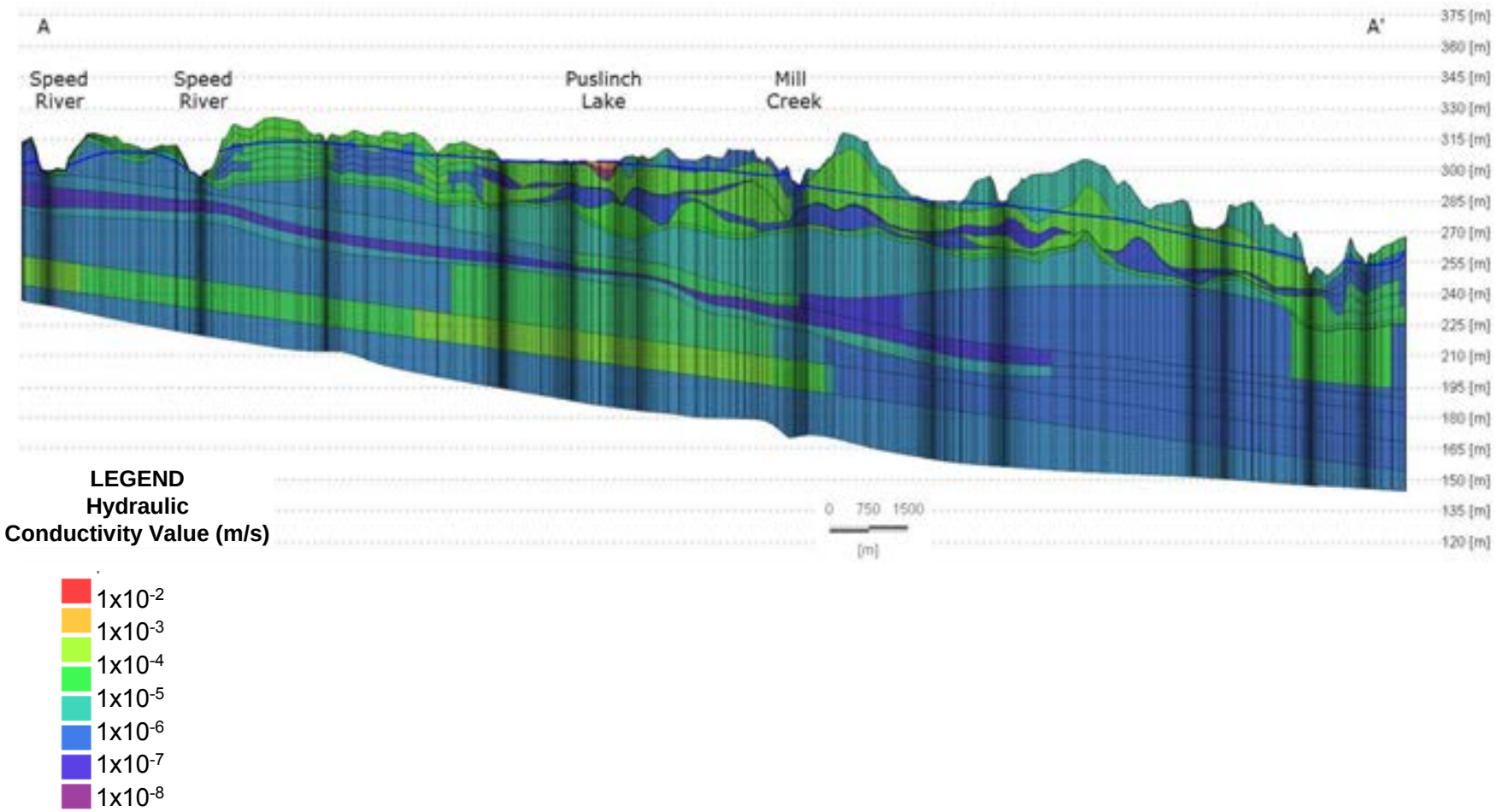
Not to scale

Layer 15 represents the lower part of the Gasport Formation and the four units R-I-R-M

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 3 Bedrock Model Layers

Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 4 FEFLOW Model Cross-Section A-A'

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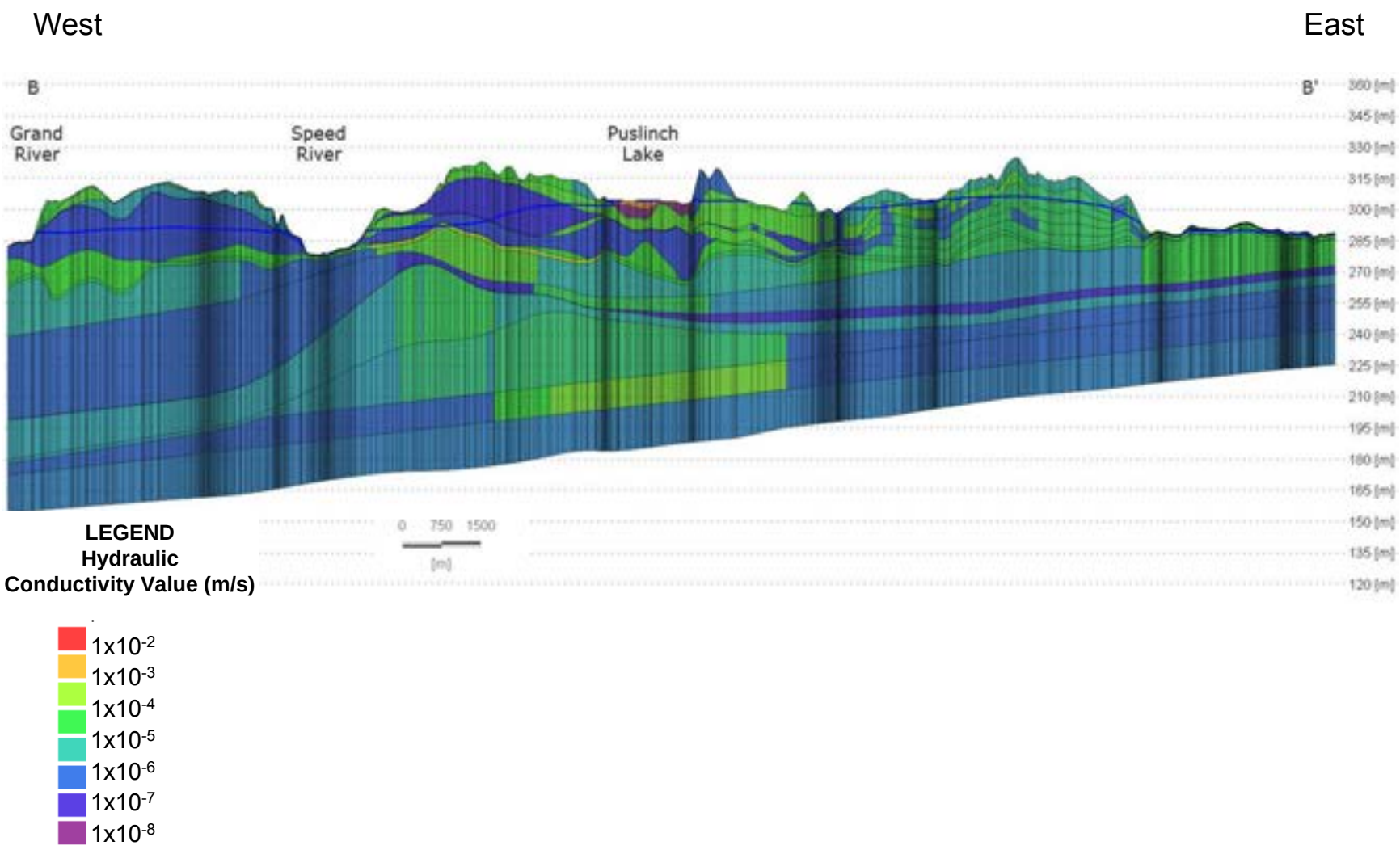

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 5 FEFLOW Model Cross-Section B-B'

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Specified-head conditions
- No flow boundary
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

The red arrows denote the section of the model boundary to which the label refers.

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 6a

Model Boundary Conditions

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Specified-head conditions
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 6b
Internal Specified-Head Conditions Representing Surface Water Features

0 0.5 1 2 3 4 5
Kilometers

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Recharge Distribution (mm/yr)

- < 50
- 50 - 100
- 100 - 150
- 150 - 200
- 200 - 250
- 250 - 300
- 300 - 350
- 350 - 400
- > 400

Scale 1:125,000

2.5 0 2.5 5

Kilometres

REFERENCES:
Base Data - Region of Waterloo, 2008; GRCA, 2008
Recharge Data - AquaResource, 2009
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Projection: UTM Zone 17N, NAD 83
Map Version: 1; Map Date: 10-06-2010; Created By: ccurry

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Figure 7
Recharge Distribution for the Cambridge Model

T:\2008027_ROW_IUS\GIS\ArcView\MXDs\SSPA\Fig7_Recharge_Distribution_Applied_in_Cambridge_Model.mxd

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well

Consumptive Use - Major Category

- Unknown
- Agricultural
- Commercial
- Dewatering
- Industrial
- Miscellaneous
- Recreational
- Remediation
- Water Supply

Consumptive Use (m^3/day)

- < 500
- 500 - 1000
- 1000 - 1500
- > 1500

- Expressway / Highway
- Major Roads
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Scale 1:125,000

2.5 0 2.5 5

Kilometres

REFERENCES:
Base Data - Region of Waterloo, 2008; GRCA, 2008
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Figure 8

Water Takings from Municipal and Non-municipal wells

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Calibration Targets: Privates Wells
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 9a

Steady-State Water Level Targets: MOE Water Well Records

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Calibration Targets: ROW Monitoring Wells
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 9b

Steady-State Water Level Targets: ROW Monitoring and Production Wells

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- GRCA Spot Flow Locations
- Tier Three Spot Flow Locations
- Gauge Stations
- Expressway / Highway
- Major Roads
- Road
- Rivers / Streams
- Lakes/Ponds
- Wetlands
- Model Domain

Scale 1:125,000

2.5 0 2.5 5

Kilometres

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Figure 9c

Streamflow Measurement and Gauging Station Locations

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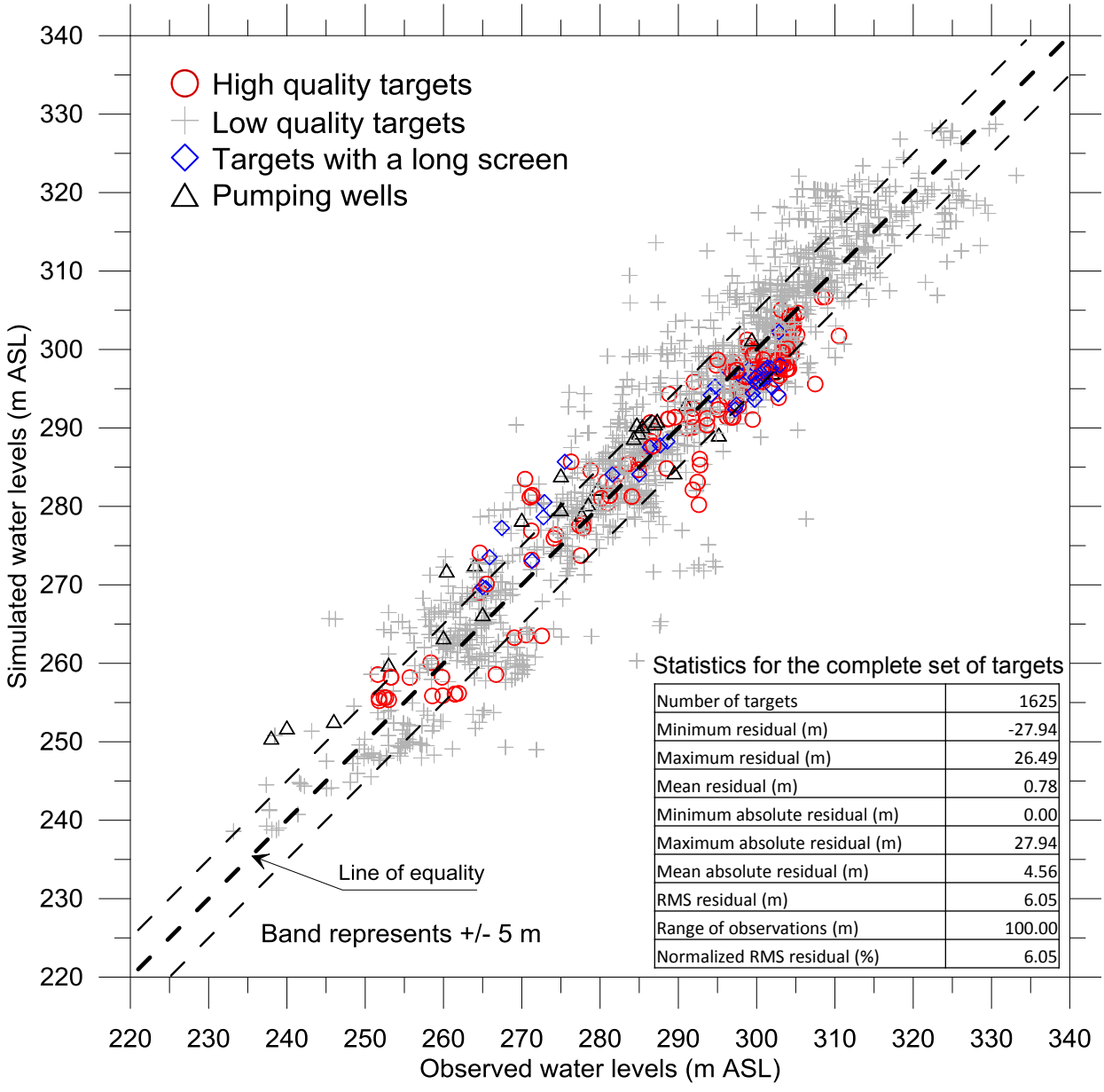
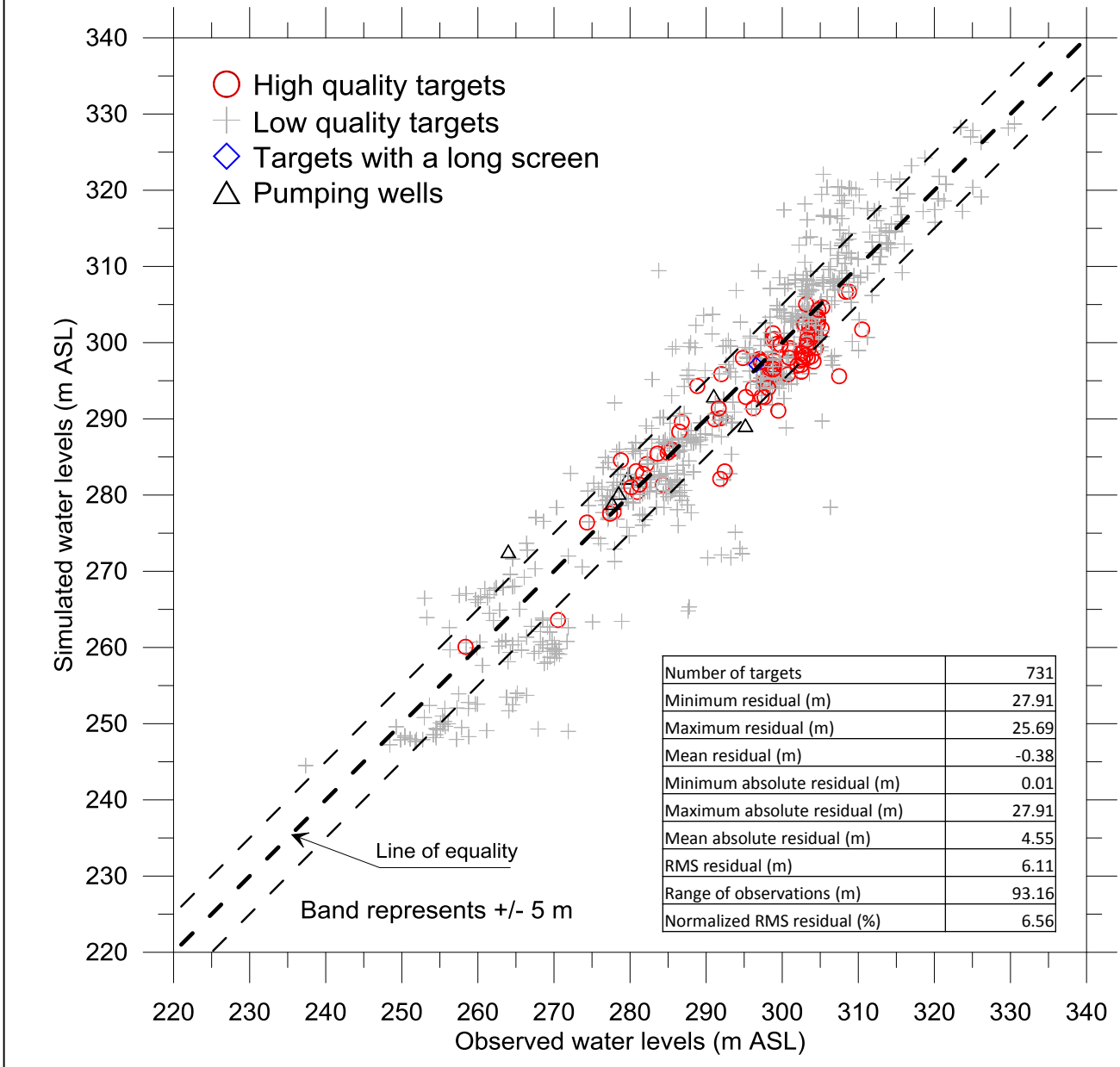


CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 10a Calibration Results: Simulated vs. Observed Groundwater Levels (all)

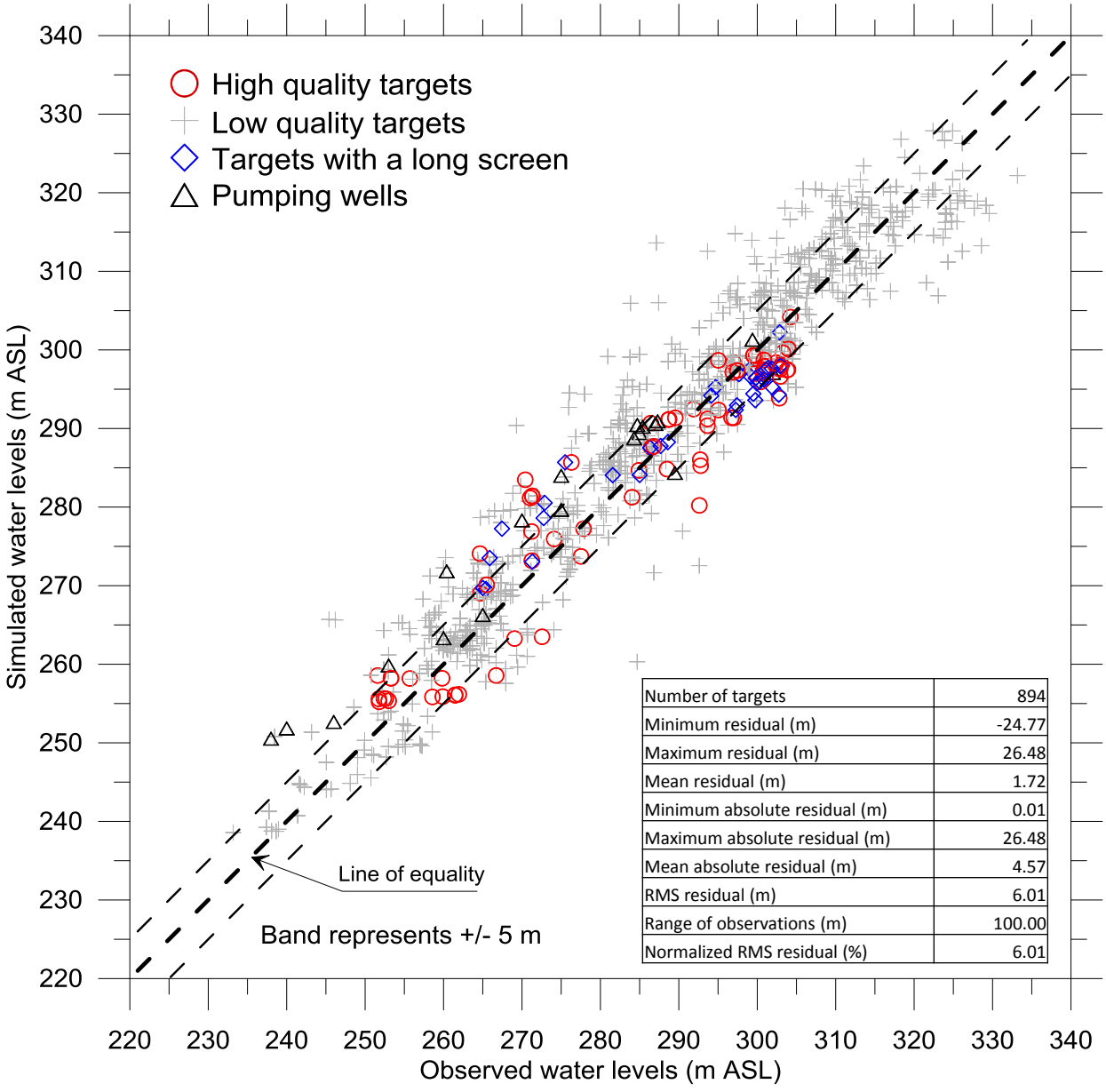
Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 10b Calibration Results: Simulated vs. Observed Groundwater Levels (Overburden and Contact Aquifer)

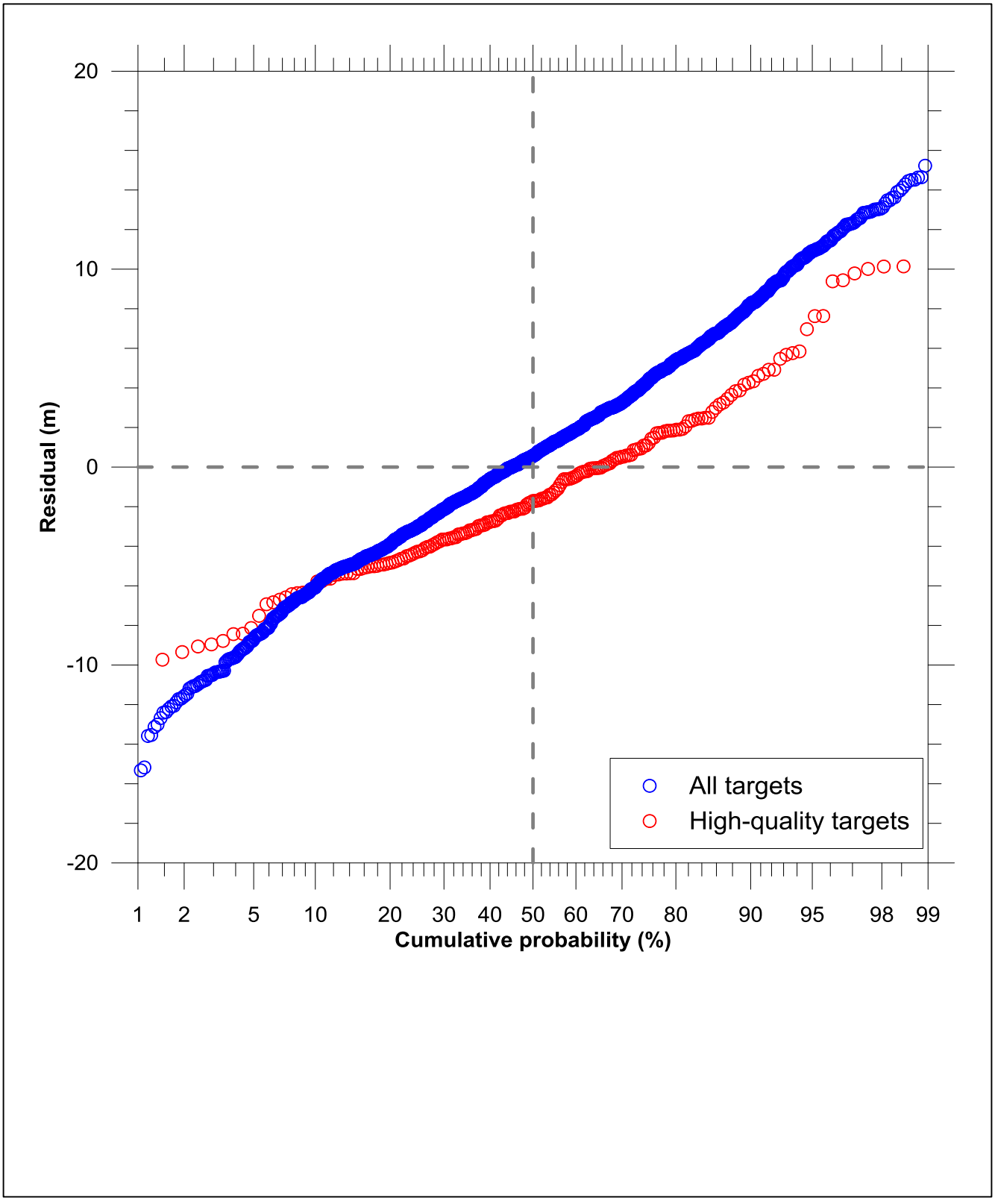
Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 10c Calibration Results: Simulated vs. Observed Groundwater Levels (Bedrock Aquifers)

Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 10d Cumulative Probability Distribution of the Residuals for the Steady-State Calibration Targets

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well

Simulated Head - Observed Head (m)

- < 10.0; Simulated Head Lower than Observed
- 10.0 - 5.0
- 5.0 - 0.0
- 0.0 - 5.0
- 5.0 - 10.0
- > 10.0; Simulated Head Greater than Observed
- Expressway / Highway
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Scale 1:125,000

2.5 0 2.5 5

Kilometres

REFERENCES:
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Projection: UTM Zone 17N, NAD 83
Map Version: 1; Map Date: 10-06-2014; Created By: ccurry

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Figure 11a

Spatial Distribution of Residuals for Calibrated Model (Overburden)

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well

Simulated Head - Observed Head (m)

- < 10.0; Simulated Head Lower than Observed
- 10.0 - 5.0
- 5.0 - 0.0
- 0.0 - 5.0
- 5.0 - 10.0
- > 10.0; Simulated Head Greater than Observed
- Expressway / Highway
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Scale 1:125,000

2.5 0 2.5 5

Kilometres

REFERENCES:
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Figure 11b

Spatial Distribution of Residuals for Calibrated Model (Bedrock)

T:\2008027_ROW_IUS\GIS\ArcView\MXDs\SSPA\Fig11b_SpatialDistribution_Residuals_CalibratedModel_BK.mxd

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Water level (5 m interval)

Simulated Head - Observed Head (m)

- > 10.0; Simulated Head Lower than Observed
- 10.0 - 5.0
- 5.0 - 0.0
- 0.0 - 5.0
- 5.0 - 10.0
- > 10.0; Simulated Head Greater than Observed
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 12a
Simulated Groundwater Levels in the Contact Aquifer

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Water level (5 m interval)

Simulated Head - Observed Head (m)

- > 10.0; Simulated Head Lower than Observed
- 10.0 - 5.0
- 5.0 - 0.0
- 0.0 - 5.0
- 5.0 - 10.0
- > 10.0; Simulated Head Greater than Observed
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 12b
Simulated Groundwater Levels in the Middle Gasport Formation

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

Water Level Difference Between Contact Aquifer and the Middle Gasport (2 m interval)

- Upward gradient
- Zero gradient
- Downward gradient
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 12c
Simulated Differences in Groundwater Levels between Upper and Lower Bedrock Systems

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Losing stream (m^3/d)
- 0 - 5
- 6 - 10
- 11 - 25
- 25 - 100
- >100

Gaining stream (m^3/d)
- > 100
- 25 - 100
- 10 - 25
- 5 - 10
- 0 - 5

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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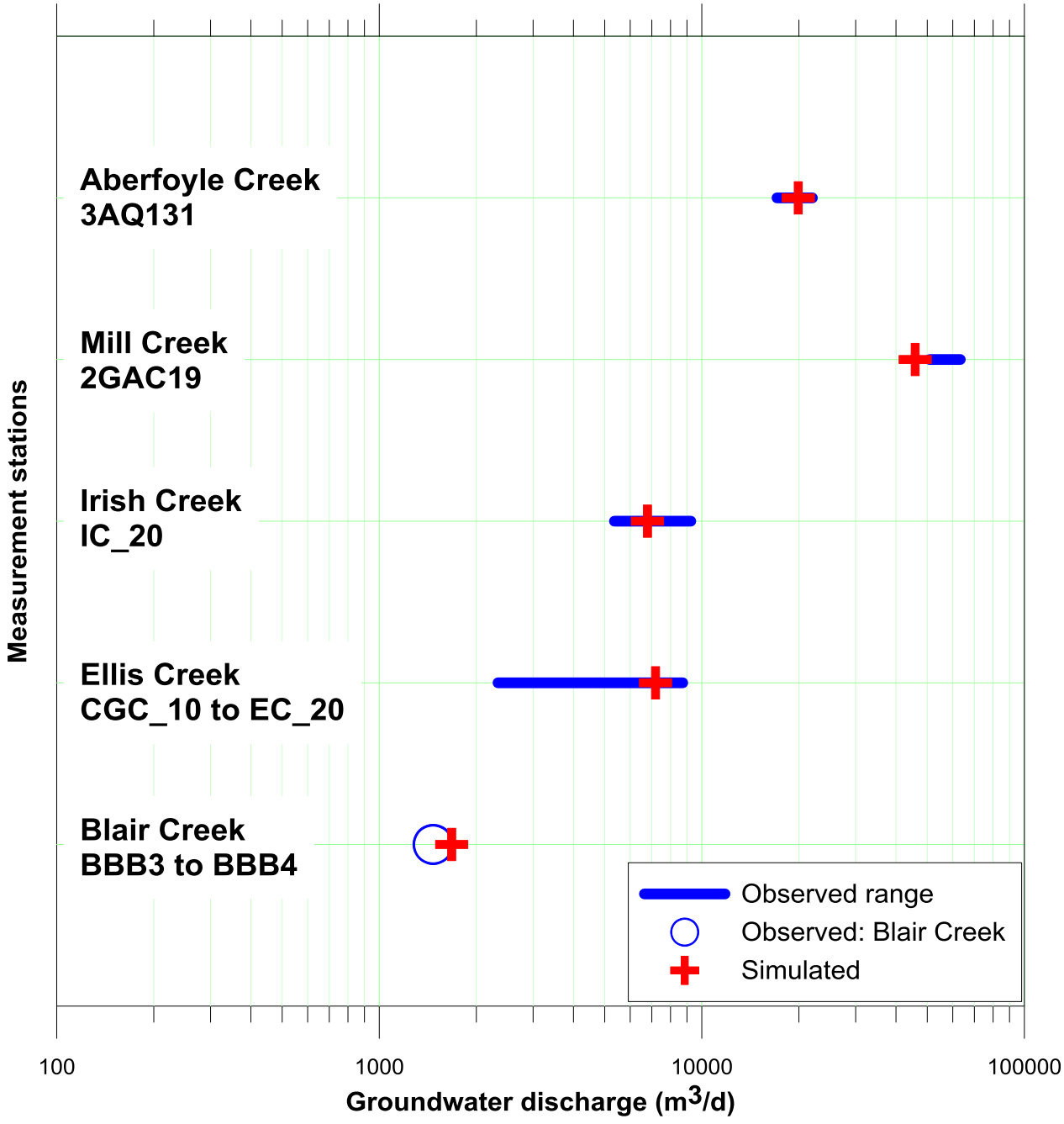
Figure 13
Model Simulated Gaining and Losing Streams



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 14 Baseflow Calibration: Measured and Simulated Values

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- Observation Wells
- Wells Pumped during the 28-day Pumping Test
- Expressway / Highway
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Wetlands
- Model Domain

Scale 1:27,500

0.5 0 0.5 1

Kilometres

REFERENCES:
Base Data - Region of Waterloo, 2008; GRCA, 2008
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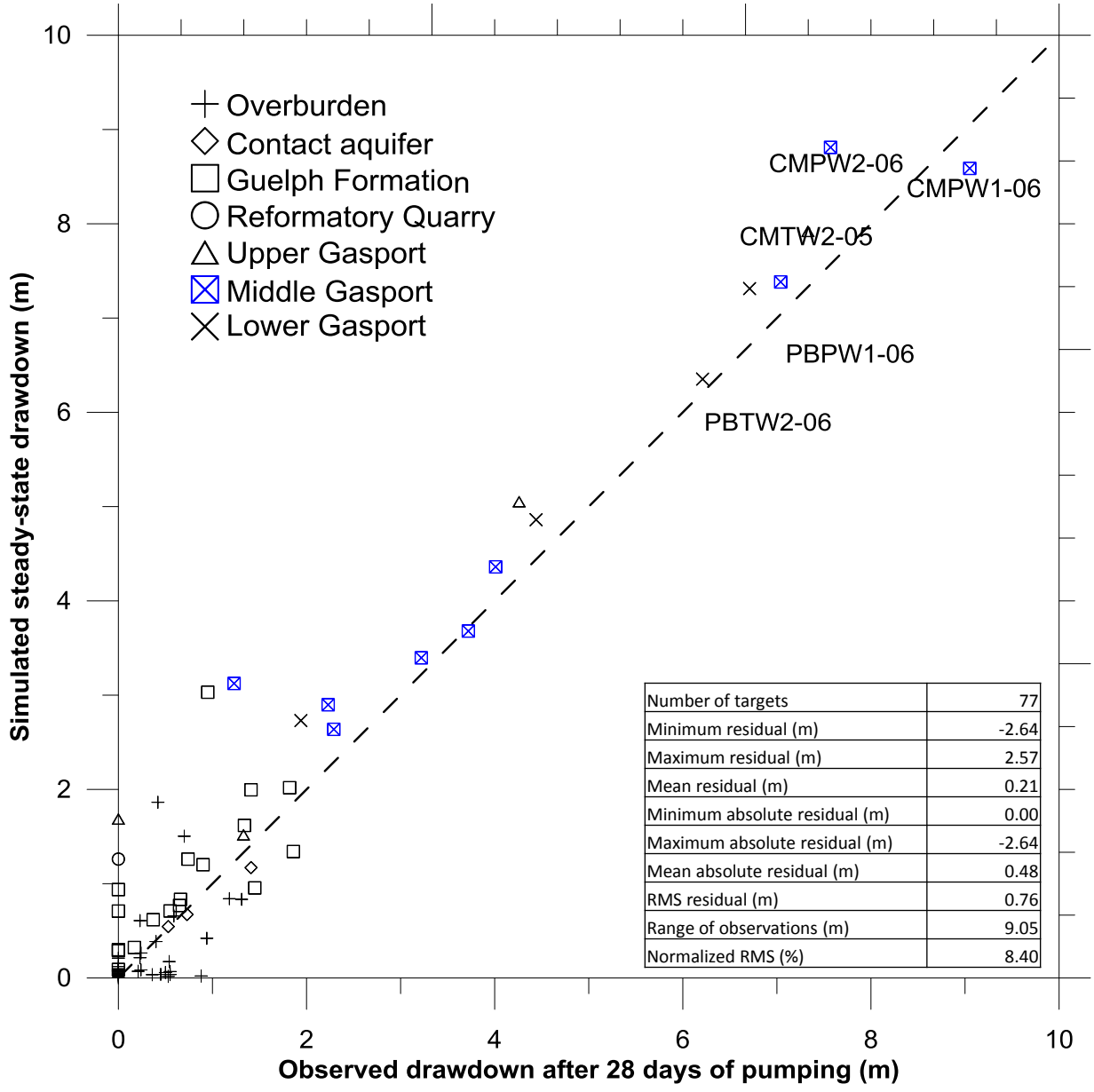
Projection: UTM Zone 17N, NAD 83
Map Version: 1; Map Date: 10-06-2014; Created By: ccurry

Figure 15
28-Day Pumping Test
Well Locations

T:\2008027_ROW_IUS\GIS\ArcView\MXDs\SSPA\Fig15_28DayPumpTest_WellLocs.mxd

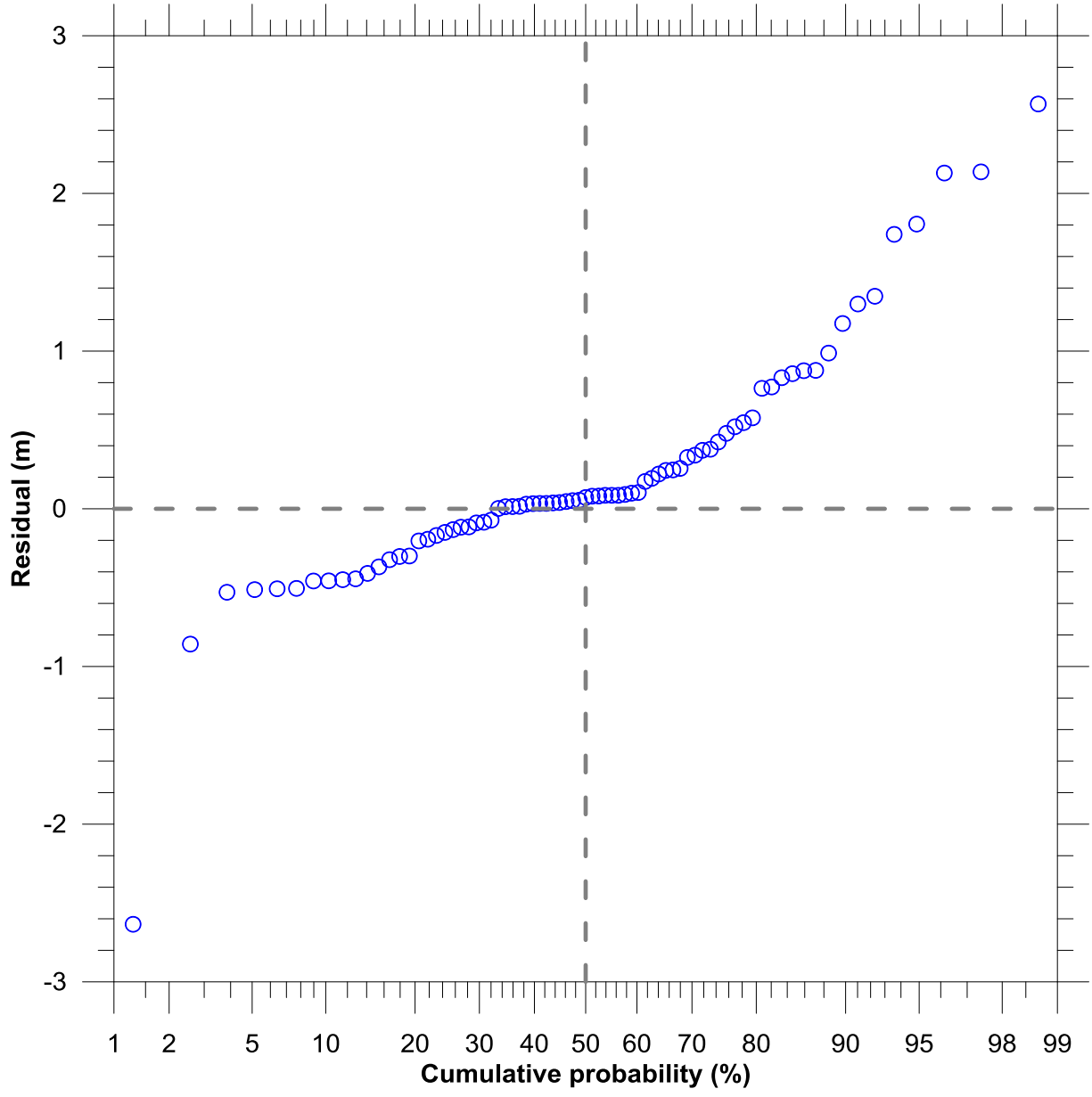


Number of targets	77
Minimum residual (m)	-2.64
Maximum residual (m)	2.57
Mean residual (m)	0.21
Minimum absolute residual (m)	0.00
Maximum absolute residual (m)	-2.64
Mean absolute residual (m)	0.48
RMS residual (m)	0.76
Range of observations (m)	9.05
Normalized RMS (%)	8.40

CAMBRIDGE EAST IUS WATER SUPPLY CLASS: GROUNDWATER MODELLING

Figure 16a 28-day Pumping Test Calibration Results: Simulated vs. Observed Drawdowns at the End of the Test (All)

Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 16b Cumulative Probability Distributions for the 28-Day Pumping Test

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well

Drawdown residuals (m)

- < 0.5; Simulated drawdown less than observed
- 0.5 - 0.25
- 0.25 - 0.0
- 0.0 - 0.25
- 0.25 - 0.5
- > 0.5; Simulated drawdown greater than observed
- Simulated drawdown in the Contact Aquifer
- Expressway / Highway
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Lable for drawdown residual:
Observed Value (m) = 1.1
Simulated Value (m) = 1.2

Scale 1:55,000

1 0 1 2

Kilometres

REFERENCES:
Base Data - Region of Waterloo, 2008; GRCA, 2008
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Projection: UTM Zone 17N, NAD 83
Map Version: 1; Map Date: 10-06-2014; Created By: ccurry

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Figure 17a

Simulated Drawdowns in the Contact Aquifer at the end of 28 days

T:\2008027_ROW_IUS\GIS\ArcView\MXDs\SSPA\Fig17a_Simulated_Declines_in_the_Contact_Zone_after_28Day_Pumping_Test.mxd

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well

Drawdown residuals (m)

- < 0.5;Simulated drawdown less than Observed
- 0.5 - 0.25
- 0.25 - 0.0
- 0.0 - 0.25
- 0.25 - 0.5
- > 0.5;Simulated drawdown more than Observed
- Expressway / Highway
- Simulated Drawdown in the Middle Gasport Fm. (m)
- Major Roads
- Roads (collector)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

Label for drawdown residual:

Observed Value (m) = 1.1
Simulated Value (m) = 1.2

Scale 1:55,000

1 0 1 2

Kilometres

REFERENCES:
Base Data - Region of Waterloo, 2008; GRCA, 2008
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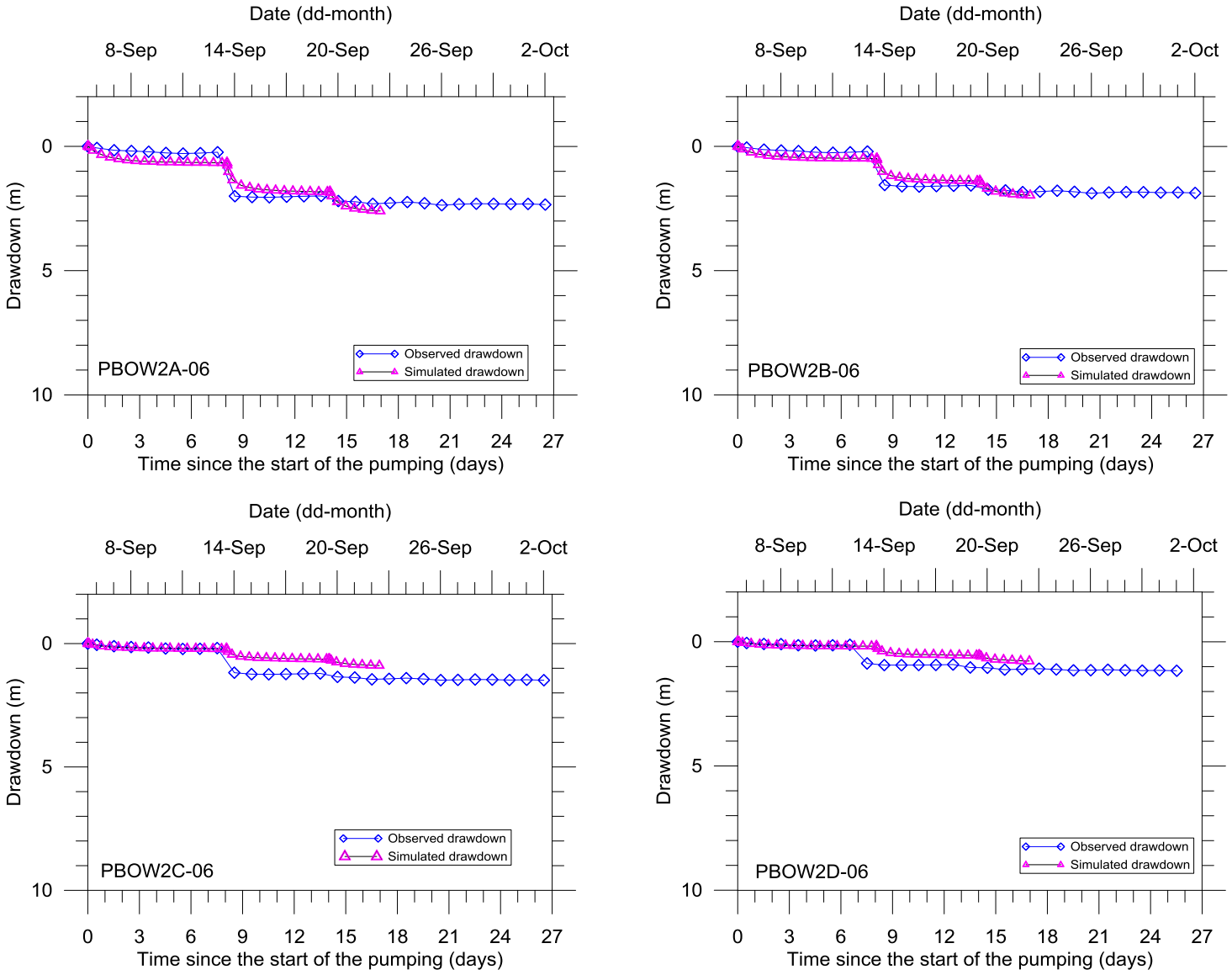
Projection: UTM Zone 17N, NAD 83
Map Version: 1; Map Date: 10-06-2014; Created By: ccurry

Matrix Solutions Inc. ENVIRONMENT & ENGINEERING

Figure 17b

Simulated Drawdowns in the Middle Gasport Fm. after 28 Days

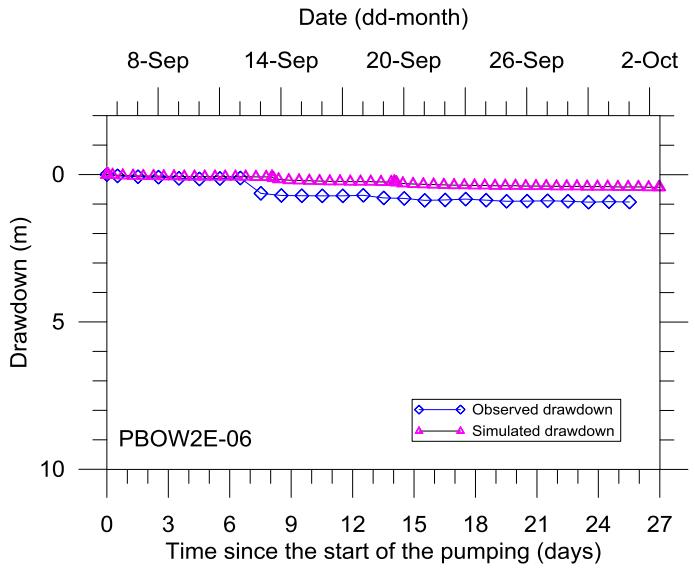
T:\2008027_ROW_IUS\GIS\ArcView\MXDs\SSPA\Fig17b_Simulated_Drawdown_Contours_after_28Day_Pumping_Test_GasportFormation.mxd

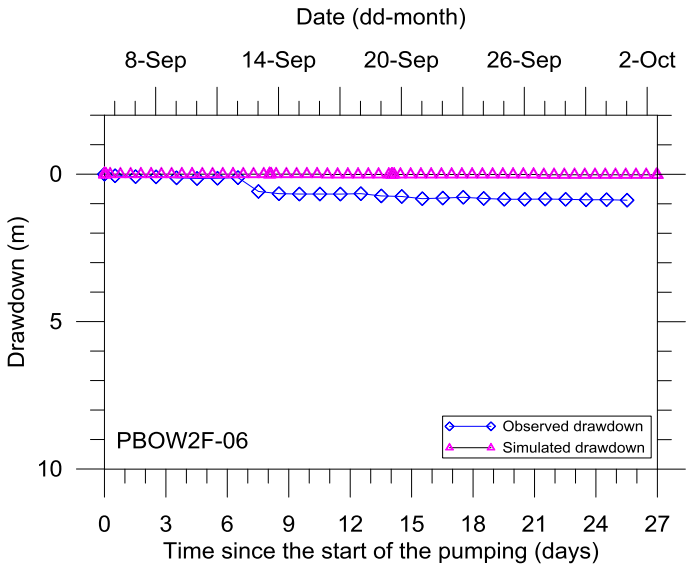


CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 18a 28-Day Pumping Test Simulated vs. Measured Drawdowns at Selected Monitoring Well Nests (PBOW2A, B, C, D-06)

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING

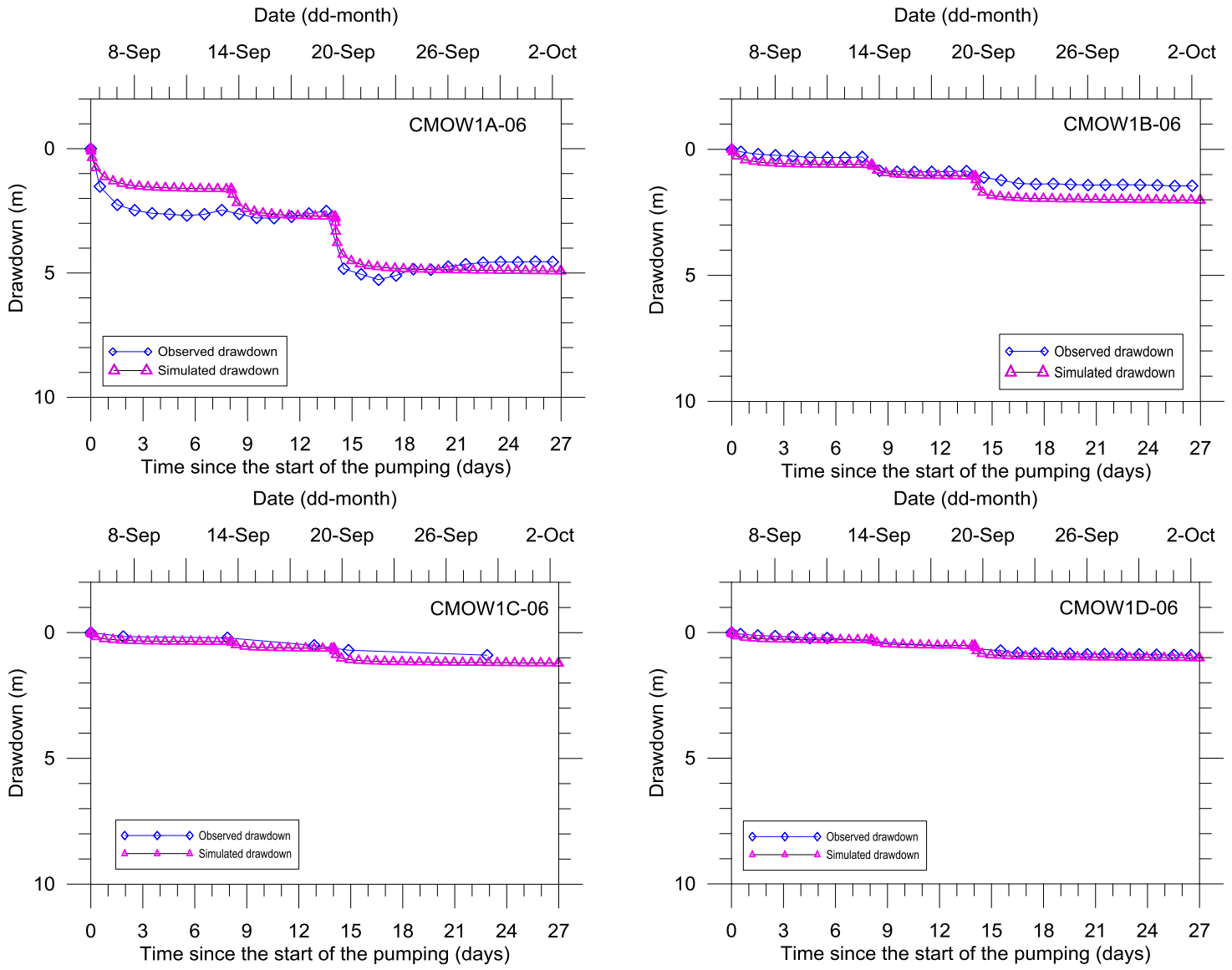




CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 18b 28-Day Pumping Test Simulated vs. Measured Drawdowns at Selected Monitoring Well Nests (PBOW2E,F-06)

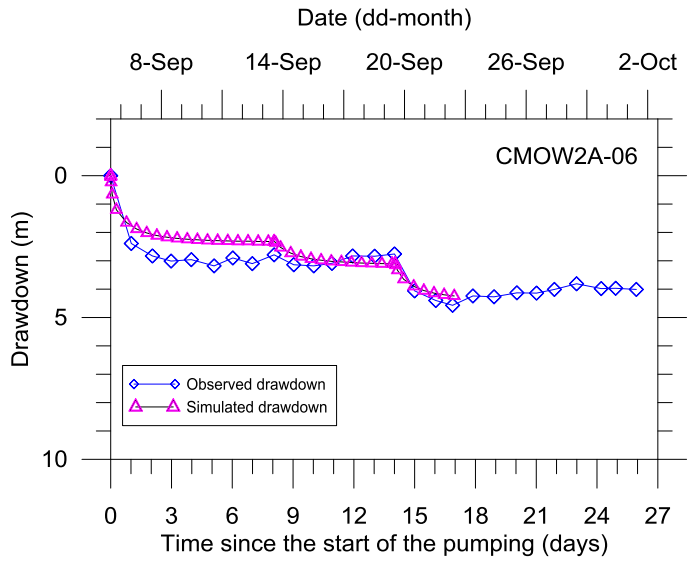
Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING

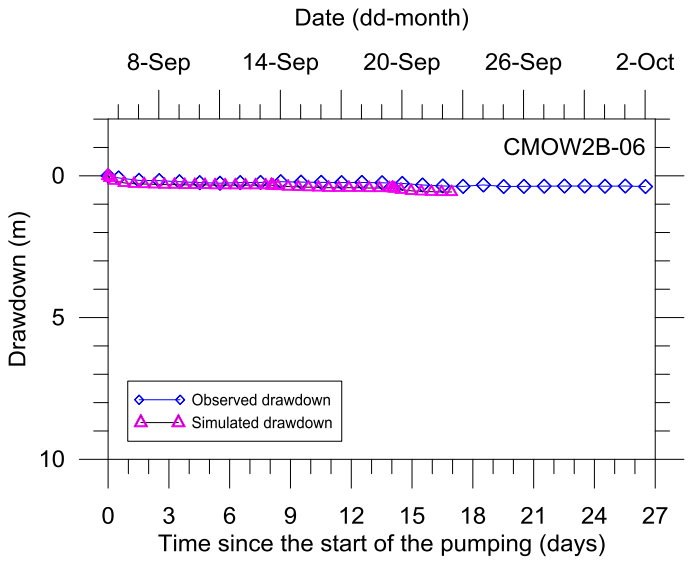


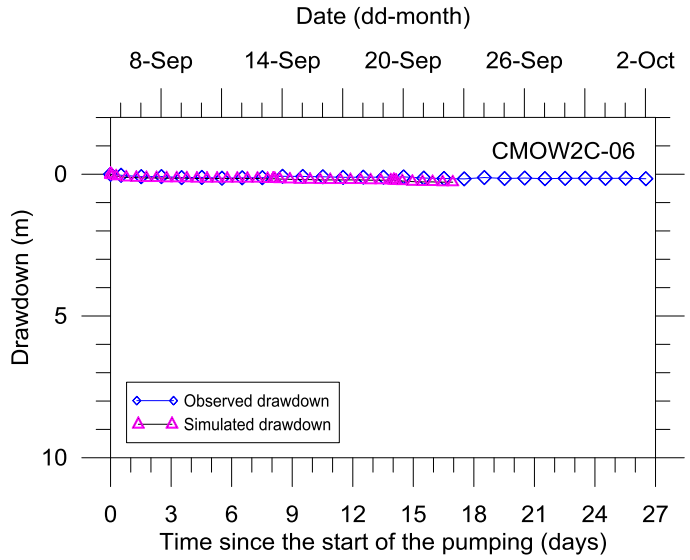
CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 18c 28-Day Pumping Test Simulated vs. Measured Drawdowns at Selected Monitoring Well Nests (CMOW1-06)

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING



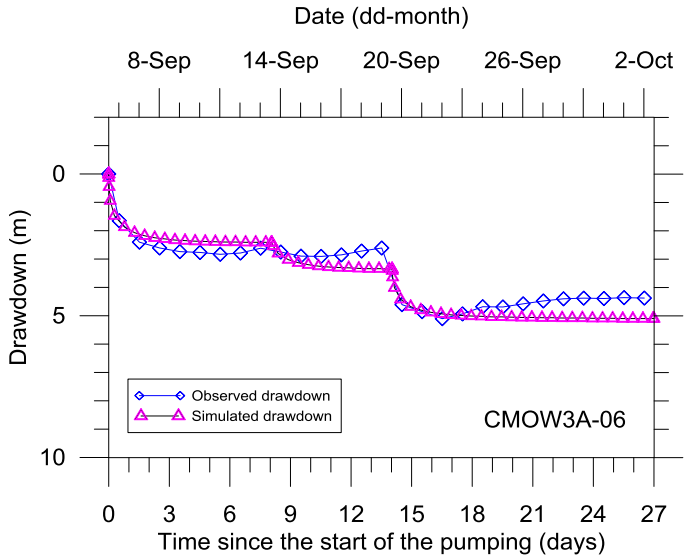


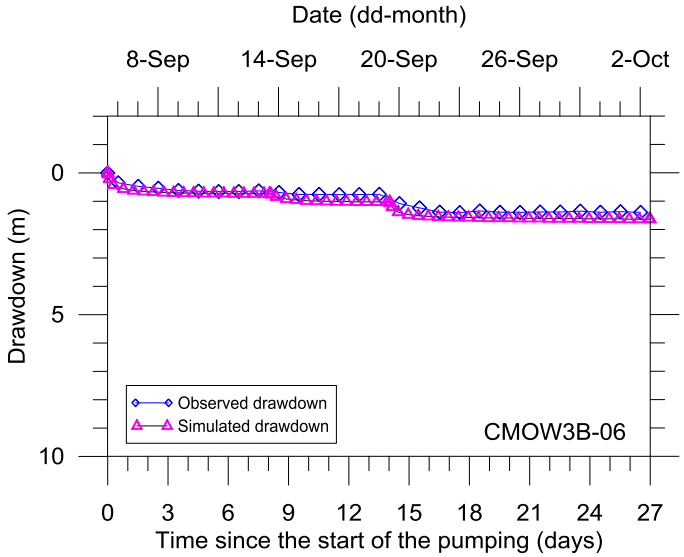


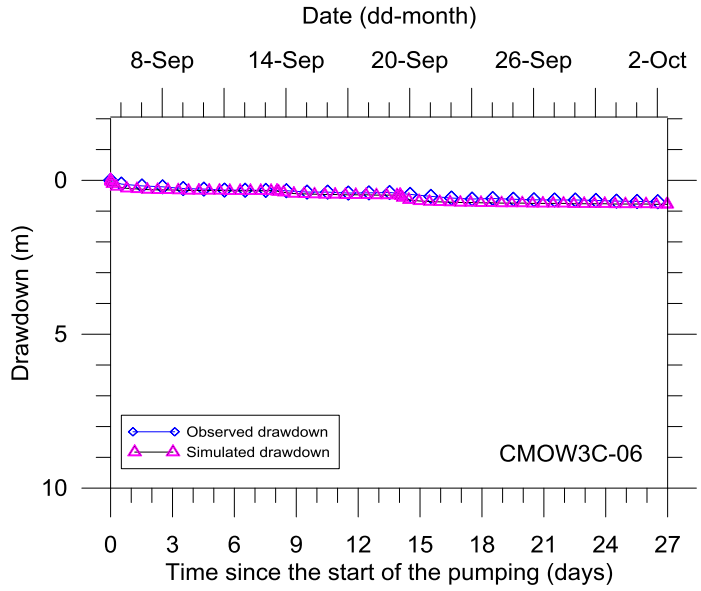
CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 18d 28-Day Pumping Test Simulated vs. Measured Drawdowns at Selected Monitoring Well Nests (CMOW2-06)

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING







CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 18e 28-Day Pumping Test Simulated vs. Measured Drawdowns at Selected Monitoring Well Nests (CMOW3-06)

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Clyde Park Pumping Well
- Monitoring Wells
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:35,000

1 0.5 0 1

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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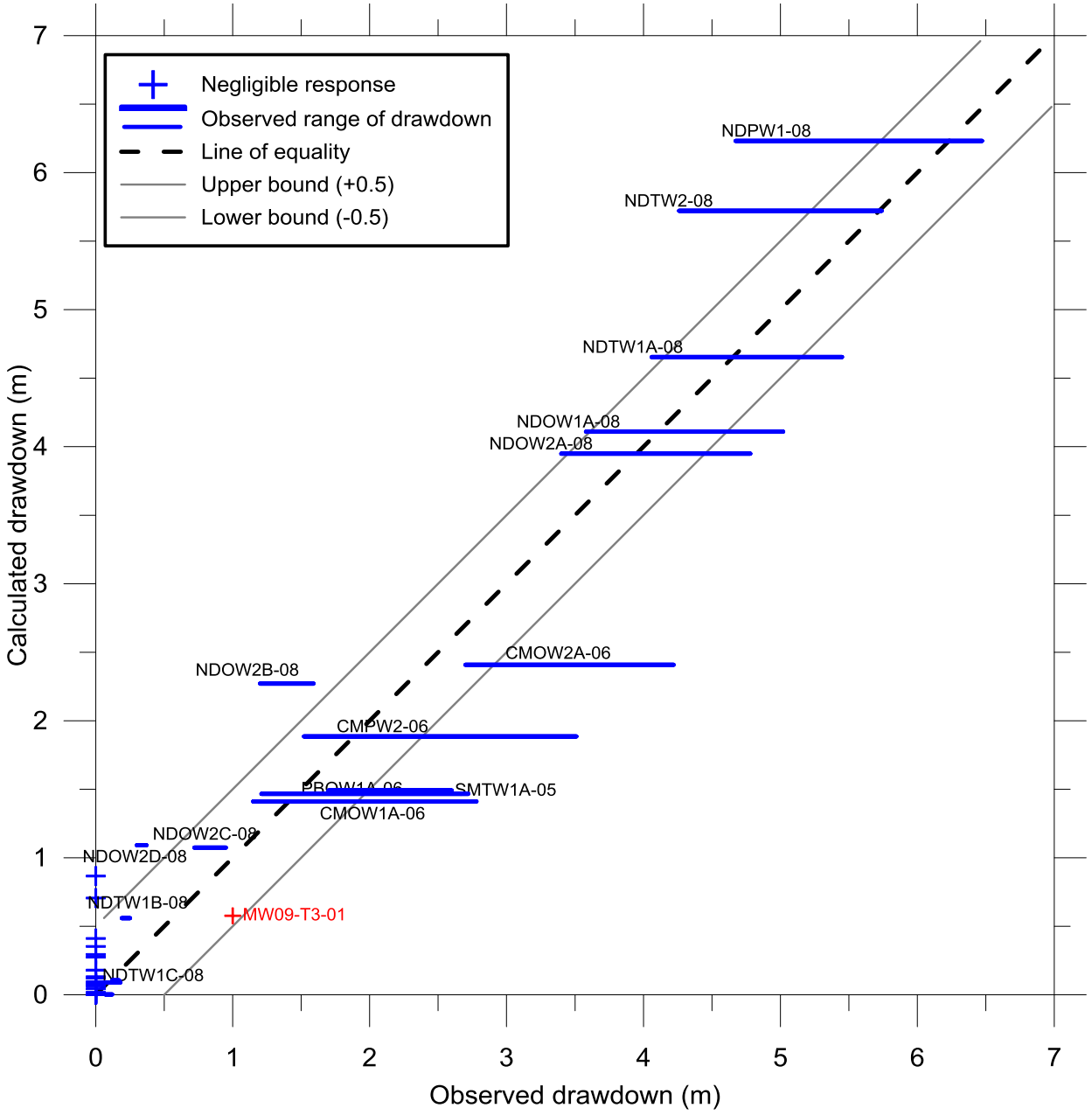
Figure 19

Clyde-Park 6-Day Pumping Test
-- Well locations



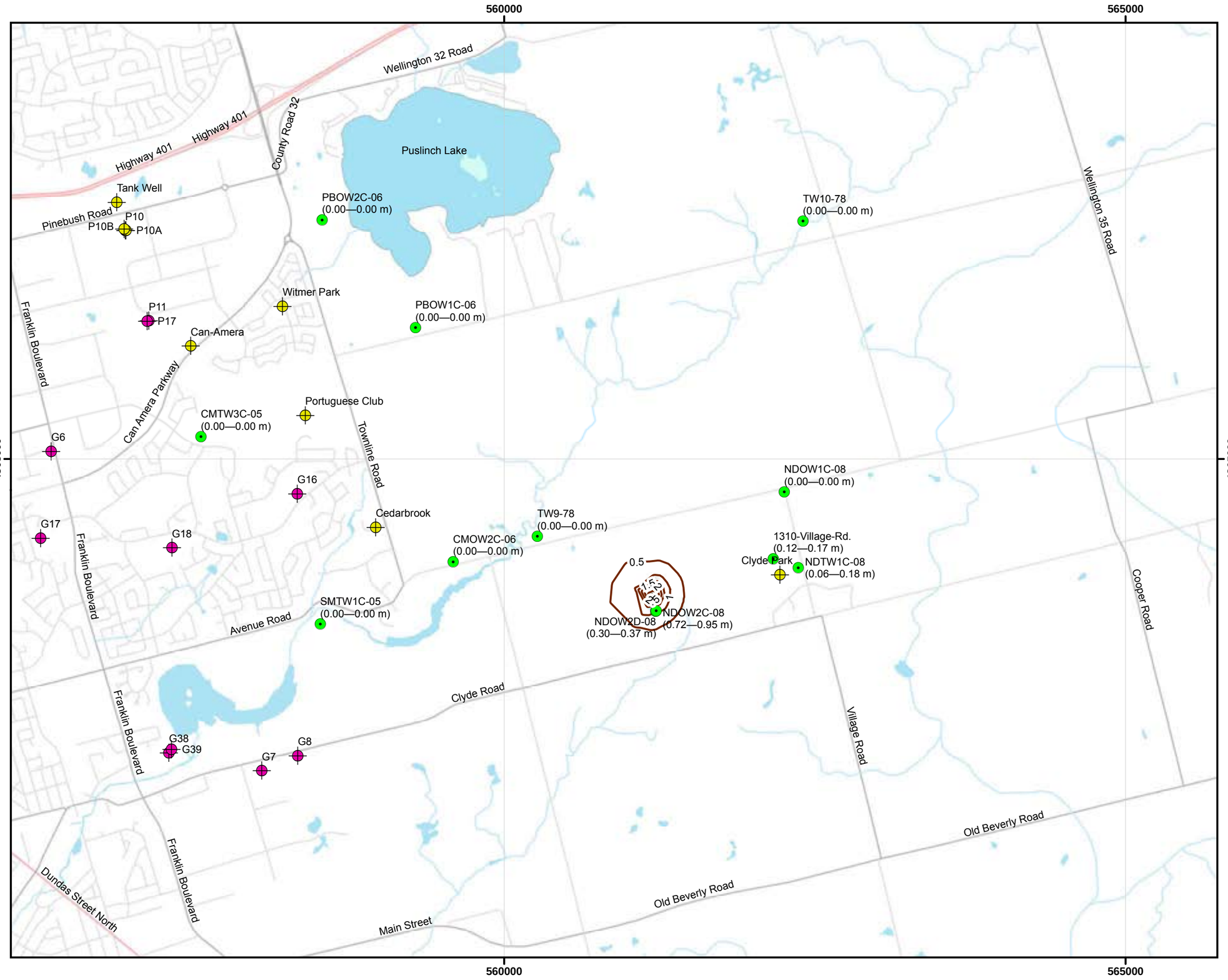
1) Maximum drawdowns were calculated using static levels prior to step-test on September 23, 2011; while minimum drawdowns were calculated using static levels immediately prior to the constant-rate test. Golder Associates conducted the drawdown calculations and provided to SSPA for model calibration.
2) For the negligible response observation points, no ranges of drawdown are provided.
3) Interference at MW09-T3-01-2 during the test due to a South Guelph pumping test at GSTW1-08 were observed. Drawdown were estimated based on recovery data.

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 20 Clyde Park Pumping Test Calibration Results: Simulated vs. Observed Drawdowns at the End of the Test (All)

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling



LEGEND

- Monitoring Well (Observed Drawdown)
- Clyde Park Pumping Well
- Municipal Supply Well
- Test Production Well
- Simulated Drawdown in the Contact Aquifer (0.5 m inerval)
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:30,000

1 0.5 0 1

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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ENVIRONMENT & ENGINEERING

Figure 21a

Simulated Declines at the End of the Clyde-Park 6-Day Pumping Test (Contact Aquifer)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Monitoring Well (Observed Drawdown)
- Clyde Park Pumping Well
- Municipal Supply Well
- Test Production Well
- Simulated Drawdown in the Gsport Formation (0.5 m inerval)
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:40,000

1 0.5 0 1

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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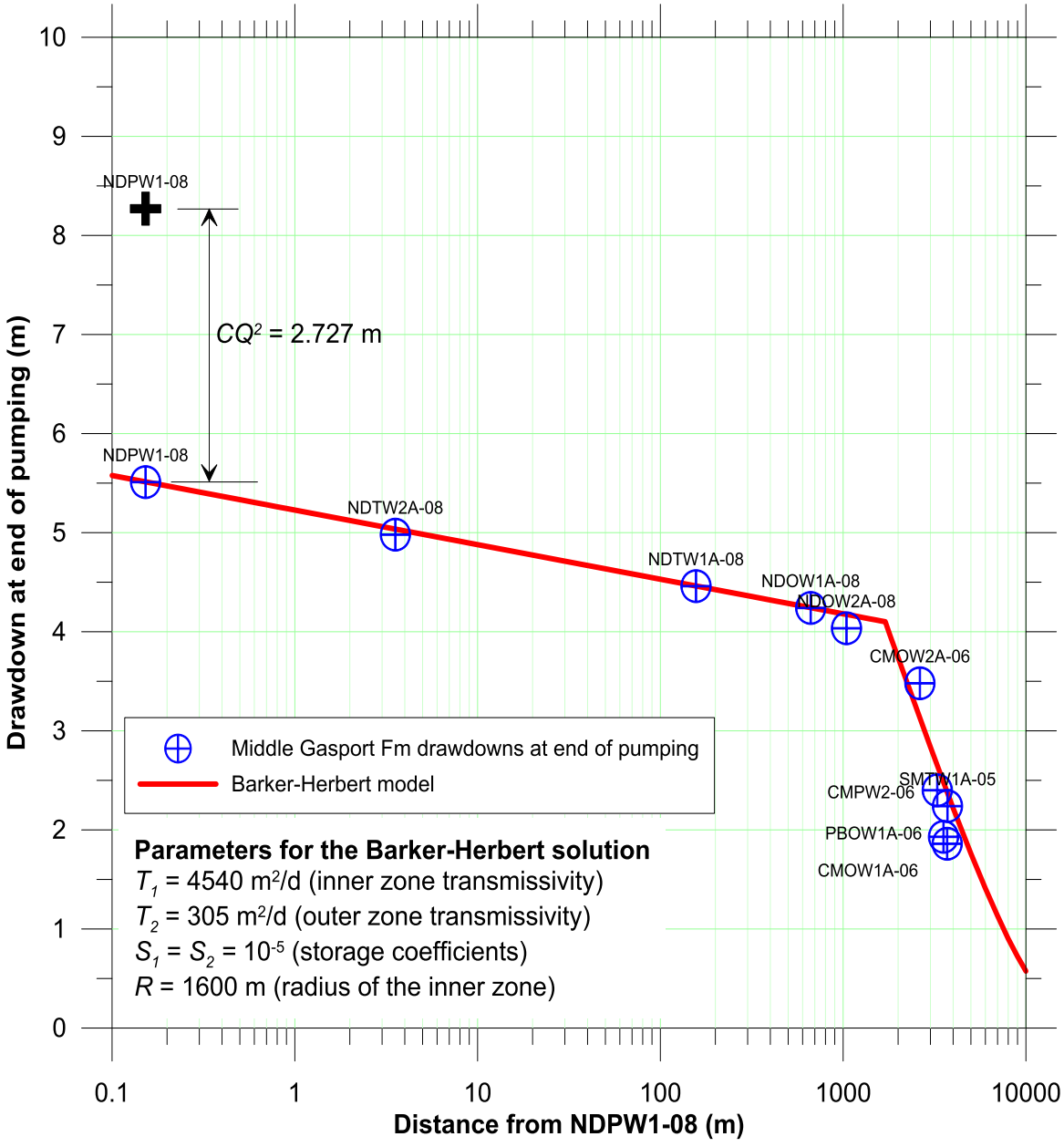
Figure 21b

Simulated Drawdowns at the End of the Clyde Park 6-Day Pumping Test (Gasport Formation)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 22 Match to Drawdowns with Analytical Solution of Barker and Herbert (1982)

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

555000

560000

4805000

Maple Grove Road
Fisher Mills Road
Queen Street West
Hespeler Road
Townline Road
Wellington 32 Road
Franklin Boulevard
County Road 32
Highway 401
Pinebush Road
Can Amera Parkway
Eagle Street North
Bishop Street North
Coronation Boulevard
Avenue Road

H4-TW1D-09
H4
H4-TW1ABC-08
H4A
C-CA-OW2-95
H3-TW1ABC-09
H3-TW1DE-10
H3
H3A
H65-1
C-PB-OW1-10
RW2 (31 Lake Ave)
Puslinch Lake
DP4-11
DP2-11
Tank Well (TW1-10)
RW1 (413 Pinebush Road)
DP6-11
P10BC-PB-TW1ABC-09
DP1-11
C-PB-TW1D-10
DP1-10SDP1-10D
P-PB-OW2D-06P-PB-OW2ABC-06
C-PB-TW1-05
P-PB-OW2E-06P-PB-OW2F-06
C-CM-OW5-94
C-CA-OW3-95
G5-TW2ABC-08
G5-TW2D-08
G5A
G5
C-PB-OW1S-02
C-PB-OW1D-02
C-PB-OW1I-02
P15
P9
C-PB-TW2-06
Witmer Park
C-PB-PW1-06
P17
DP5-11B
DP5-11S
C-CM-OW1-92-SW
C-PB-OW1AB-11
C-CM-OW1A-92
DP3-11
C-PB-TW3-07
Can-Amera
C-CM-OW1BC-92
C-PB-PW2-08
P-PB-OW1E-06
P-PB-OW1ABC-06
P-PB-OW1D-06
P-PB-OW1F-06
Portuguese Club
C-CM-OW1E-06
C-CM-OW1ABC-06
C-CM-OW1D-06
C-CM-OW1F-06
C-CM-TW3-05
G6
G16
G17
G18
Cedarbrook
P6

LEGEND

- Pinebush 15-Day Test Pumping Wells
- Monitoring Wells
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:25,000

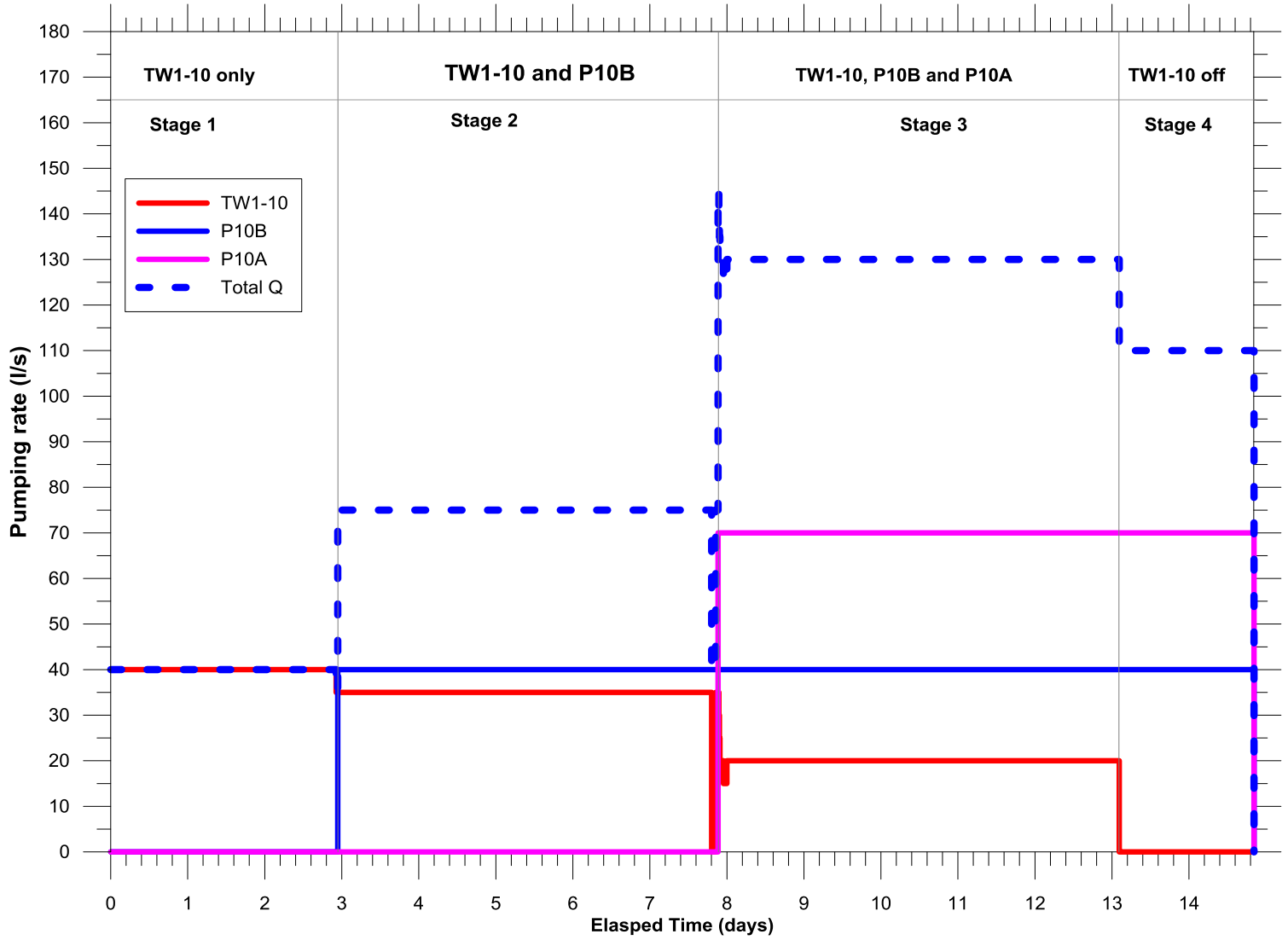
0.4 0.2 0 0.4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING

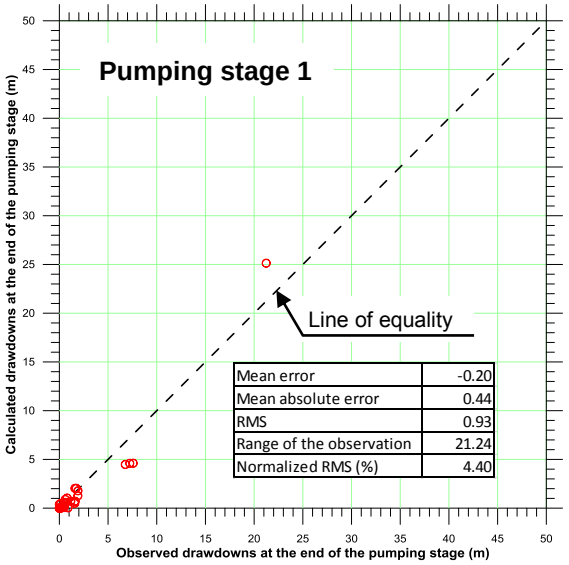
Figure 23
Pinebush 15-Day Pumping Test
-- Well Locations

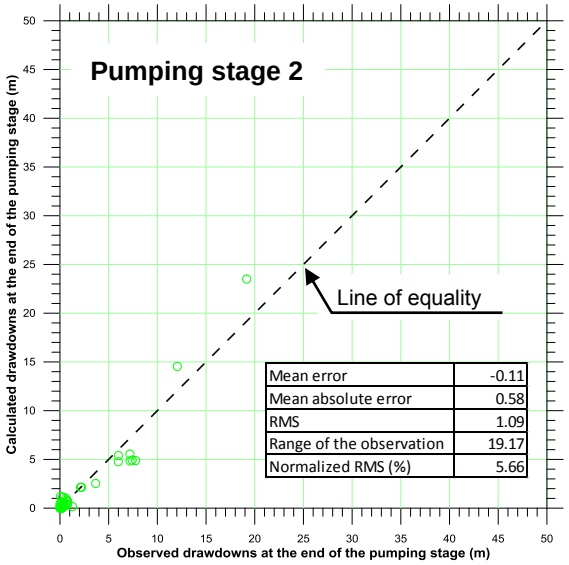


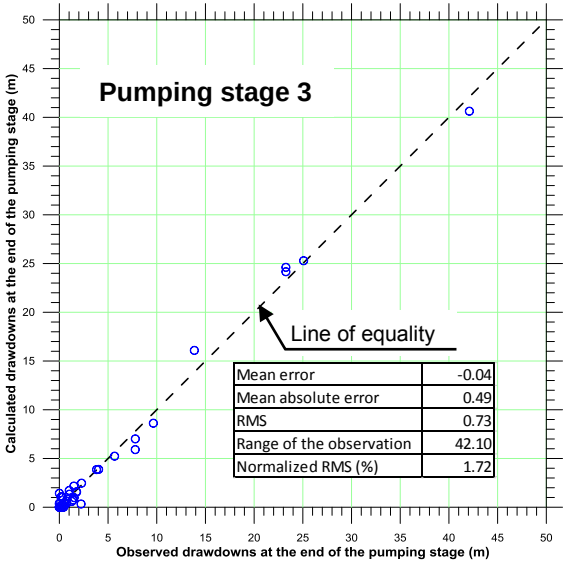
CAMBRIDGE EAST IUS WATER SUPPLY
CLASS EA: GROUNDWATER MODELLING

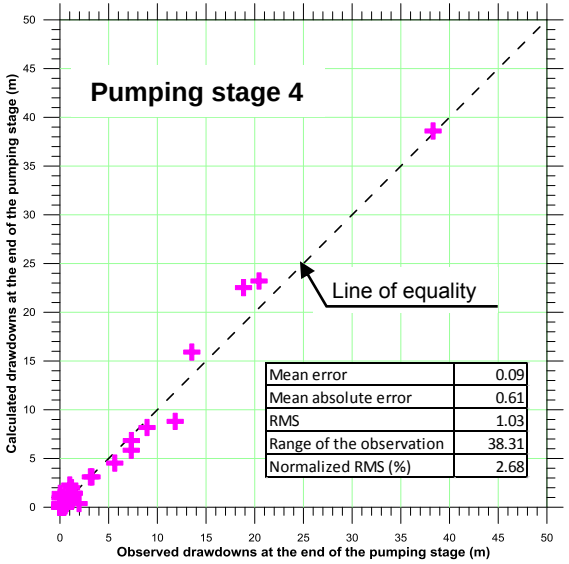
Figure 24 Pumping History of Pinebush 15-Day Pumping Test

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING





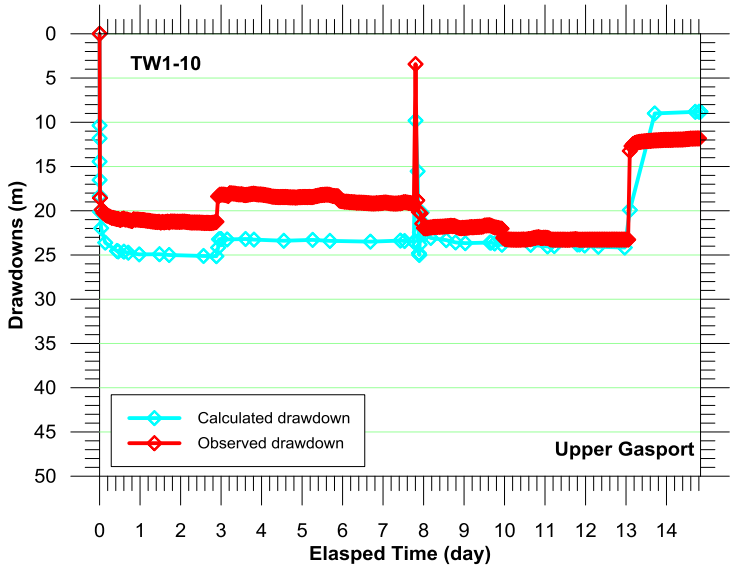


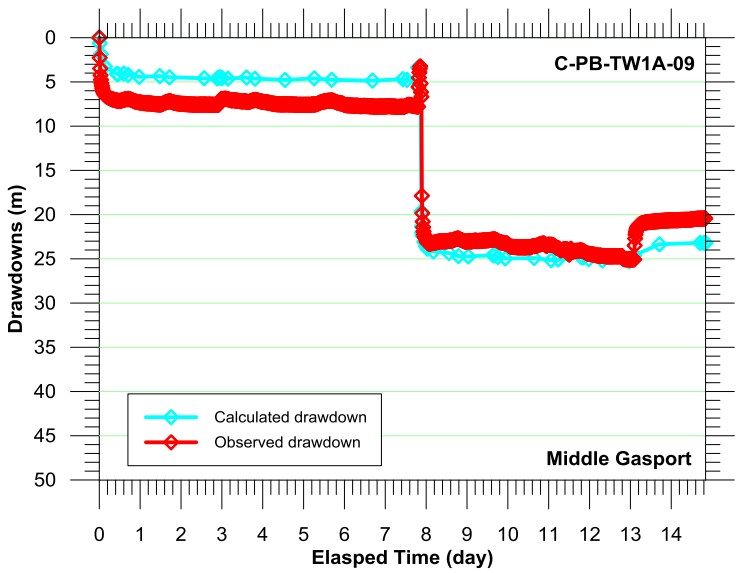


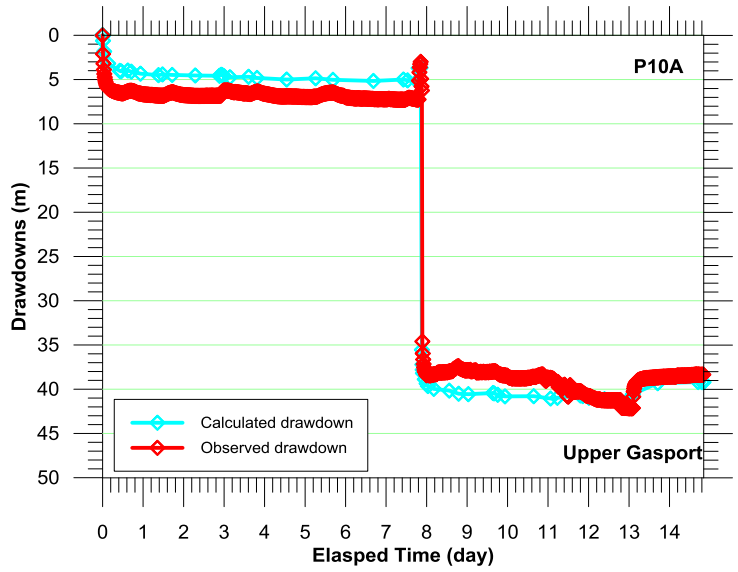
CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

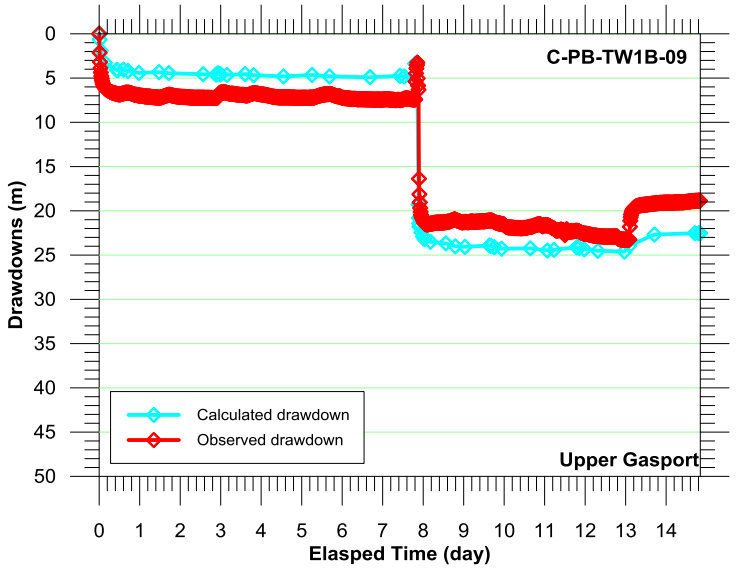
Figure 25 Pinebush 15-Day Pumping Test Calibration Results: Simulated vs. Observed Drawdowns at the End of Each Pumping Stage

Matrix Solutions Inc.





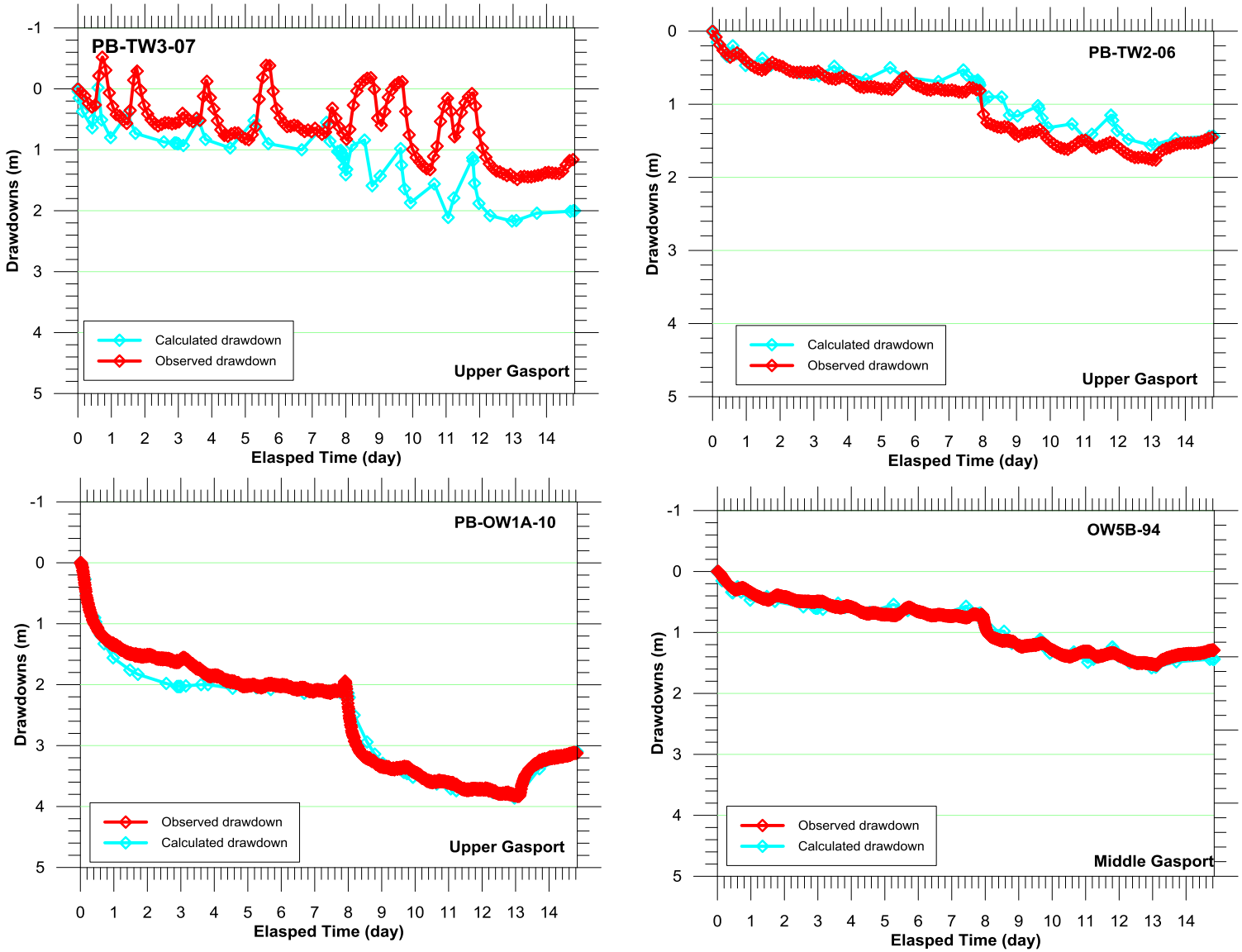




CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 26a Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Deep Bedrock (TW1-10, P10A, C-PB-TW1A,B-09)

Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 26b Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Deep Bedrock (TW3-07, TW2-06, OW1A-10 and OW5B-94)

Matrix Solutions Inc.
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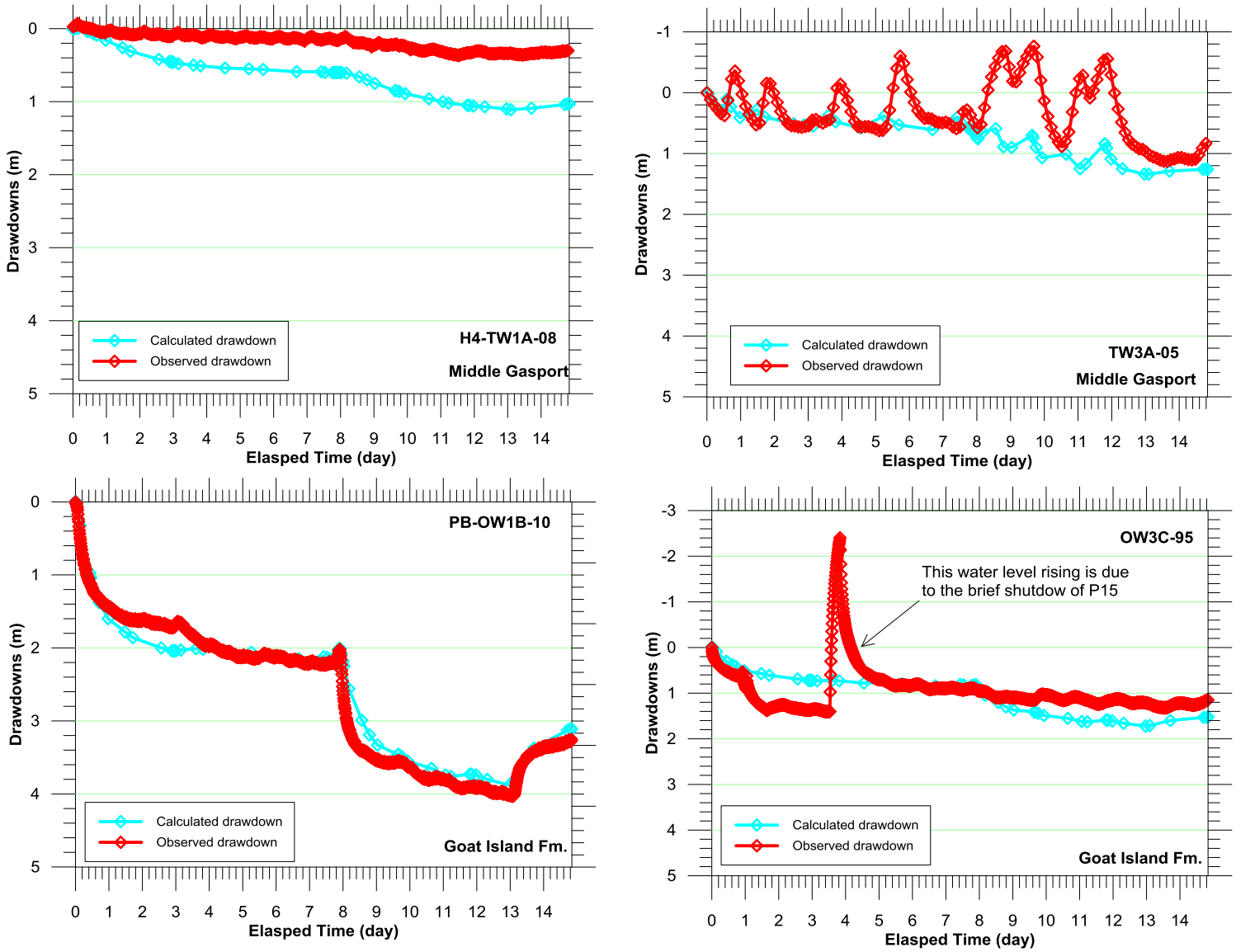
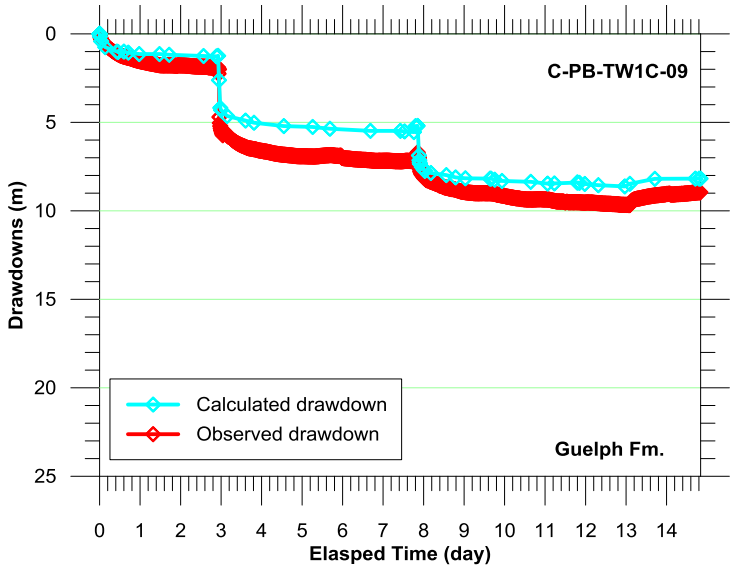
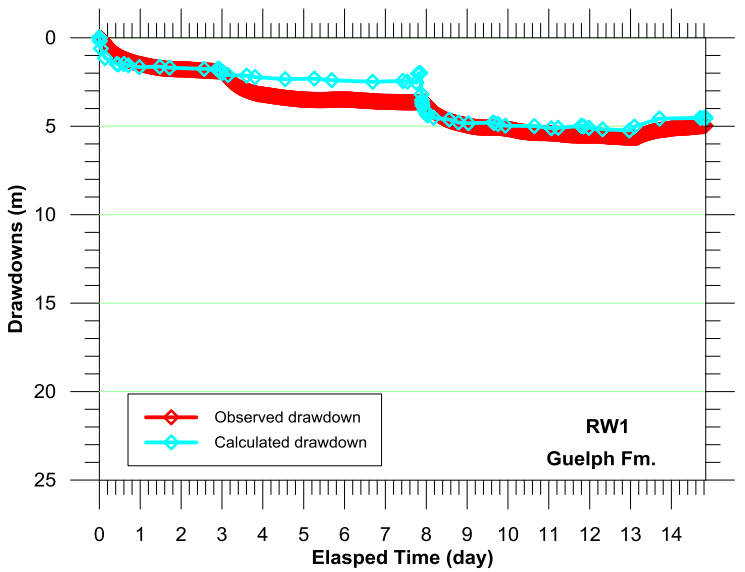


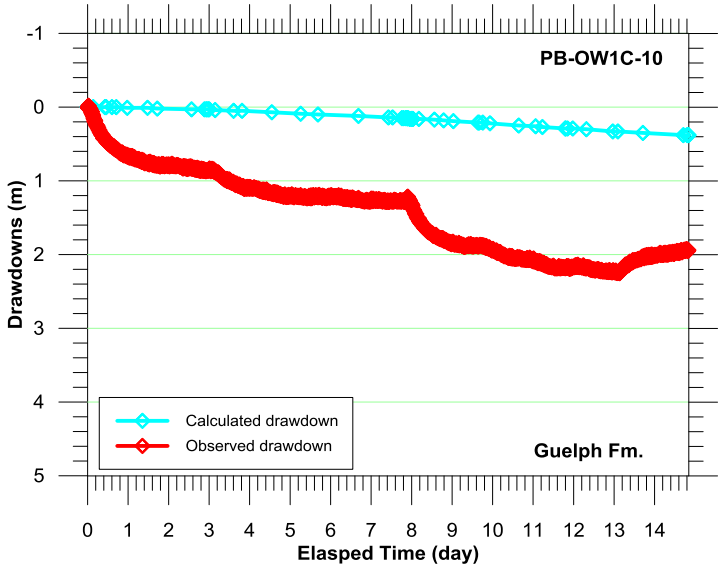
CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

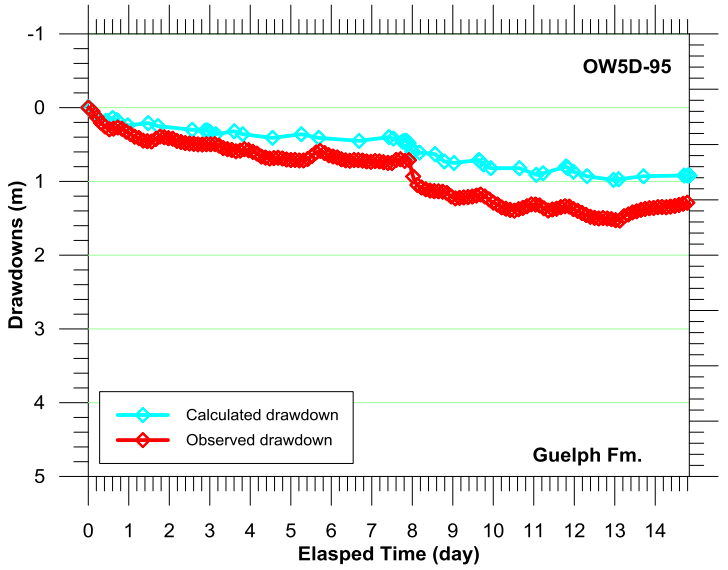
Figure 26c Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Deep Bedrock (H4-TW1A-08, TW3A-05, PB-OW1B-10, OW3C-95)

Matrix Solutions Inc.





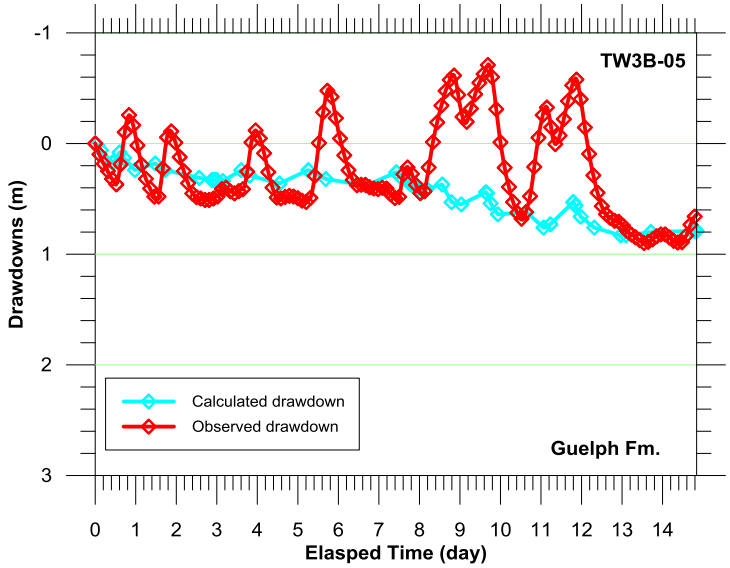


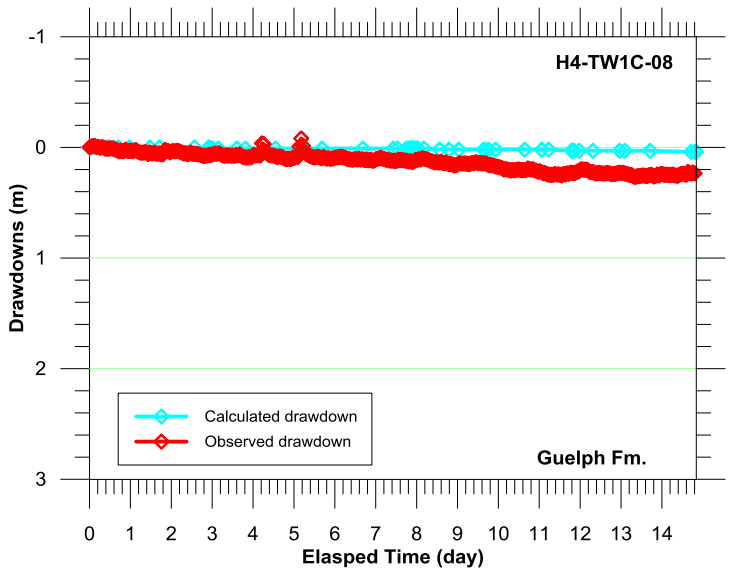


CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 27a Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Upper Bedrock (C-PB-TW1C-09,RW1, PB-OW1C-10, OW5D-95)

Matrix Solutions Inc.





CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 27b Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Upper Bedrock (TW3B-05 and H4-TW1C-08)

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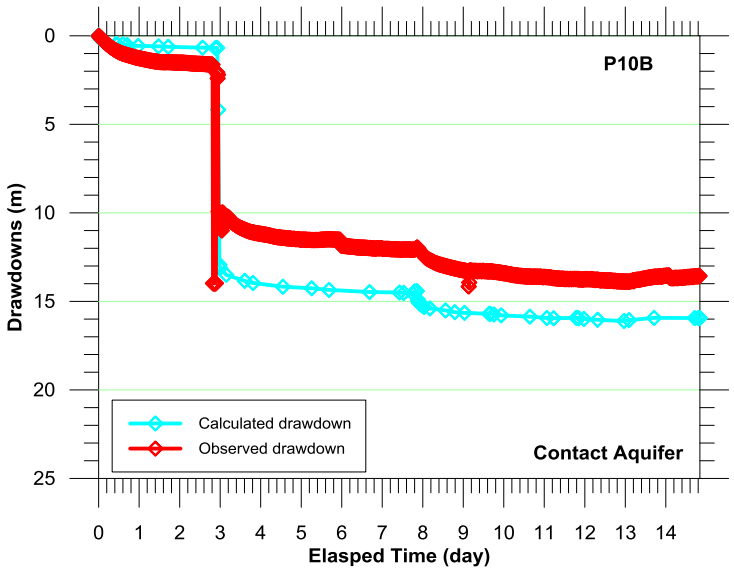

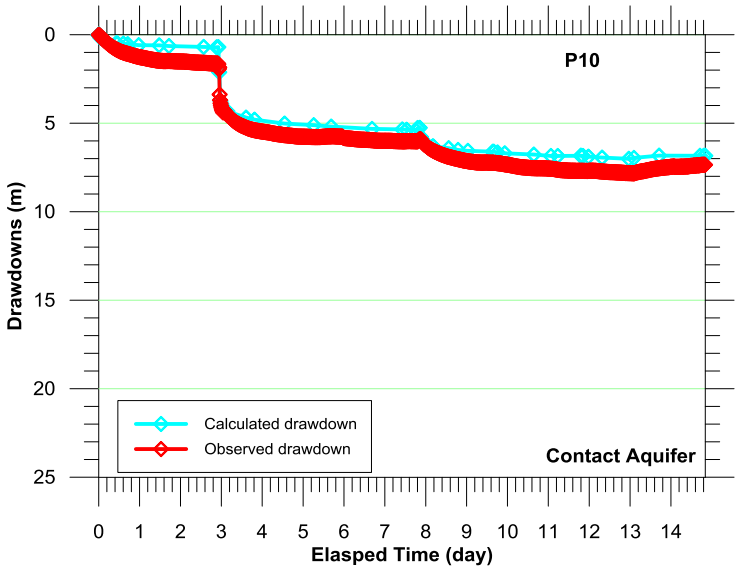

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 28 Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Contact Aquifer (P10B and P10)

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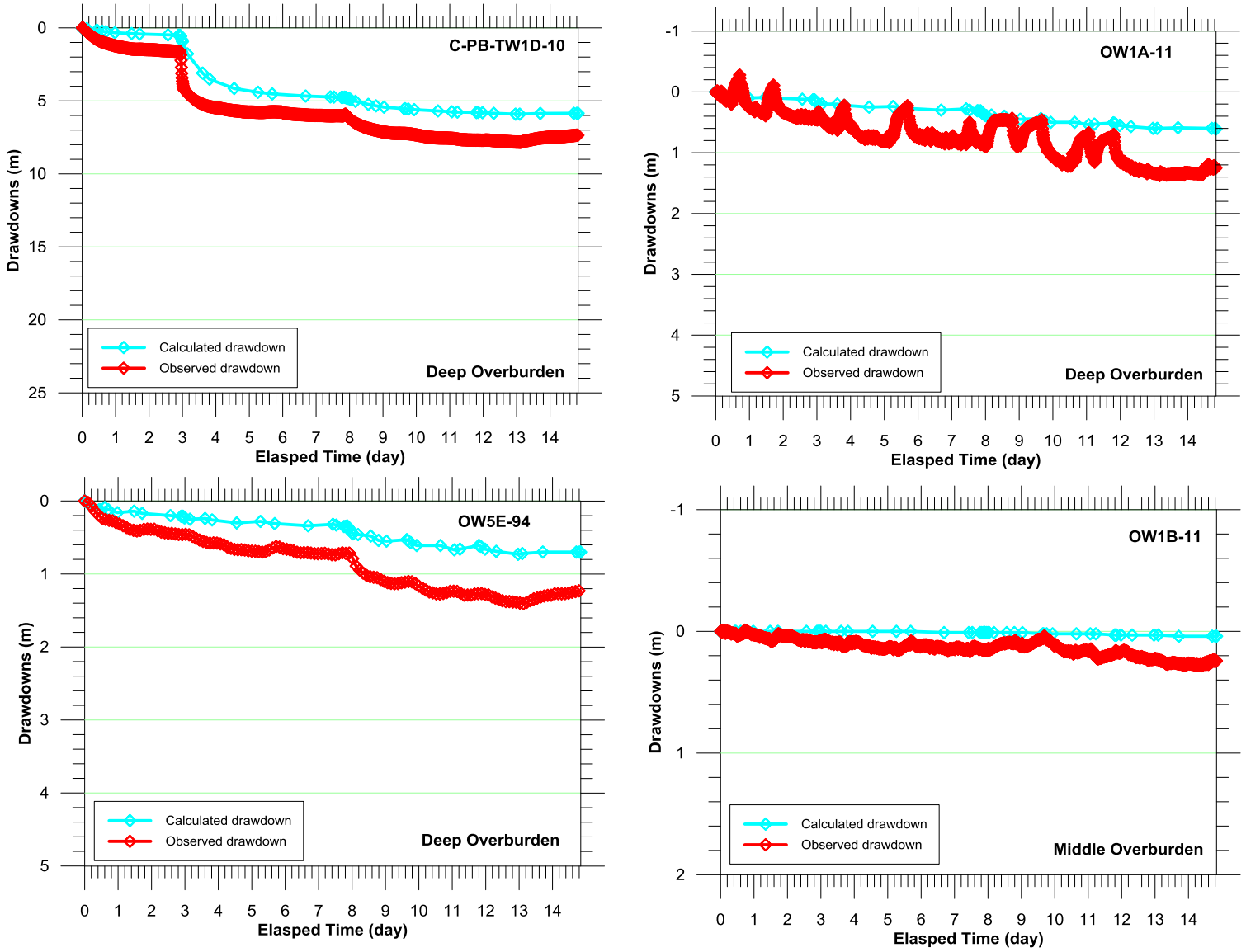

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 29 Pinebush 15-Day Pumping Test, Simulated vs. Observed Drawdowns at Selected Monitoring Wells in the Overburden (C-PB-TW1D-10, OW1A,B-11, OW5E-94)

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

555000 560000 4805000

Maple Grove Road
Fisher Mills Road
Queen Street West
Hespeler Road
Townline Road
Wellington 32 Road
Franklin Boulevard
County Road 32
Highway 401
Pinebush Road
Eagle Street North
Hespeler Road Hespeler Road
Can Amera Parkway
Bishop Street North
Coronation Boulevard
Avenue Road

H4A
H4
H4-TW1C-08
(0.3 m)
H3
H3A
C-PB-OW1C-10
(1.96 m)
Puslinch Lake
Tank Well (TW1-10)
P10B
P10
(13.54 m)
(7.33 m)
C-PB-TW1C-09
(8.94 m)
3.5 m
2.5 m
2 m
1.5 m
1 m
0.5 m
C-CA-OW3F-95
(0.39 m)
C-CA-OW3D-95
(0.61 m)
G5
G5A
P9
P15
C-CM-OW5D-94
(1.29 m)
Witmer Park
P17
P11
P17
P11
Can-Amera
C-CM-OW1C-06
(0.32 m)
C-CM-OW1B-06
(0.65 m)
C-CM-TW3C-05
(0.15 m)
C-CM-TW3B-05
(0.69 m)
Portuguese Club
G6
G16
Cedarbrook
G17
G18
P6

LEGEND

- Monitoring Well (Observed Final Drawdown)
- Pinebush 15-Day Test Pumping Wells
- Municipal Supply Well
- Test Production Well
- Simulated Drawdown in the Contact Aquifer (0.5 m Interval)
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:25,000

0.5 0.25 0 0.5

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

Matrix Solutions Inc. ENVIRONMENT & ENGINEERING

Figure 30

Simulated Drawdown Contours at the End of Pinebush 15-Day Pumping Test (Contact Aquifer)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

555000

560000

4805000

Maple Grove Road
Fisher Mills Road
Townline Road
Wellington 32 Road
Queen Street West
Hespeler Road
Franklin Boulevard
Highway 401
County Road 32
Pinebush Road
Eagle Street North
Bishop Street North
Can Amera Parkway
Coronation Boulevard
Avenue Road
Puslinch Lake

H4-TW1A-08 (0.24 m)
H4-TW1B-08 (0.3 m)
H4
H4A
H3
H3A
C-PB-OW1A-10 (3.12 m)
TW1-10 (11.83 m)
Tank Well (TW1-10)
C-PB-TW1A-09-P10A (20.44 m-(38.31 m)
P10A
C-PB-TW1B-09 (18.85 m)
C-CM-OW5B-94 (1.46 m)
G5
G5A
P15
P9
C-PB-TW2-06 (1.46 m)
Witmer Park
P17 (10.47 m)
P11
P11A
C-PB-TW3-07 (1.19 m)
Can-Amera
Portuguese Club
C-CM-OW1A-06 (0 m)
C-CM-TW3A-05 (0.87 m)
G6
G16
G17
G18
Cedarbrook
P6

1
2
3
4
5
6
7
8
9
11

LEGEND

- Monitoring well (Observed Final drawdown)
- Pinebush 15-Day Test Pumping Wells
- Municipal Supply Well
- Test Production Well
- Simulated Drawdown in the Gasport Formation (1 m Interval)
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:25,000

0.5 0.25 0 0.5

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 31

Simulated Drawdown Contours at the End of Pinebush 15-Day Pumping Test (Gasport Formation)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Well Locations
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:40,000

1 0.5 0 1

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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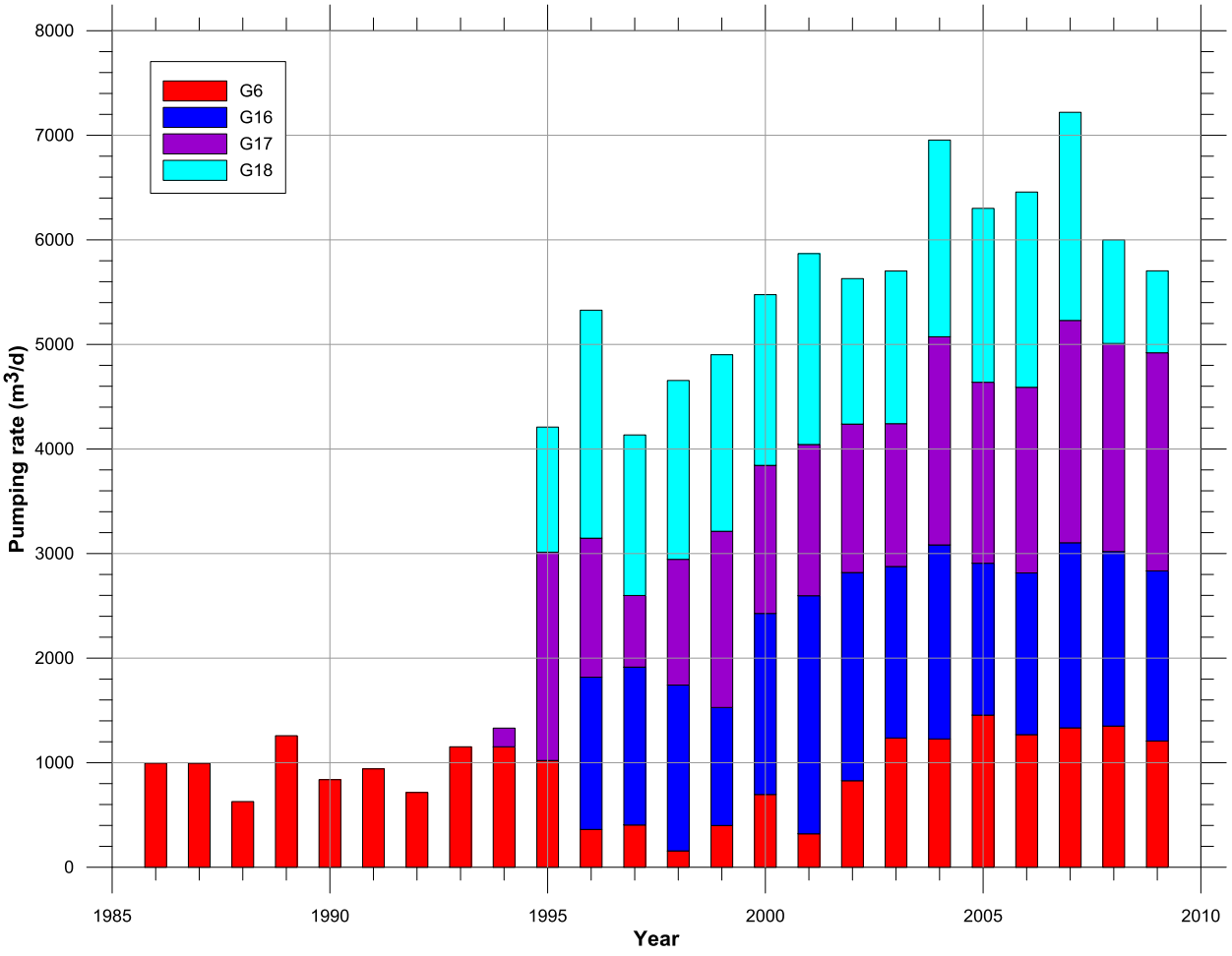
Figure 32

Well Location for Assessing Drawdowns due to the Historical (Mid-1990s) Increase in Municipal Pumping



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 33 Historical Annual Pumping Rates at the Clemens Mill Well Field

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Well Locations
- Municipal Supply Well
- Test Production Well
- Simulated Drawdown in the Contact Aquifer (1 m interval)
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:40,000

1 0.5 0 1

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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Figure 34a

Simulated Drawdown in the Contact Aquifer Due to the Historical (Mid-1990s) Increase in Municipal Pumping

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Well Locations
- Municipal Supply Well
- Test Production Well
- Simulated Drawdown in the Middle Gasport Fm. (1 m interval)
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:40,000

1 0.5 0 1

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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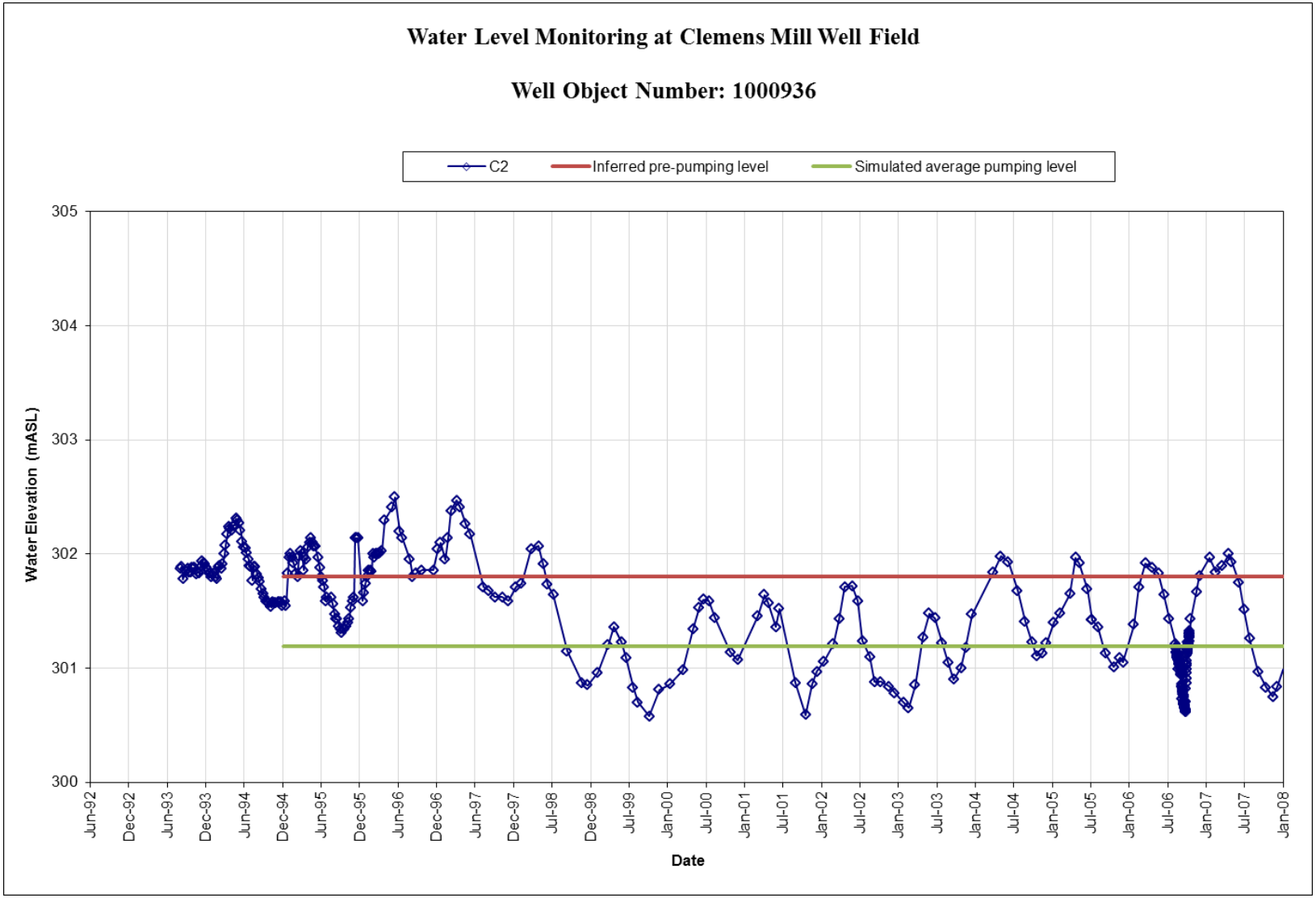
Figure 34b

Simulated Drawdown in the Middle Gasport Fm. Due to the Historical (Mid-1990s) Increase in Municipal Pumping



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 35 Comparison of Observed Water Levels at C2 with the Simulated Water Level Decline Due to Increase in Pumping

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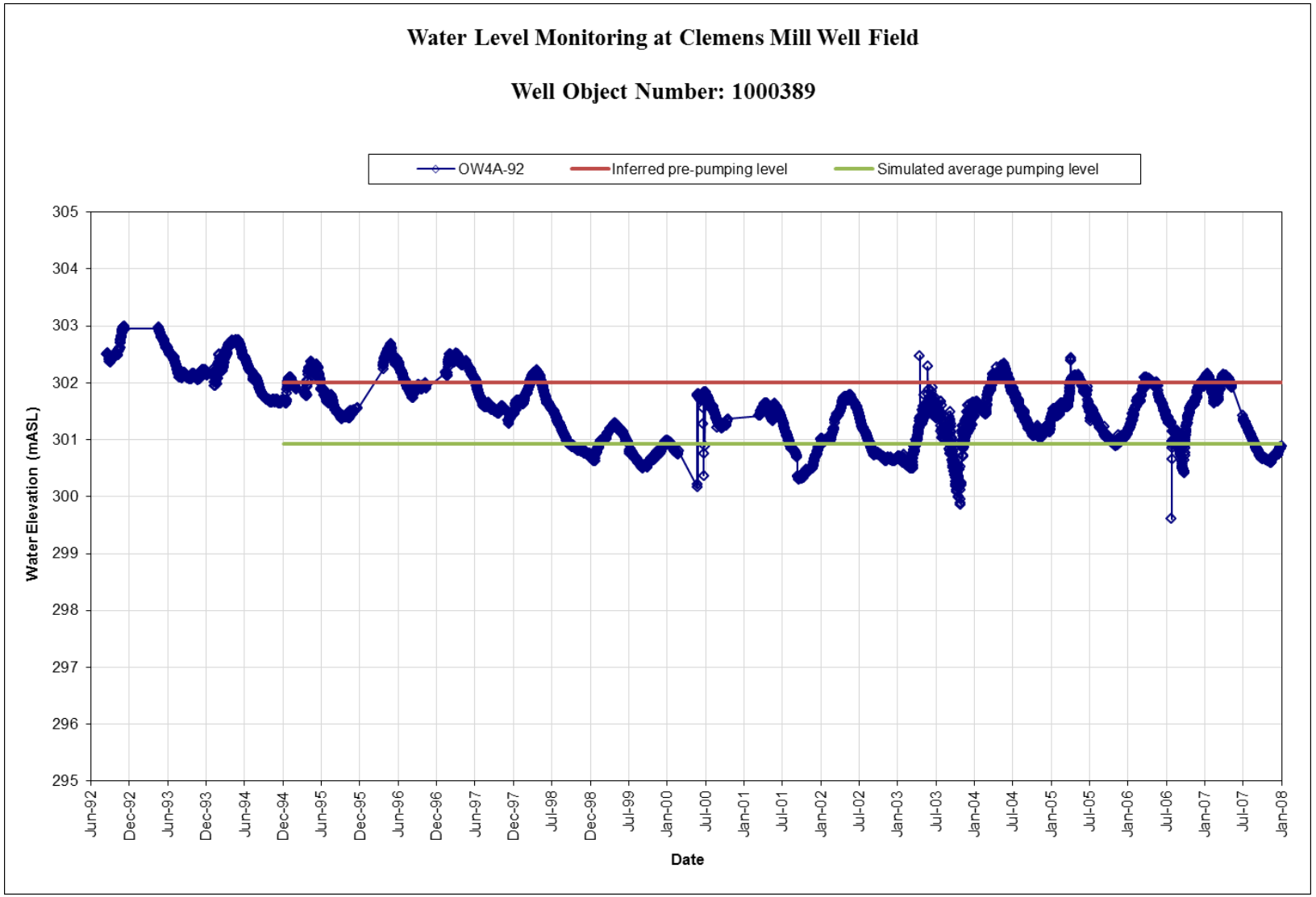

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 36 Comparison of Observed Water Levels at OW4A-92 with the Simulated Water Level Decline Due to Increase in Pumping

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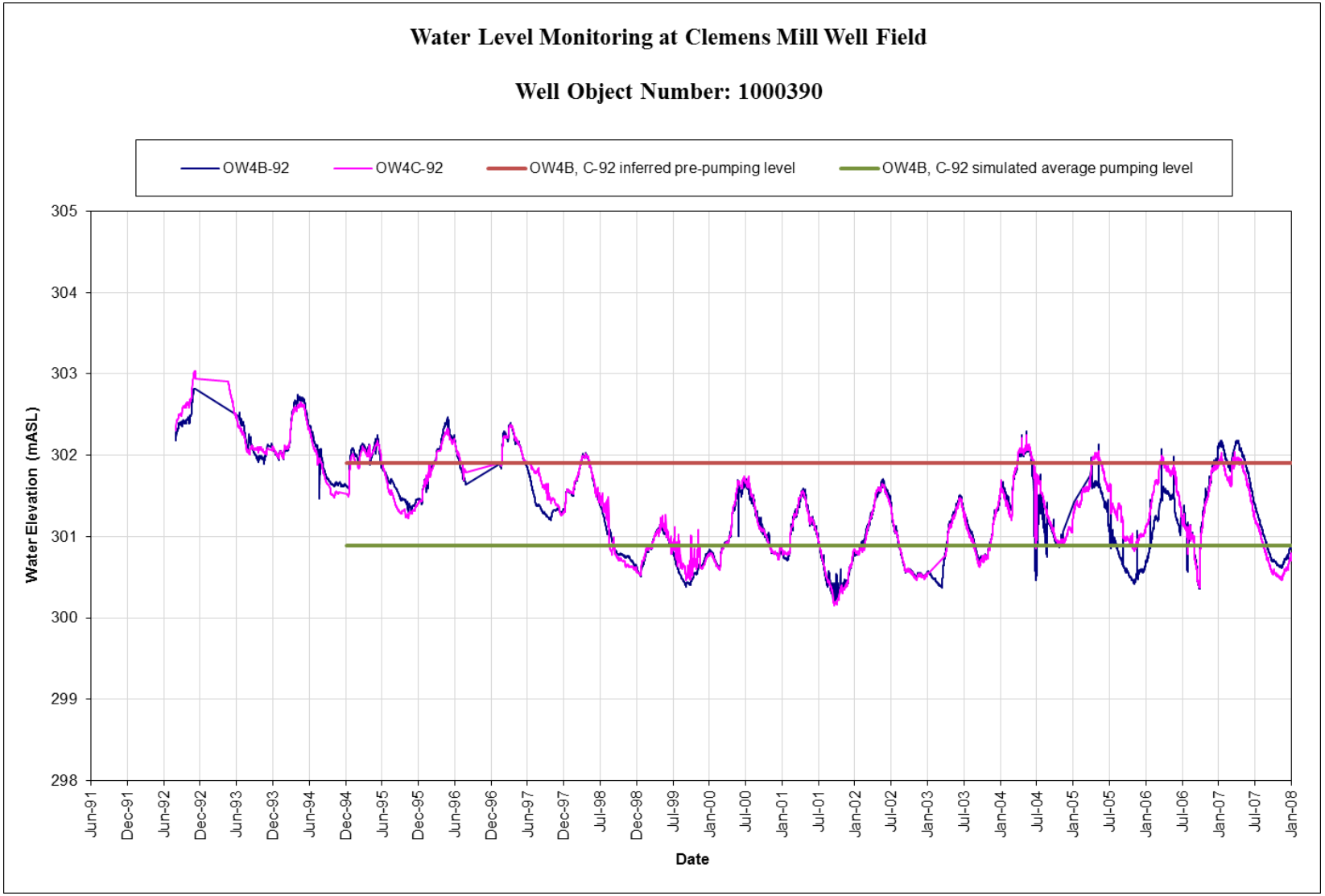

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 37 Comparison of Observed Water Levels at OW4B,C-94 with the Simulated Water Level Declines Due to Increase in Pumping

Matrix Solutions Inc.
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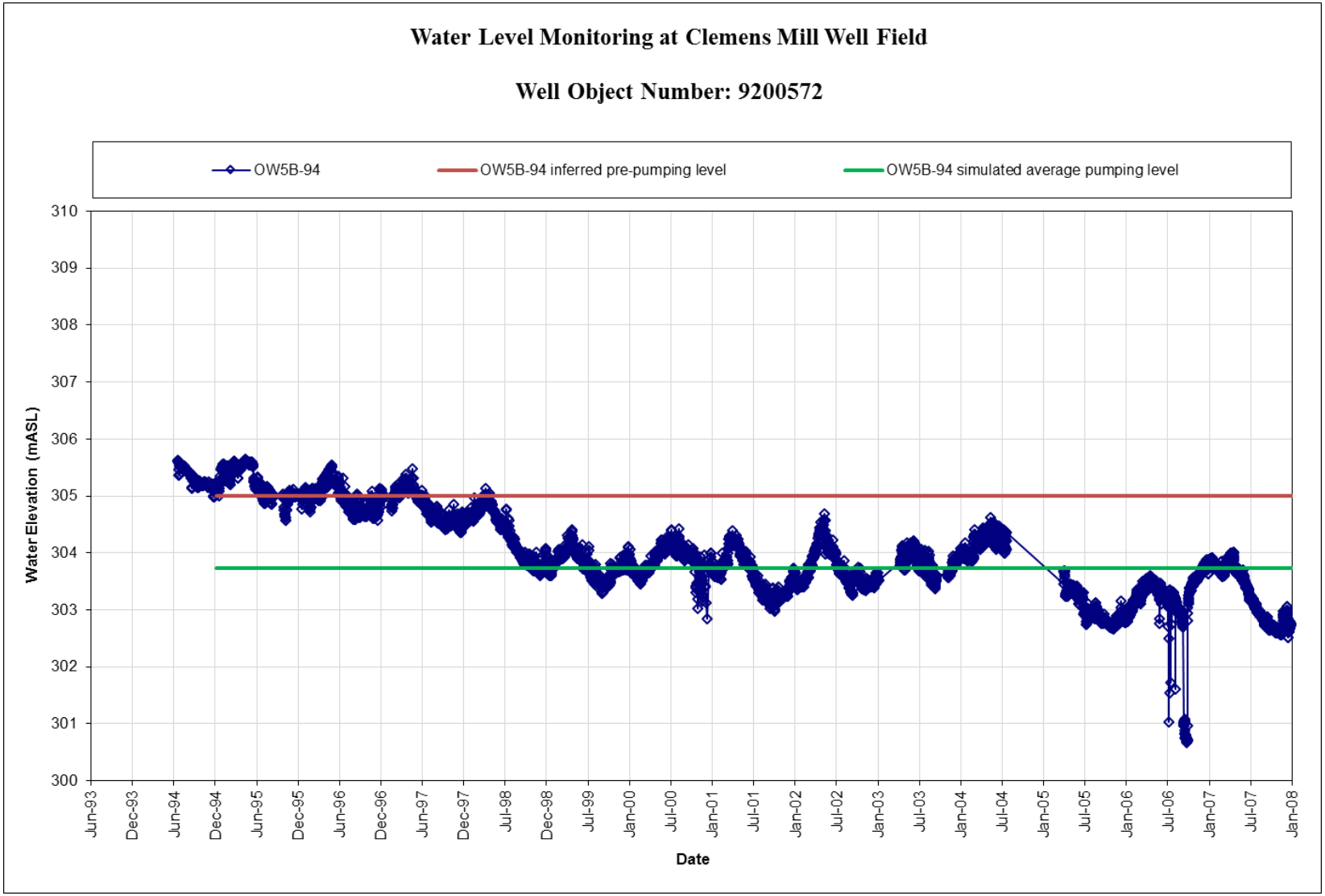

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 38 Comparison of Observed Water Levels at OW5B-94 with the Simulated Water Level Decline Due to Increase in Pumping

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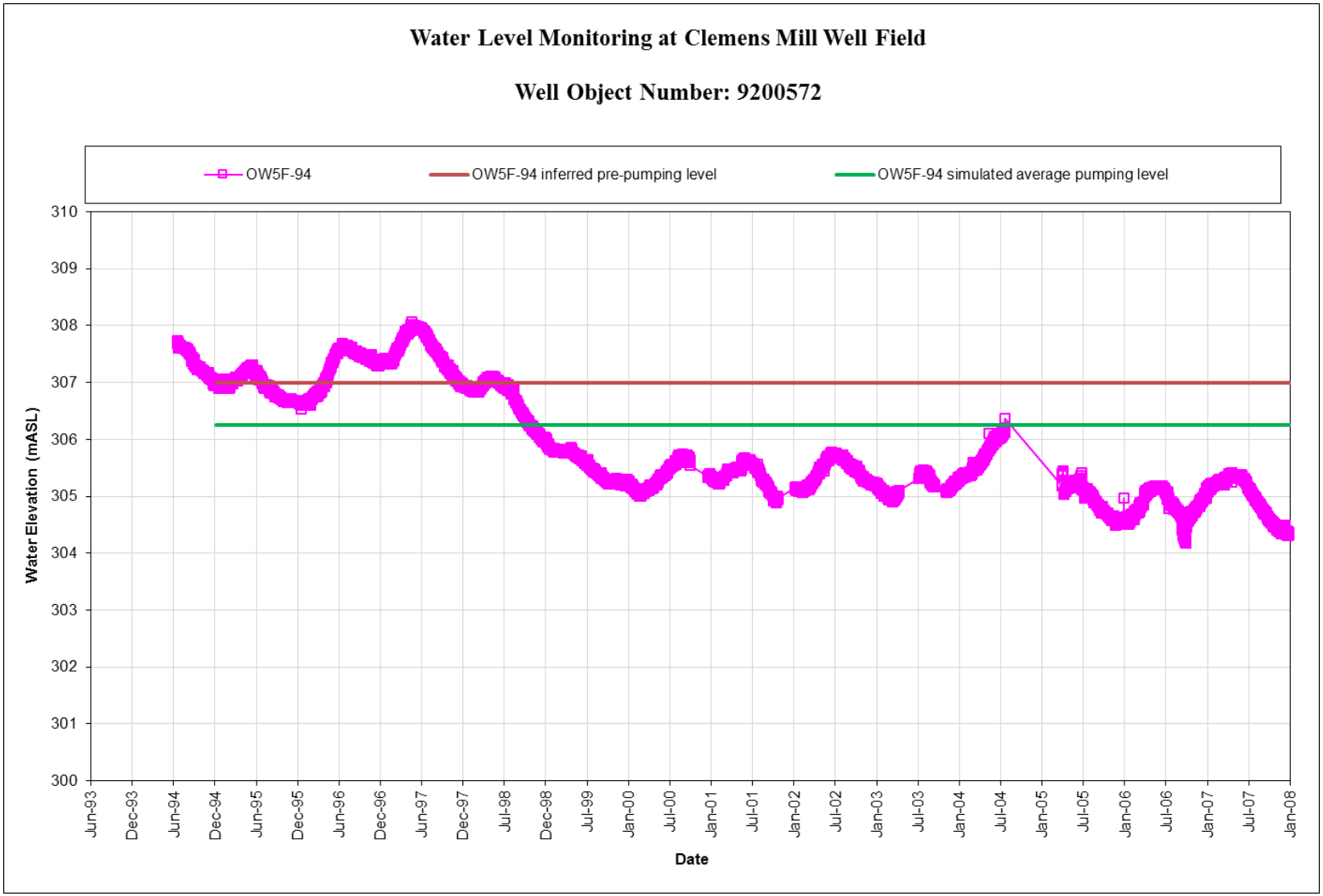

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 39 Comparison of Observed Water Levels at OW5F-94 with the Simulated Water Level Decline Due to Increase in Pumping

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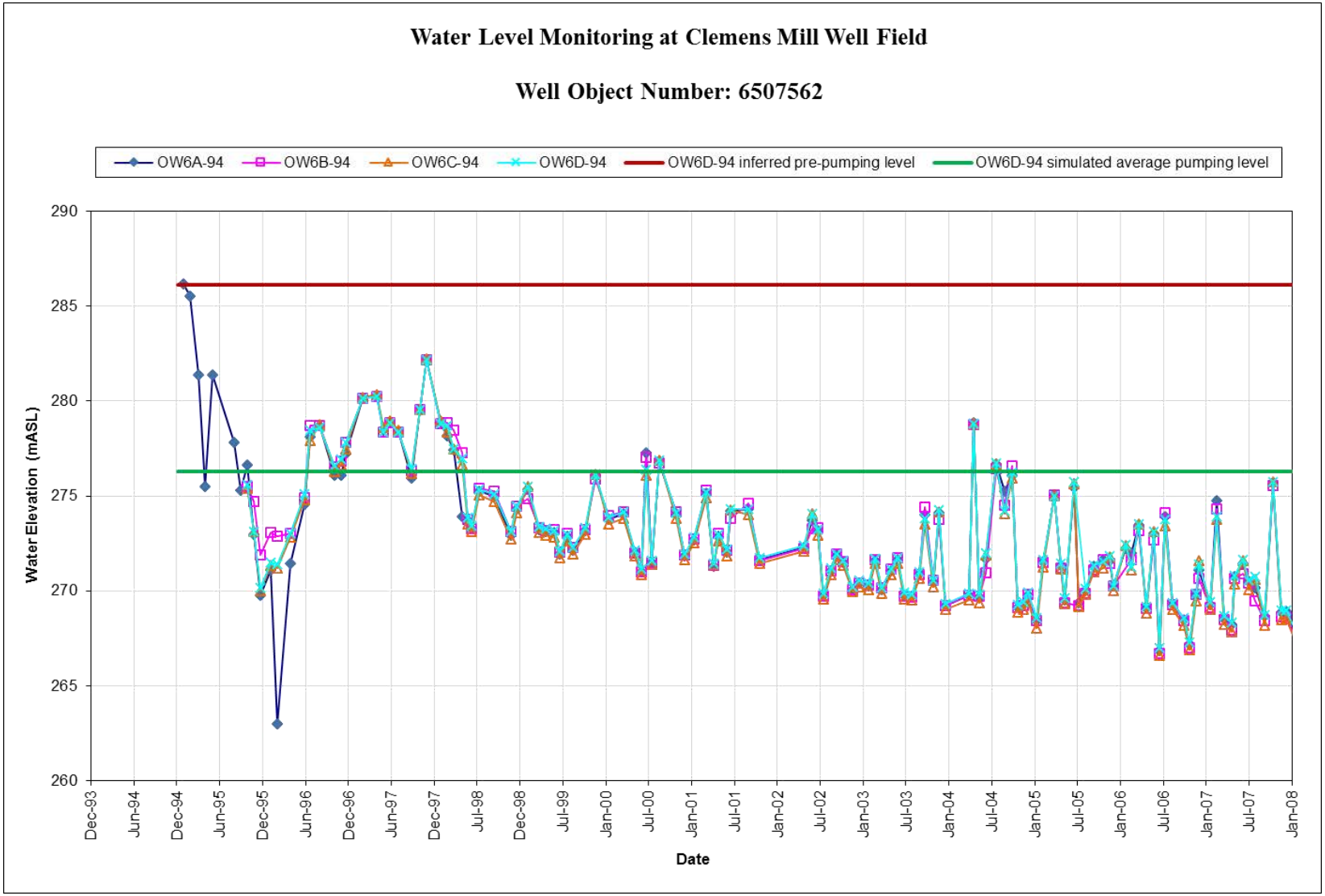

CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 40 Comparison of Observed Water Levels at OW6A, B, C, D-94 with the Simulated Water Level Declines Due to Increase in Pumping

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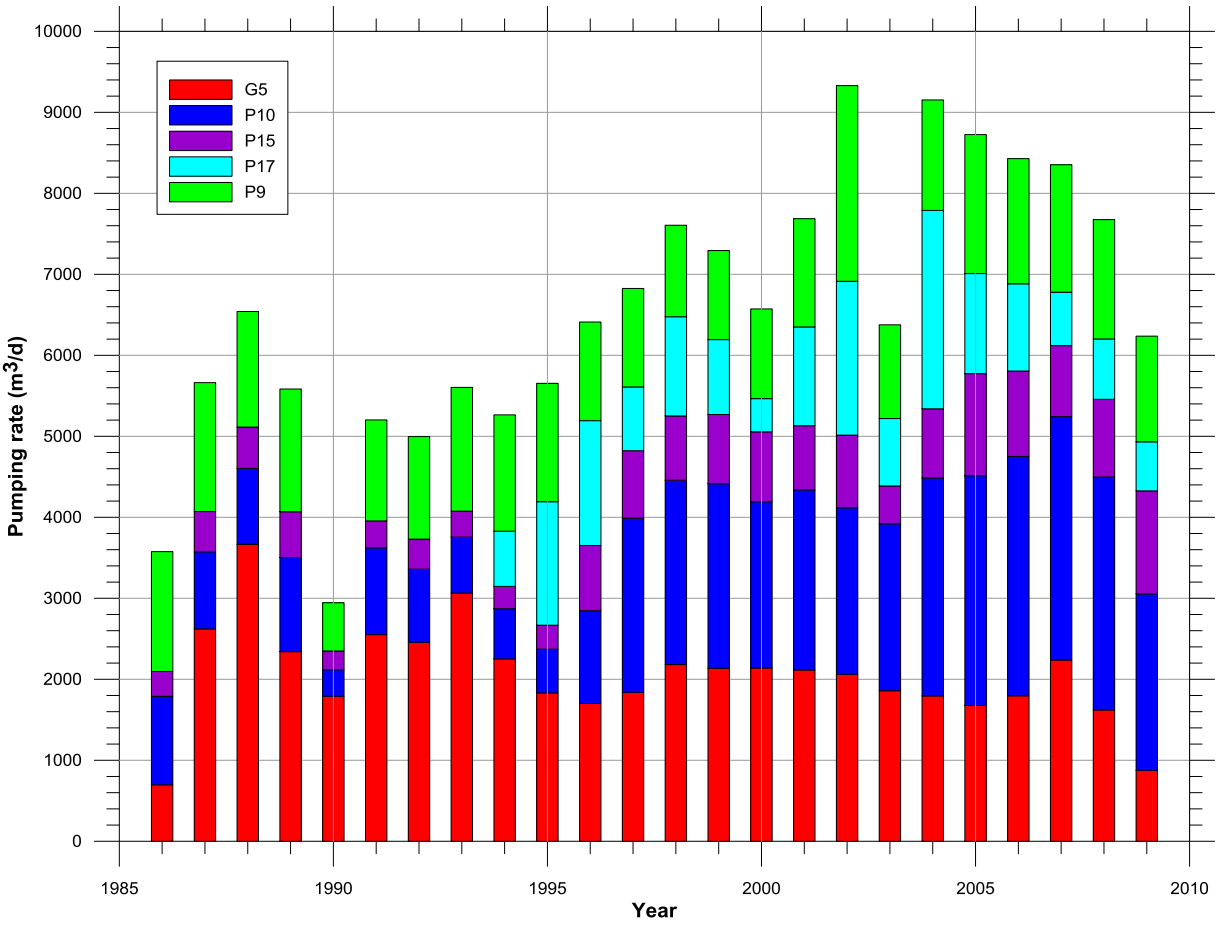

CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 41 Historical Annual Pumping Rates at the Pinebush Well Field

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- G5A Test Well
- Monitoring Well (Observed Drawdown)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

1:13,573.69

0.3 0.15 0 0.3

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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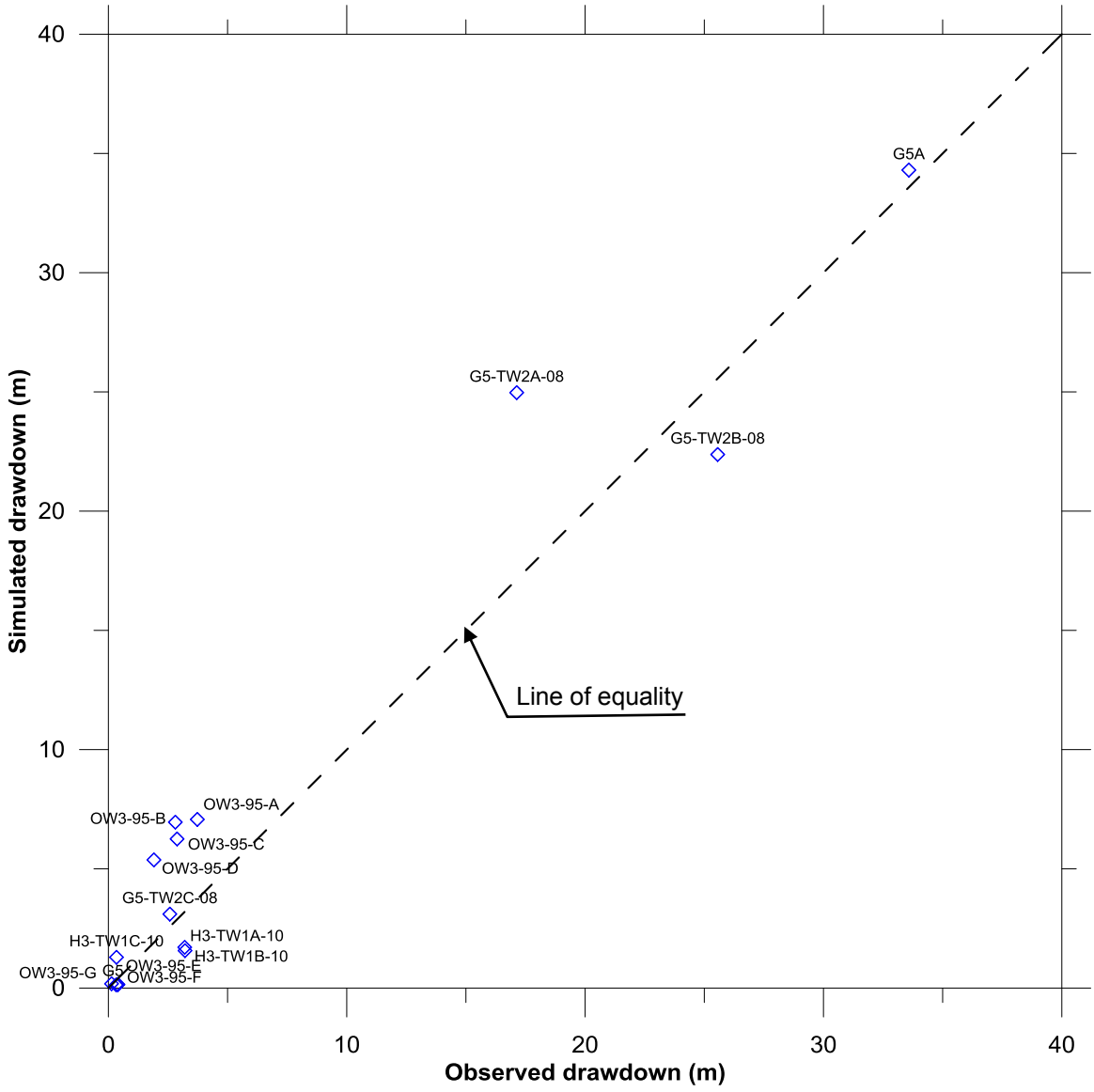
Figure 42

Locations of Pumping Well and Observation Wells of G5A Pumping Test



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 43 Scatterplot of Drawdowns at the End of the G5A 6-day Pumping Test

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain
- Polygon for Water Balance Calculation

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 2, Map date 2014-05-27 J. Zhang

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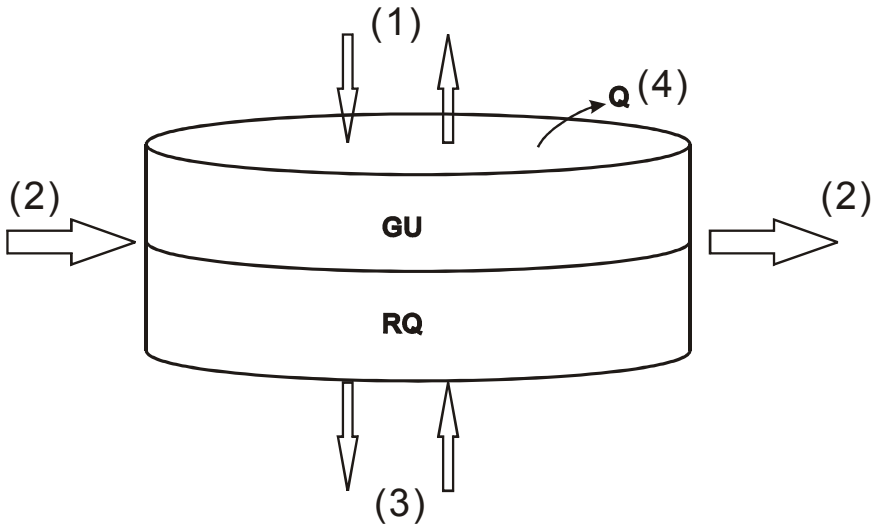
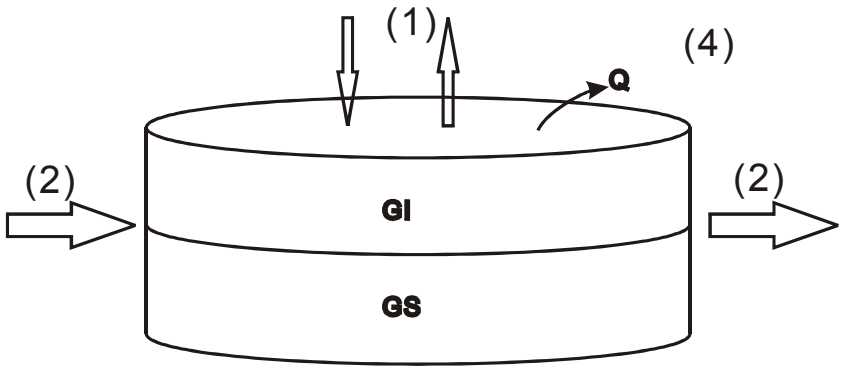
Figure 44

Zone of Influence (ZOI) for Water Balance Calculation

(1) Vertical flow across the top slice
(2) Horizontal flow
(3) Vertical flow across the bottom slice
(4) Groundwater withdrawals by pumping



(a) Water balance components for the upper bedrock aquifer
(GU = Guelph Formation; RQ = Reformatory Quarry Member of Eramosa Formation



(b) Water balance components for the lower bedrock aquifer
(GI = Goat Island Formation; GS = Gasport Formation)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Municipal Supply Well
- Test Production Well
- Expressway / Highway

Major Roads

- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds

Subwatershed

- Fairchild Creek
- Halton Region
- Irish Creek
- Lower Mill Creek
- Lower Speed River
- Middle Grand River
- Moffat Creek
- Upper Mill Creek

1:71,635.31

2.5 1.25 0 2.5

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 46

Delineation of Subwatersheds for Water Balance Calculation

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Water Level Elevation (5 m interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 47

Simulated Water Level Elevations (Contact Aquifer) under the Base Case Condition

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Water Level Elevation (5 m interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 48

Simulated Water Level Elevations (MIddle Gasport) under the Base Case Condition

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 49

Simulated Steady-State Drawdown in the Contact Aquifer
(Scenario 1 vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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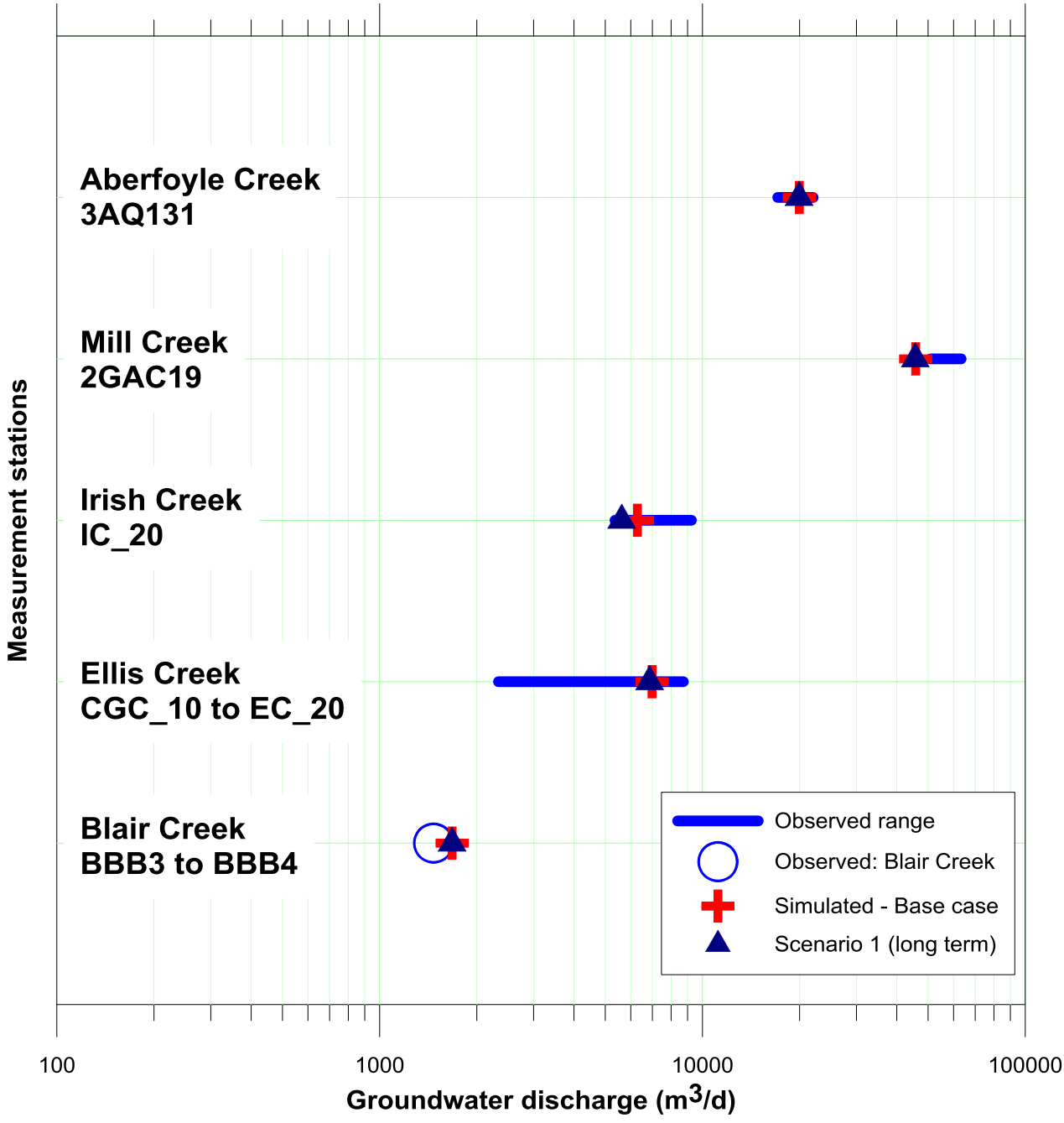
Figure 50

Simulated Steady-State Drawdown in the Middle Gasport Formation (Scenario 1 vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 51 Simulated Steady-State Changes in Groundwater Discharge to Streams (Scenario 1 vs. the Base Case)

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 52

Simulated Steady-State Drawdown in the Contact Aquifer
(Scenario 2 vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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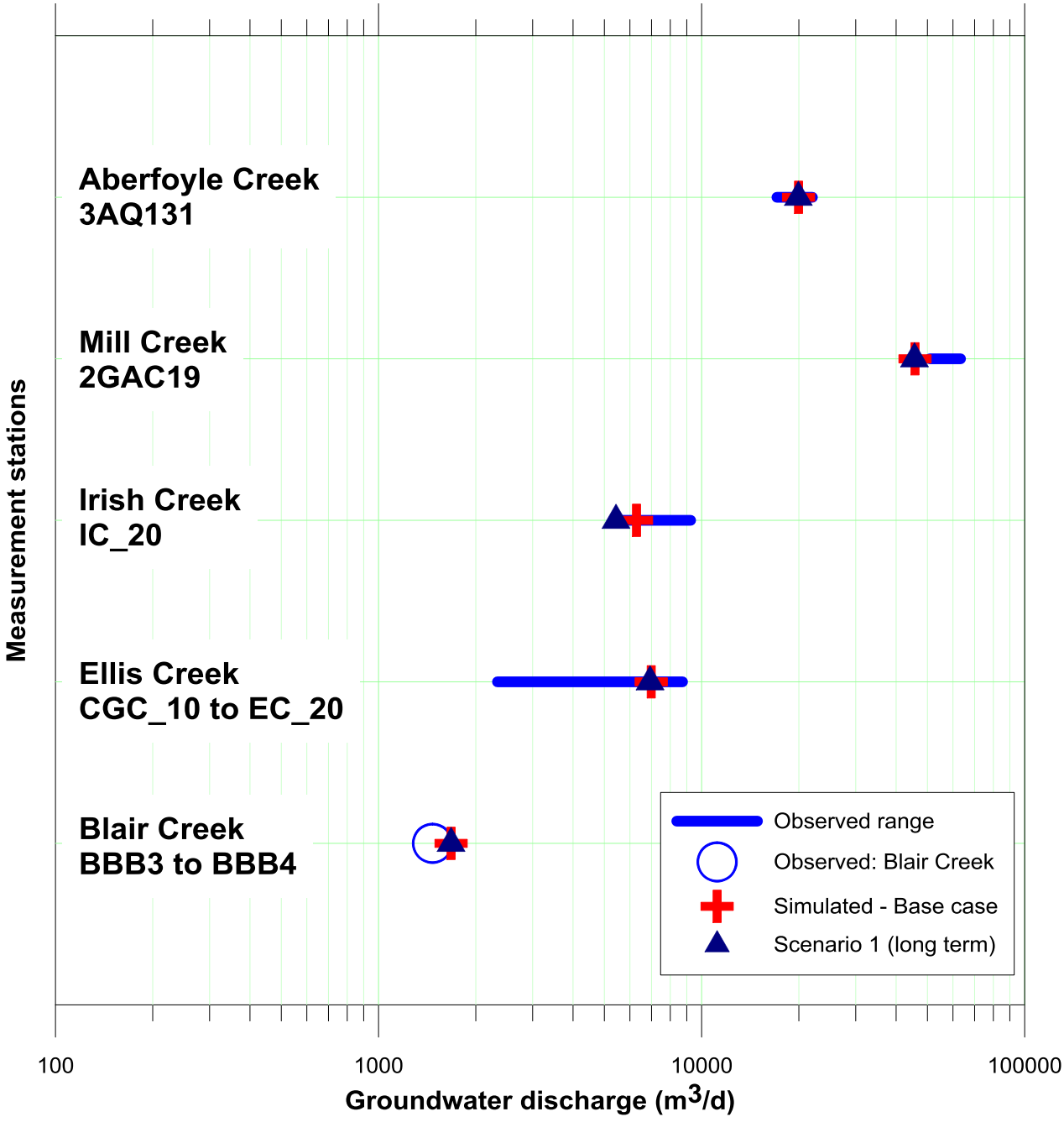
Figure 53

Simulated Steady-State Drawdown in the Middle Gasport Formation (Scenario 2 vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 54 Simulated Steady-State Changes in Groundwater Discharge to Streams (Scenario 2 vs. the Base Case)

Σ²Π

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

Matrix Solutions Inc.
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Figure 55

Simulated Steady-State Drawdown in the Contact Aquifer (Scenario 3 vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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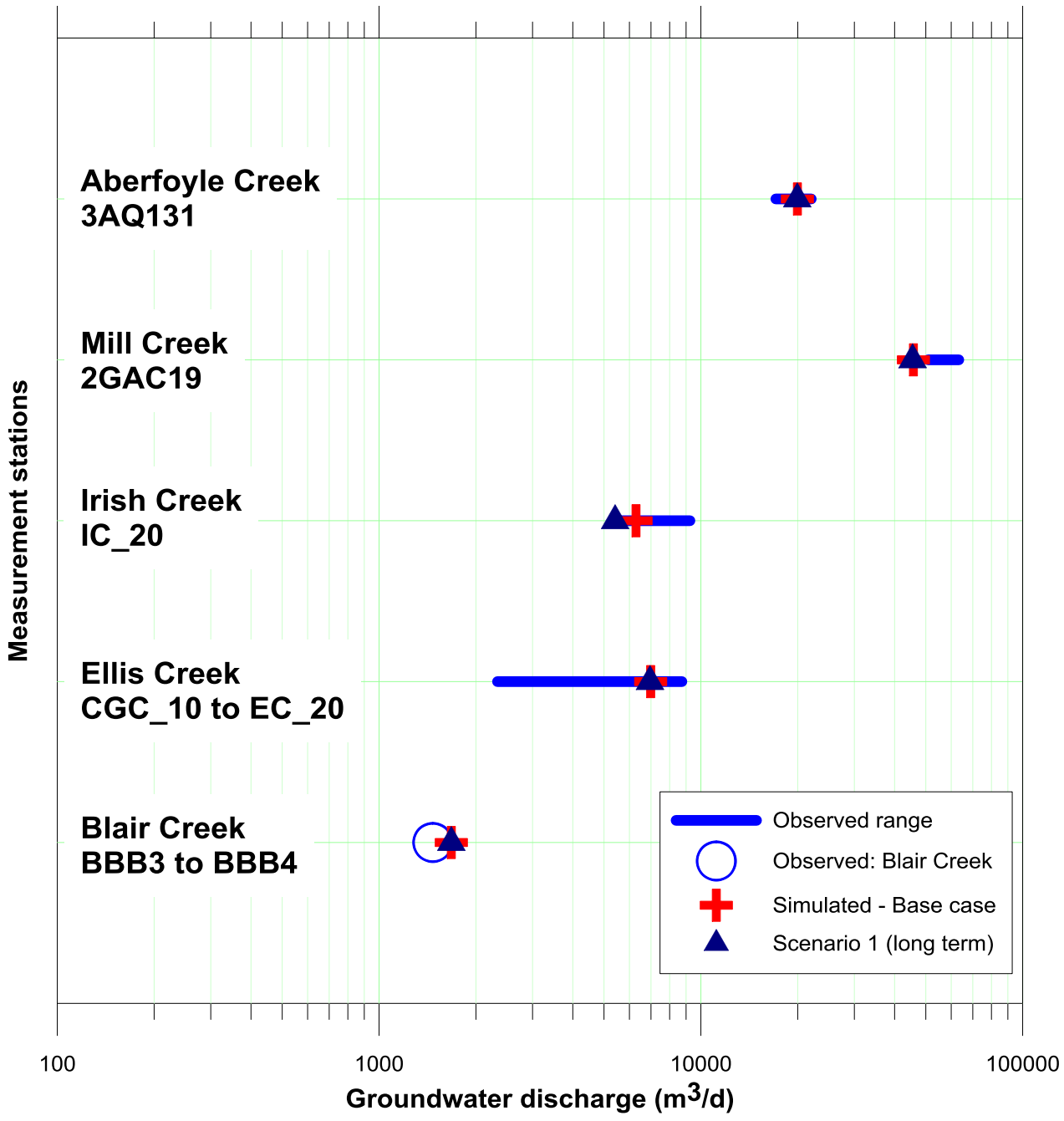
Figure 56

Simulated Steady-State Drawdown in the Middle Gasport Formation (Scenario 3 vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 57 Simulated Steady-State Changes in Groundwater Discharge to Streams (Scenario 3 vs. the Base Case)

Σ²Π

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 58

Simulated Drawdowns in the Contact Aquifer after 2 years (Scenario A vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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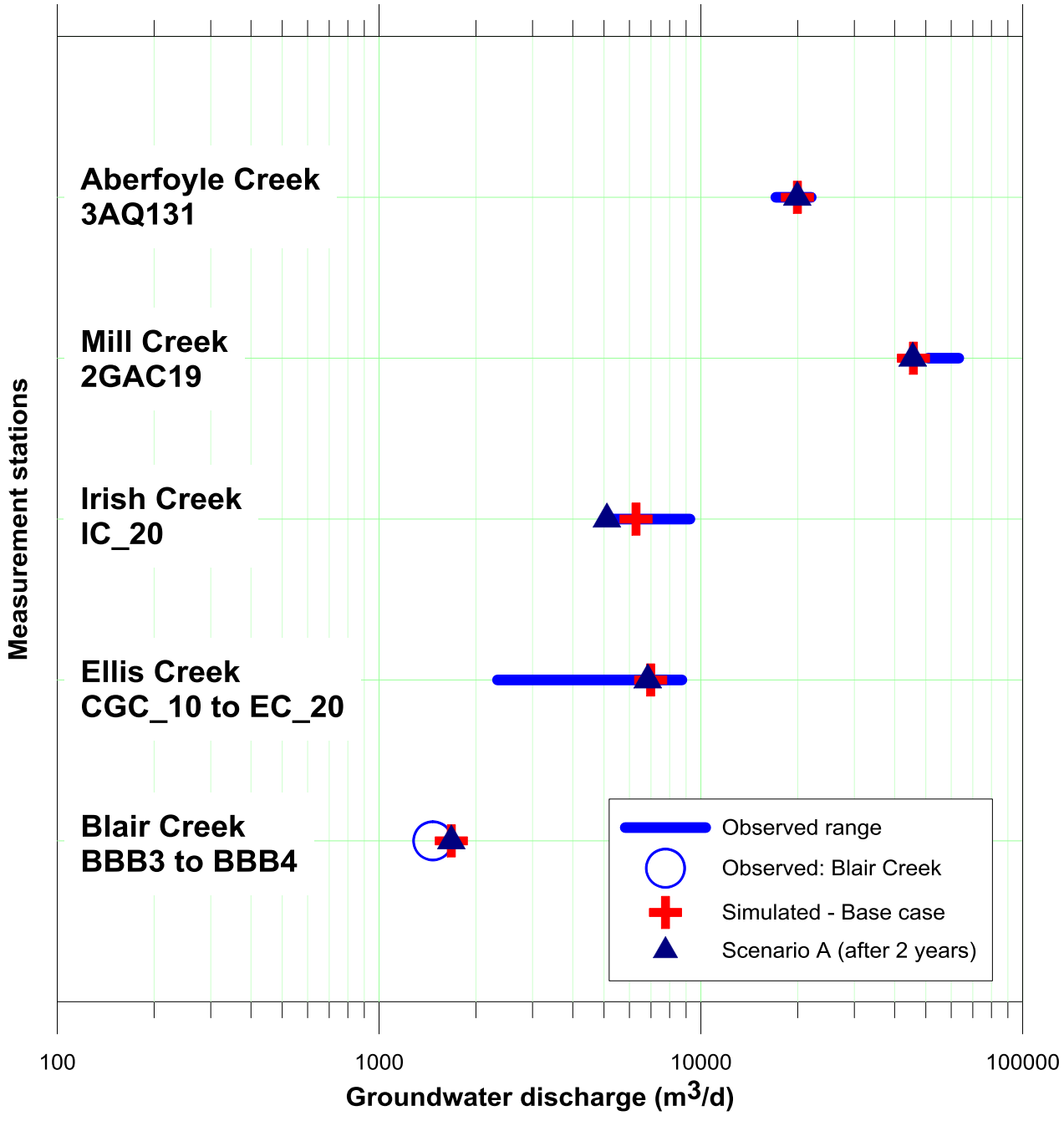
Figure 59

Simulated Drawdowns in the Middle Gasport Formation after 2 years (Scenario A vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING	**Figure 60** Simulated Transient Changes in Groundwater Discharge to Streams after 2 years (Scenario A vs. the Base Case)	Σ²Π Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

Matrix Solutions Inc.
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Figure 61

Simulated Drawdowns in the Contact Aquifer after 2 years (Scenario B vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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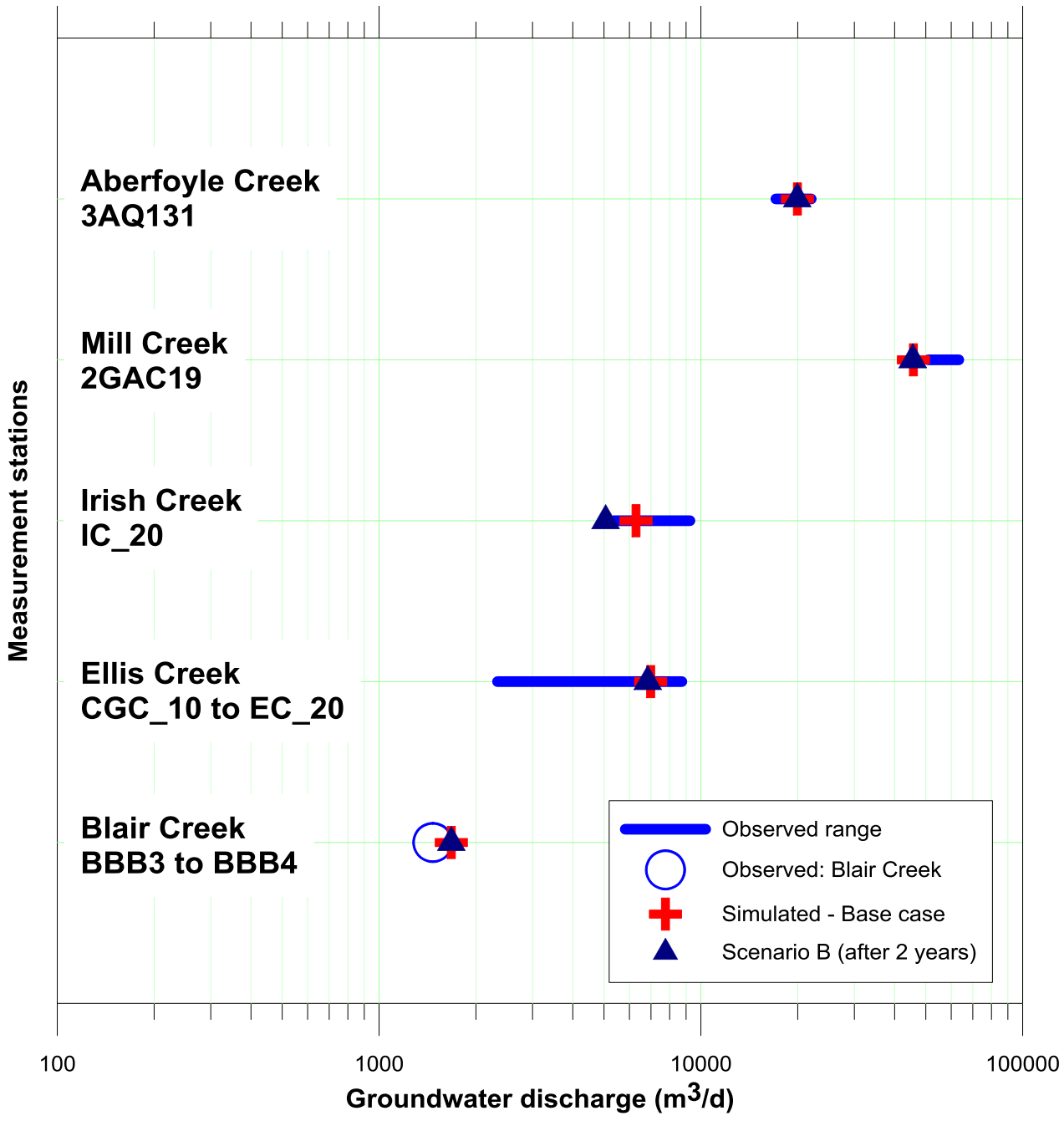
Figure 62

Simulated Drawdowns in the Middle Gasport Formation after 2 years (Scenario B vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 63 Simulated Transient Changes in Groundwater Discharge to Streams after 2 years (Scenario B vs. the Base Case)

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 64

Simulated Drawdowns in the Contact Aquifer after 2 years (Scenario C vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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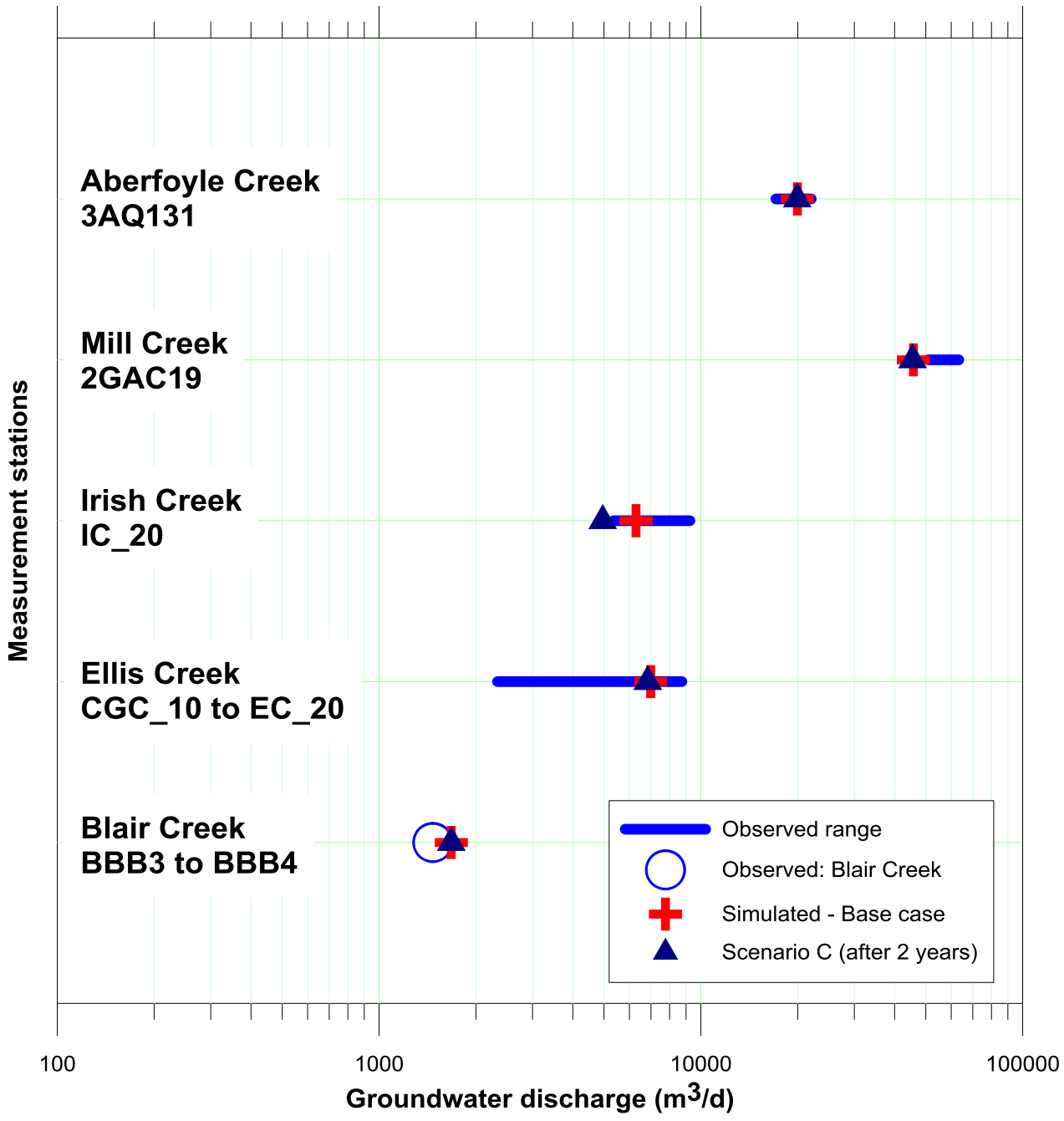
Figure 65

Simulated Drawdowns in the Middle Gasport Formation after 2 years (Scenario C vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 66 Simulated Transient Changes in Groundwater Discharge to Streams after 2 years (Scenario C vs. the Base Case)

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 67

Simulated Drawdowns in the Contact Aquifer after 2 years (Scenario D vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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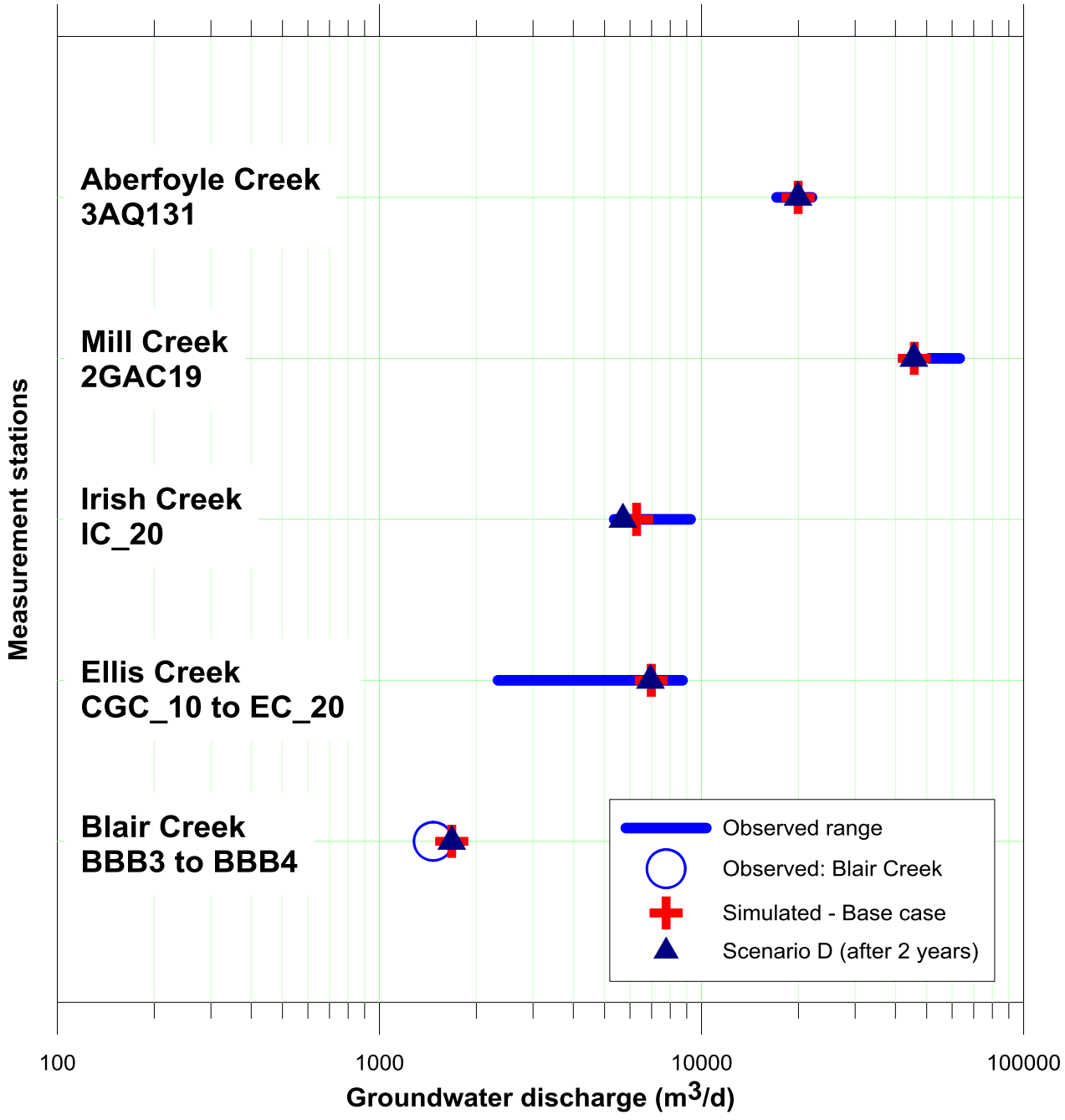
Figure 68

Simulated Drawdowns in the Middle Gasport Formation after 2 years (Scenario D vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 69 Simulated Transient Changes in Groundwater Discharge to Streams after 2 years (Scenario D vs. the Base Case)

Matrix Solutions Inc.

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 70

Simulated Drawdowns in the Contact Aquifer after 2 years (Scenario E vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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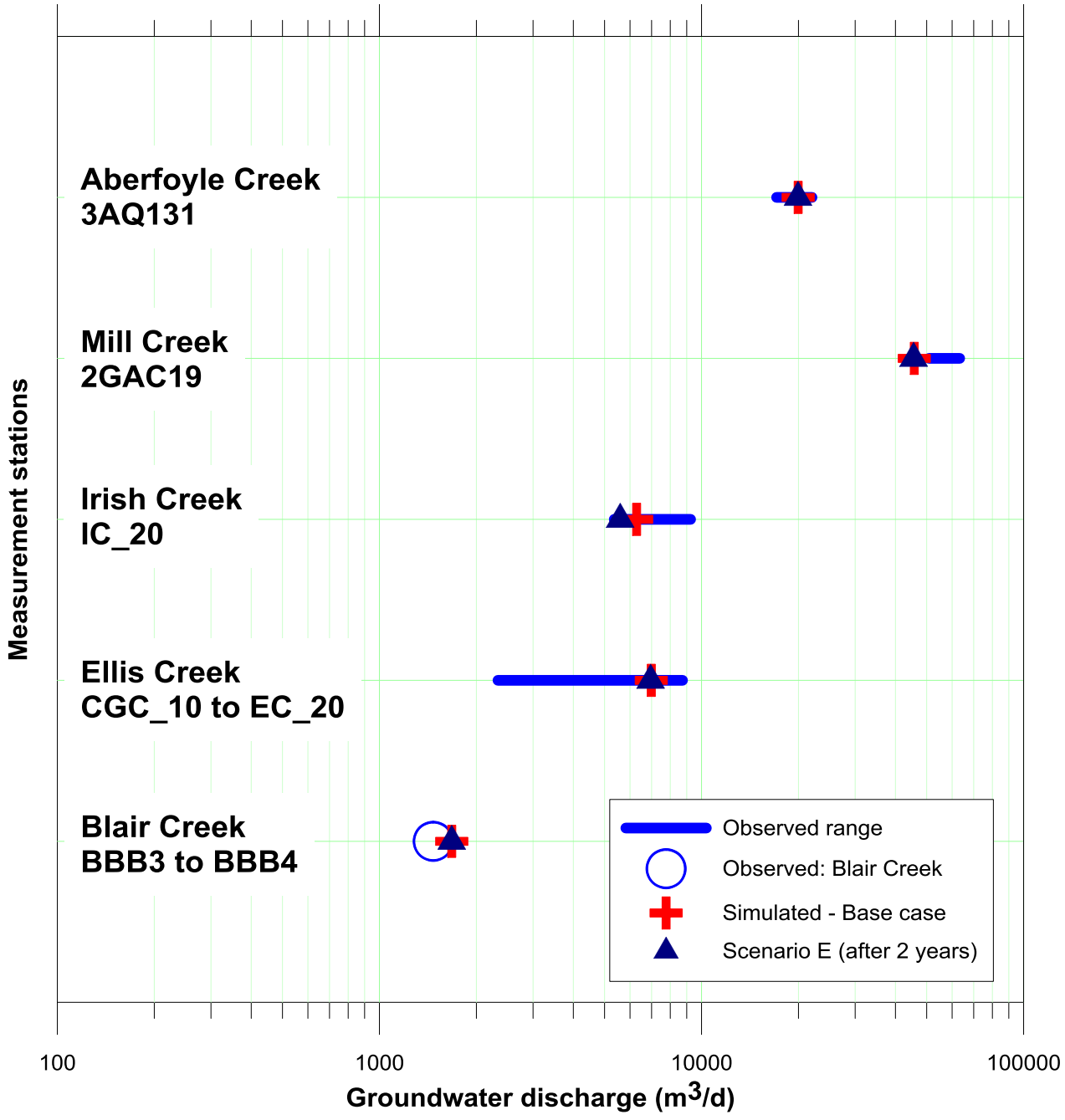
Figure 71

Simulated Drawdowns in the Middle Gasport Formation after 2 years (Scenario E vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 72 Simulated Transient Changes in Groundwater Discharge to Streams after 2 years (Scenario E vs. the Base Case)

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Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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Figure 73

Simulated Drawdowns in the Contact Aquifer after 2 years (Scenario F vs. the Base Case)

Cambridge East IUS Water Supply Class EA: Groundwater Modelling

LEGEND

- Simulated Drawdown (1 m Interval)
- Municipal Supply Well
- Test Production Well
- Expressway / Highway
- Major Roads
- Roads (collectors)
- Rivers / Streams
- Lakes and Ponds
- Model Domain

1:125,000

4 2 0 4

Kilometers

Projection: UTM Zone 17N, NAD 83
Map Version: 3, Map date 2015-09-10 J. Zhang

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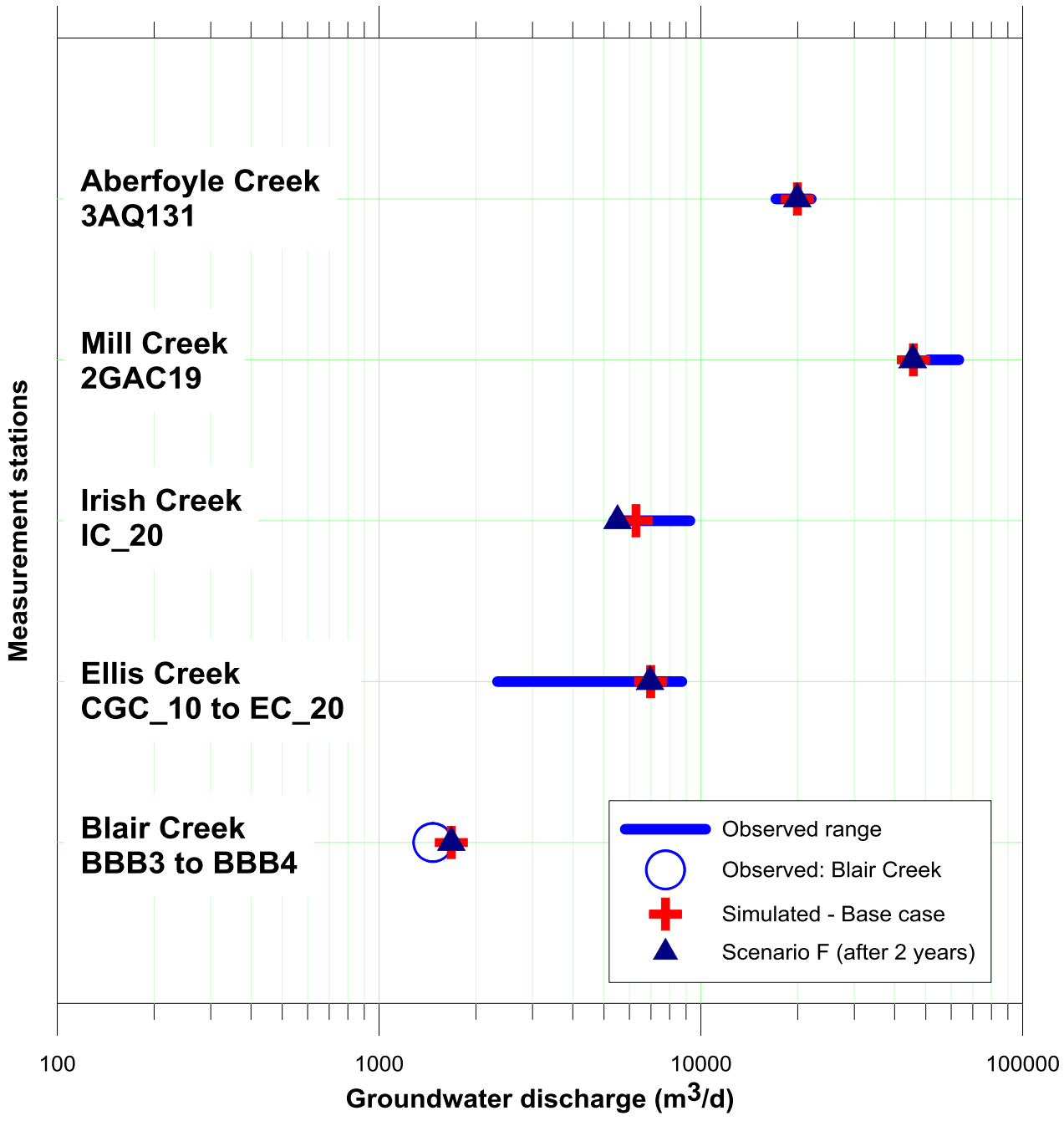
Figure 74

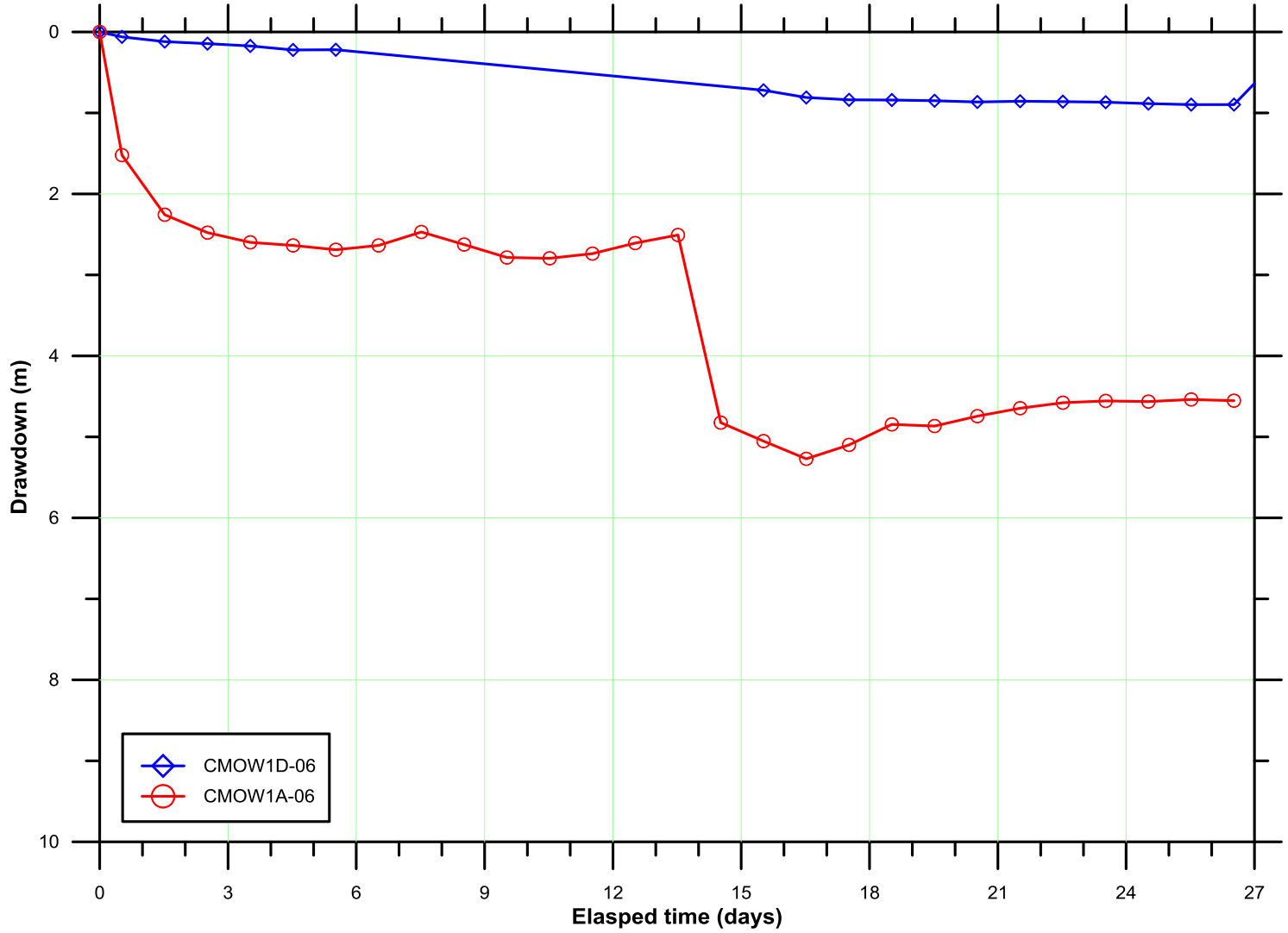
Simulated Drawdowns in the Middle Gasport Formation after 2 years (Scenario F vs. the Base Case)



CAMBRIDGE EAST IUS WATER SUPPLY CLASS : GROUNDWATER MODELLING

Figure 75 Simulated Transient Changes in Groundwater Discharge to Streams after 2 years (Scenario F vs. the Base Case)

Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 76 Example Shallow and Deep Responses Observed During the 28-day Pumping Test

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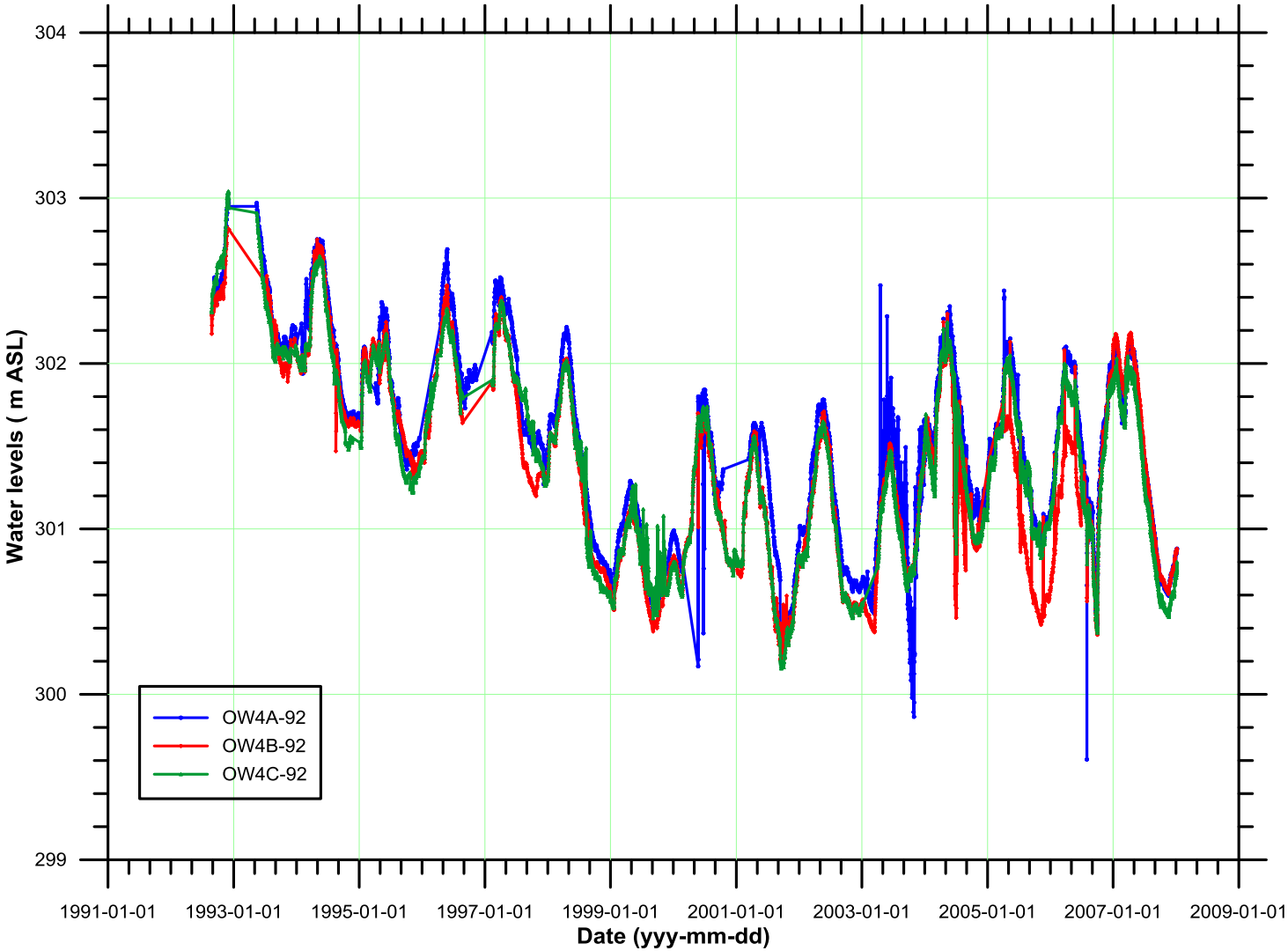
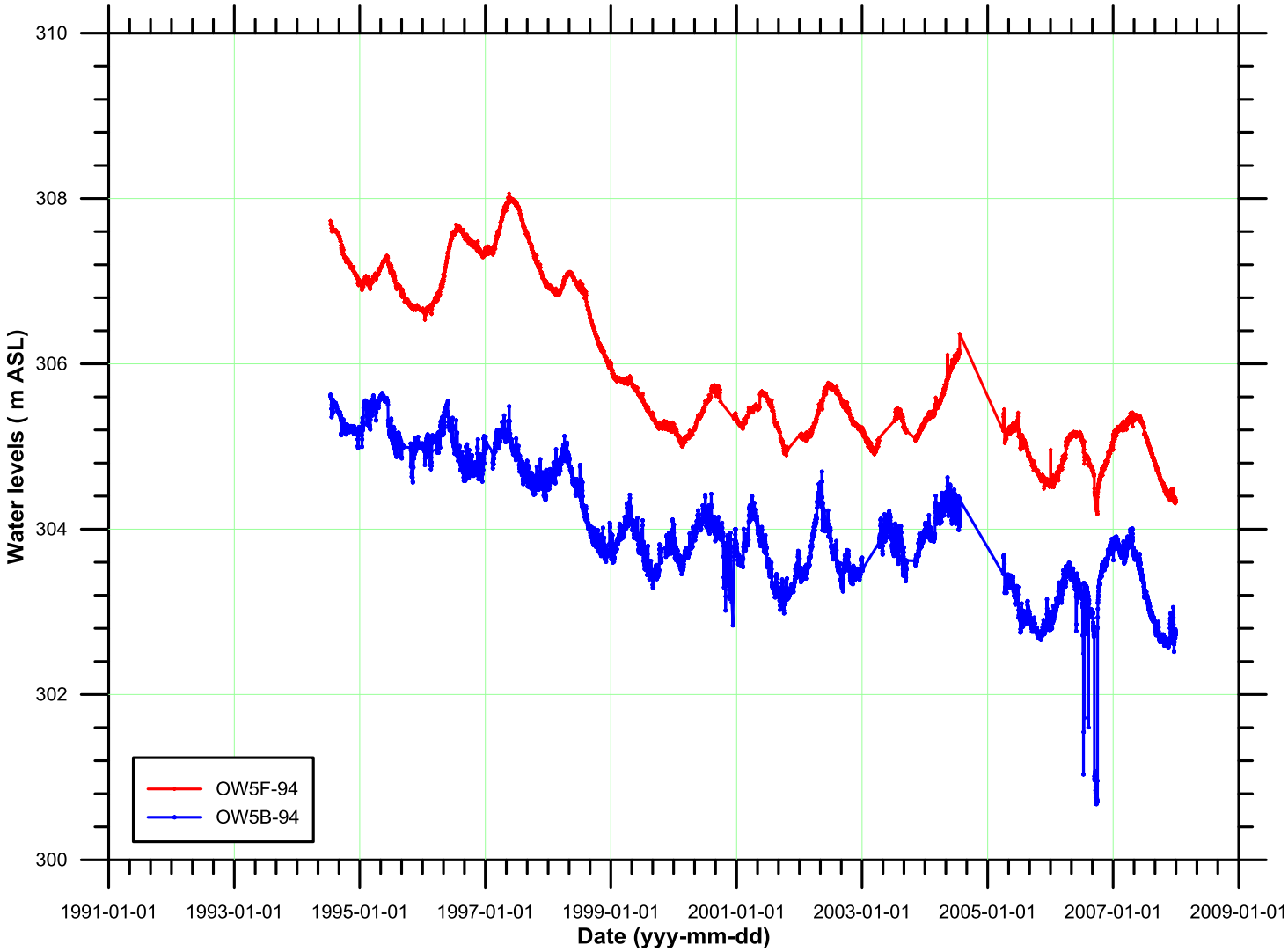


CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 77 Long-term Groundwater Levels at OW4-92

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 78 Long-term Groundwater Levels at OW5-94

Matrix Solutions Inc.
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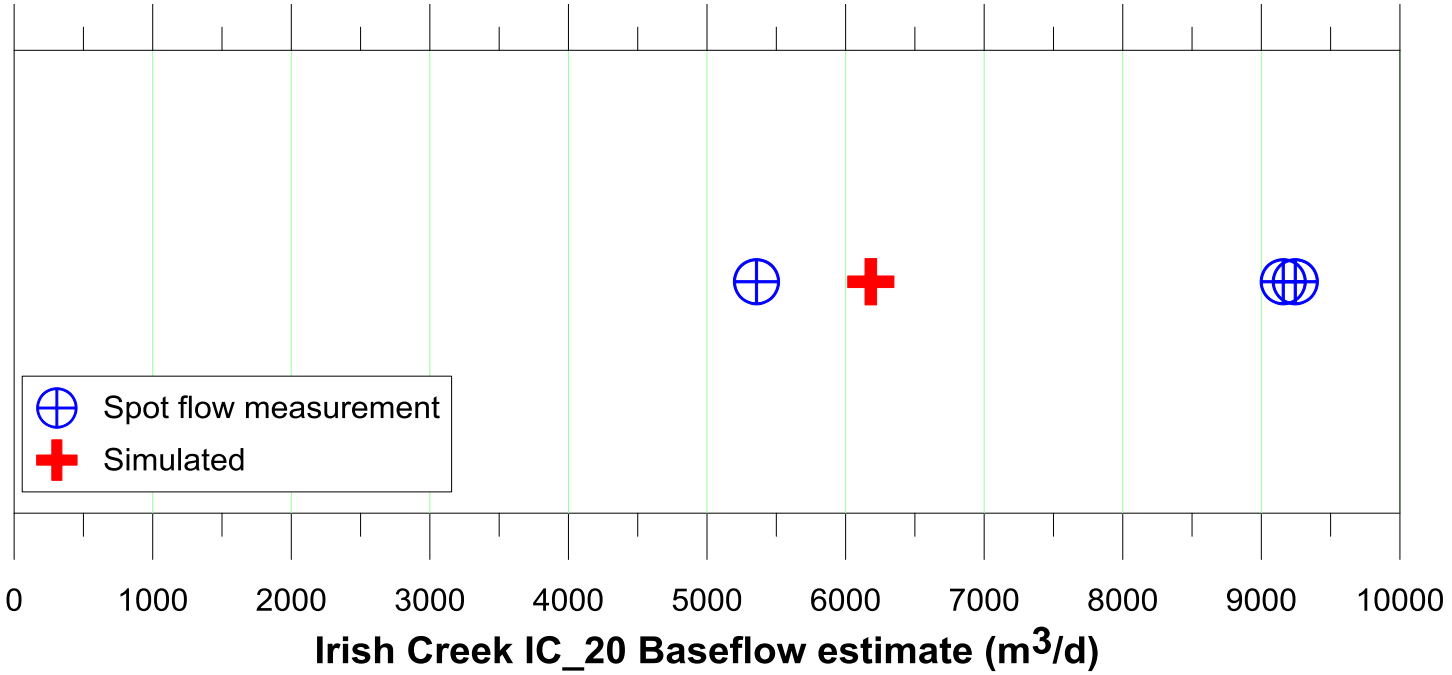
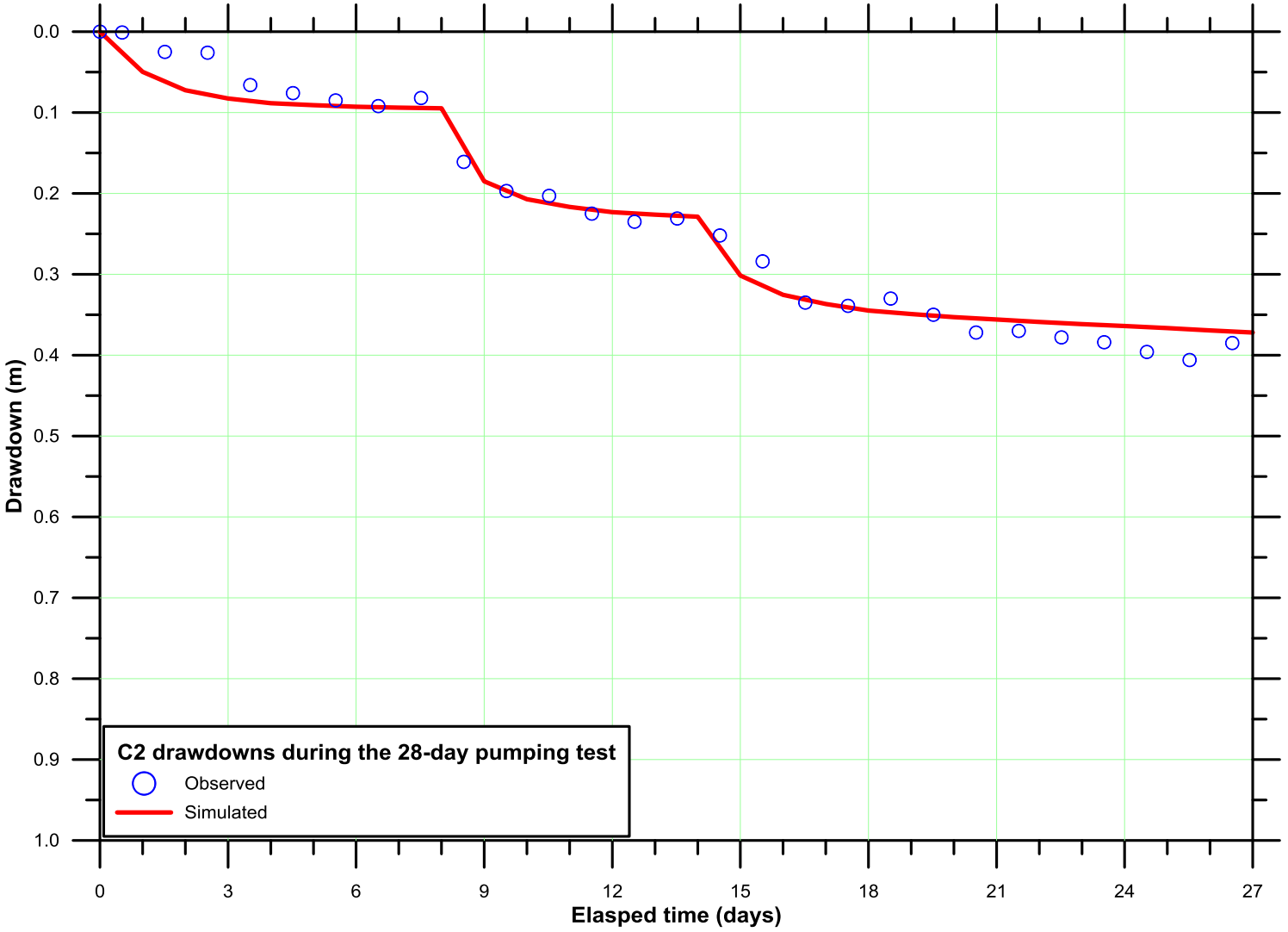


CAMBRIDGE EAST IUS WATER SUPPLY
CLASS EA: GROUNDWATER MODELLING

Figure 79 Baseflow Estimates for Irish Creek; Spot Flow Measurements at IC_20

Matrix Solutions Inc.



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 80 Observed and Simulated Drawdowns at C2 During the 28-Day pumping Test

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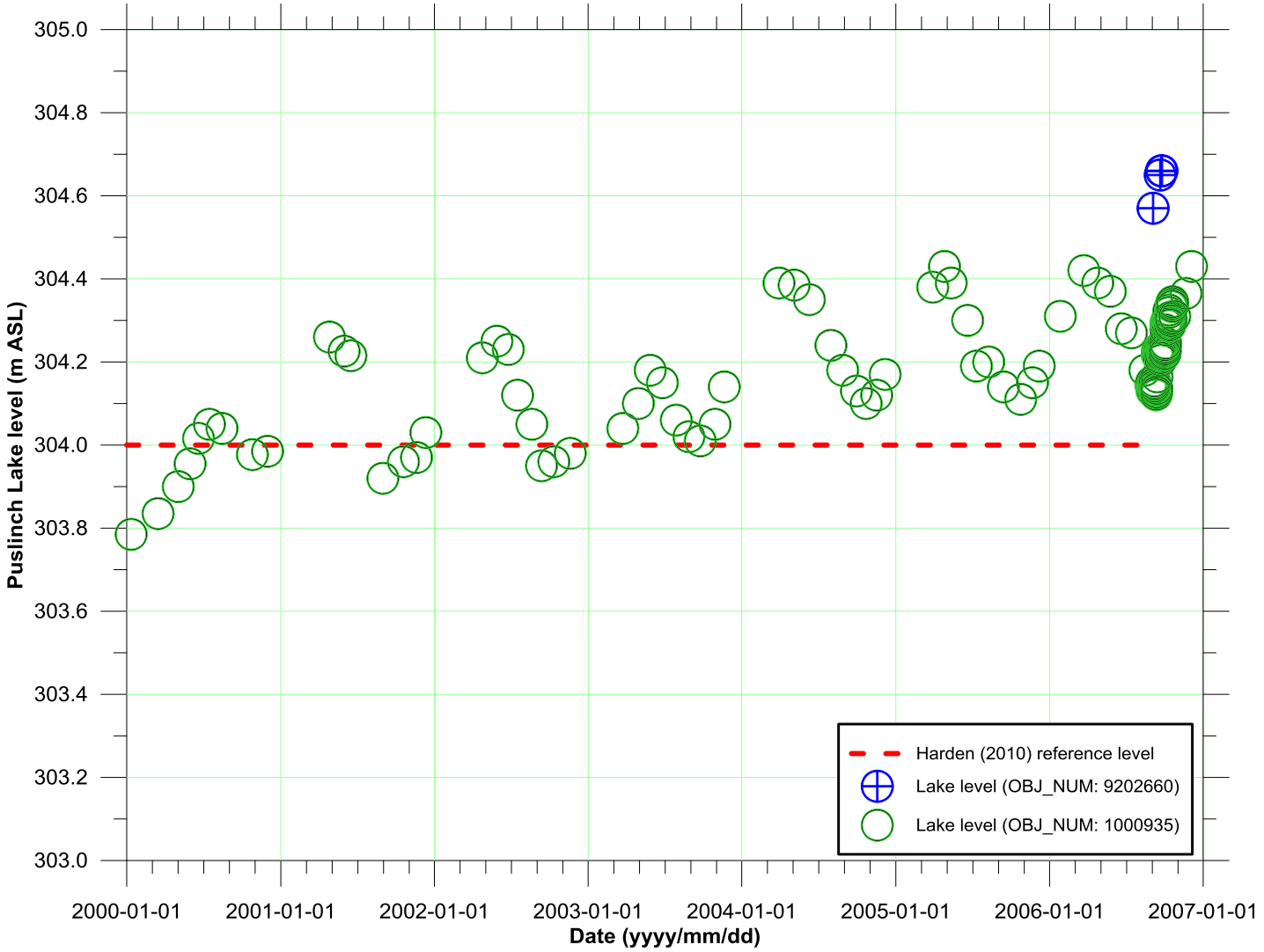
ENVIRONMENT & ENGINEERING



CAMBRIDGE EAST IUS WATER SUPPLY CLASS EA: GROUNDWATER MODELLING

Figure 81 Water Levels in Puslinch Lake

Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING

S.S. PAPADOPULOS & ASSOCIATES, INC.
Environmental & Water-Resource Consultants

Memorandum

Date:	April 20, 2017
From:	Christopher Neville and Jinhui Zhang
To:	Richard Wootton, The Regional Municipality of Waterloo
Project:	SSP-1234
Subject:	**Scenarios A and B: Three-month transient simulation results**

Overview

As requested by Mr. Richard Wootton from the Regional Municipality of Waterloo and Mr. John Piersol from Golder Associates, the transient scenarios A and B for the Cambridge East IUS Water Supply Class EA have been re-simulated. The simulation results after three months of pumping are presented in this memorandum.

1. Introduction

In the final report on the groundwater modeling to support the Cambridge East IUS Water Supply Class EA report, results are presented for six transient scenarios developed to assess the redundancy of supply for the Integrated Urban System (IUS) in the Cambridge area (SSP&A and Matrix Solutions, 2016). Specifically, the scenarios were developed to predict the potential changes arising after two years of additional pumping from Cambridge East wells to make up for a shutdown of Middleton Street well G2. For both scenarios A and B, it was assumed that pumping from G2 was stopped. The loss of supply was made up by additional pumping from the Pinebush and Clemens Mill Well Fields, with all other municipal pumping in the Cambridge area maintained at projections of 2031 rates developed for the Tier Three Water Budget Study. Scenario A was developed to assess the potential effects of making up the pumping mainly from the Pinebush Well Field with the remainder from the Clemens Mill Well Field. Scenario B was developed to assess the effects of making up the pumping mainly from the Clemens Mill Well Field with the remainder from the Pinebush Well Field. Detailed descriptions about the setup of Scenarios A and B are presented in Section 7.4.1 and 7.4.2 of the final EA report.

The same initial conditions have been specified for the revised simulations presented here; however, the approaches for the calculation of the changes in groundwater levels (drawdowns) are different. The initial conditions for Scenarios A and B are specified as the results of the steady-state simulation with the Tier 3 2031 projected pumping rates; these conditions correspond to the results from steady-state Scenario 1 in the final EA report (Section 7.3.1). In the final report on the modeling the drawdowns were calculated by subtracting the simulated transient water levels from the simulated Scenario 1 levels. In contrast, for the re-simulations presented here the drawdowns are calculated by subtracting the simulated groundwater levels for Scenario A and B at the end of the 3-month simulation period from the groundwater levels simulated for 2008 average pumping rates (Section 7.2 in the final EA report). Mr. John Piersol from Golder Associates requested the revised approach. The resulting drawdown contours reflect the cumulative effects of pumping using Tier Three projected 2031 pumping rate and the additional pumping from Pinebush and Clemens Mill Well Field to make up for the temporary loss of supply from G2.

2. Results for Scenario A

The contours of simulated drawdowns in the Contact Aquifer and the Middle Gasport Formation for Scenario A after three months of pumping are shown in Figures 1 and 2, respectively. Positive values of the drawdown denote declines in the water levels with respect to the 2008 average pumping conditions.

As shown in Figure 1, in the Contact Aquifer the 1 m drawdown contour extends over the eastern part of the Pinebush and northern part of the Clemens Mill Well Fields. The maximum drawdown in the aquifer is centered around P10 (more precisely its replacement well P10B), and is about 4 m. The drawdown cone around P10B is steep. As shown in Figure 2, the maximum drawdown in the Middle Gasport Formation is about 27 m, and is predicted to occur around P10A. The drawdowns around the Cedarbrook well are about 4 m.

The calculated groundwater discharges to the surface water streams for Scenario A are listed on Table 1. The only appreciable difference between the Base Case (2008 average pumping) and Scenario A is the reduction in the net groundwater discharge to Irish Creek.

Table 1 Simulated groundwater discharges to streams – Scenario A

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario A Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19945	19937
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45605
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5456
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove and Kossuth Rd.	6984	6843
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd. and Blair Rd.	1677	1676

The calculated components of the Puslinch Lake water balance after three months of pumping are listed on Table 2. Negative value of flows on Table 2 denote outflows from Puslinch Lake. The

Table 2 Simulation Results for Puslinch Lake – Scenario A

Quantity	Base Case (2008 average rate)	Scenario A
Net lateral flow (negative for outflow)	-1088 m^3/d	-747 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-984 m^3/d	-858 m^3/d

3. Results of Scenario B

The contours of simulated drawdowns in the Contact Aquifer and the Middle Gasport Formation for Scenario B after three-month pumping are shown in Figures 3 and 4, respectively. Positive values of drawdowns denote declines in the water levels with respect to 2008 average pumping conditions.

As shown in Figure 3, in the Contact Aquifer the 1 m drawdown contour extends over the eastern part of the Pinebush and part of the Clemens Mill Well Fields. The maximum drawdown in the aquifer is centered around P10B, and is about 3 m. As shown in Figure 4, the maximum drawdown in the Middle Gasport Formation is about 22 m, and is predicted to occur around P10A. The drawdowns around the Cedarbrook well are about 8 m.

The calculated groundwater discharges to Irish Creek streams for Scenario A are listed on Table 3. The only appreciable difference between the Base Case (2008 average pumping) and Scenario A is the reduction in the net groundwater discharge to Irish Creek.

Table 3 Simulated groundwater discharges to streams – Scenario B

Stations	Station description	Base Case Simulated Net Groundwater Discharge to Streams (m^3/d)	Scenario A Simulated Net Groundwater Discharge to Streams (m^3/d)
GRCA gauges			
Aberfoyle Creek (3AQ131)	GRCA Aberfoyle Creek at Twp Rd. 7	19945	19937
Mill Creek (2GAC19)	GRCA Mill Creek at Sideroad 10	45739	45593
Spotflow measurements			
Irish Creek (IC_20)	Irish Creek at Townline Rd.	6288	5433
Ellis Creek (between EC_20 and CGC10)	Ellis Creek between Maple Grove and Kossuth Rd.	6984	6844
Blair Creek (between BBB3 and BBB4)	Blair Creek between Dickie Settlement Rd. and Blair Rd.	1677	1676

The calculated components of the Puslinch Lake water balance after three months of pumping are listed on Table 4.

Table 4 Simulation Results for Puslinch Lake – Scenario B

Quantity	Base Case (2008 average rate)	Scenario A
Net lateral flow (negative for outflow)	-1088 m^3/d	-747 m^3/d
Net vertical flow across the lake bed (negative for downwards flow out of the lake)	-974 m^3/d	-869 m^3/d

4. References

S. S. Papadopulos & Associates and Matrix Solution Inc., 2016. Cambridge East IUS Water Supply Class EA: Groundwater Modelling, Final report submitted to the Regional Municipality of Waterloo, January 5, 2016.