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DESIGN OF GROUNDWATER INTERCEPTION SYSTEM FOR ROSE CREEK TAILINGS STORAGE FACILITY, FARO MINE, YUKON TERRITORY

Volume 1 - Main Report



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On behalf of

Faro Mine Closure Planning Office

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DESIGN OF GROUNDWATER INTERCEPTION SYSTEM, ROSE CREEK TAILINGS STORAGE FACILITY, FARO MINE, YUKON TERRITORY

By Robertson GeoConsultants Inc.

EXECUTIVE SUMMARY

As part of the 2005/06 closure studies, a comprehensive groundwater study was carried out under Task 22H(ii) entitled "Complete Groundwater Collection Design". The primary objective of this study was to design a groundwater interception system (GIS) for the Rose Creek Tailings Storage Facility. This study included (i) the installation of 2 pumping wells and performance of several pumping tests, (ii) calibration of a groundwater flow model and subsequent simulation of alternative groundwater interception scenarios; and (iii) a pre-feasibility design of a groundwater interception system at the toe of the Cross Valley Dam (including preliminary costing).

The results of the pumping tests indicated that groundwater yields in the south-central portion of the aquifer are generally higher than on the north side, sustaining pumping rates in excess of 45 L/s (600 USGPM) compared to an estimated 23-30 L/s (300-400 USGPM) on the north side. A three-dimensional groundwater flow model was developed for the lower reach of the Rose Creek aquifer using the finite difference model MODFLOW^{96}. The calibrated flow model suggests that the total flux of groundwater passing beneath the Cross Valley Dam is about 58 L/s. The calibrated groundwater flow model predicted that 10 pumping wells with a drawdown of 4m in the high permeability sediments and 5m in the low permeability sediments would be required to provide successful capture (<1% by-pass). For this scenario, the predicted total amount of groundwater pumped would be 72.0 L/s with individual pumping rates ranging from 0.3 to 22 L/s (or 7 to 344 USGPM).

An adaptive management plan is proposed for the design and implementation of the groundwater interception system (GIS) that would ensure that the required capture efficiency is met with a high degree of confidence. For the purpose of this pre-feasibility design, we recommend the installation of ten (10) extraction wells at a horizontal spacing of 55 metes at the toe of the Cross Valley Dam. The installation of a cut-off wall is included as a contingency measure. A detailed monitoring program would be put in place to monitor the performance of the GIS. If initial testing suggests that pumping can not achieve the required capture efficiency (e.g. due to significant heterogeneity) then a cut-off wall (possibly extending down to bedrock, if required) would be installed immediately downgradient of the fence of pumping wells.

Additional characterization work will be required prior to final design of a groundwater interception system at the Cross Valley Dam. Recommendations for future site characterization include (i) additional pumping tests in the deeper portion of the aquifer, (ii) drilling and hydraulic testing on the north and south side of the proposed alignment of the fence of wells (including in shallow bedrock). The results of these tests should be used to re-calibrate the groundwater flow model and, if required, update the design of the groundwater interception system.

DESIGN OF GROUNDWATER INTERCEPTION SYSTEM ROSE CREEK TAILINGS STORAGE FACILITY, FARO MINE, YUKON TERRITORY

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DESIGN OF GROUNDWATER INTERCEPTION SYSTEM ROSE CREEK TAILINGS STORAGE FACILITY, FARO MINE, YUKON TERRITORY

1 INTRODUCTION

1.1 Terms of Reference

One of the primary issues in the context of closure planning for the Anvil Range Mining Complex will be the development and assessment of decommissioning options for the Rose Creek Tailings Facility. As part of the 2004/05 closure studies, a water and load balance study was completed for the Rose Creek Tailings Storage Facility (Robertson GeoConsultants Inc., 2006). The water and load balance study predicted that seepage from the Rose Creek tailings facility may significantly impact the water quality in the Rose Creek aquifer. It was concluded that a groundwater interception system would likely be required to protect the water quality in Rose Creek.

The design of a groundwater collection system for the Rose Creek Tailings Storage Facility was covered in the 2005/06 budget under Task 22 H(i) entitled "Complete Groundwater Collection Design". For the purpose of budgeting for 2005-06, the scope for Task 22H(i) was generally defined as follows:

"*This task consists of pumping tests at the Cross Valley Dam. The pumping test component will start with an office assessment to estimate the scope and duration of the pump test(s). Pumping and observation wells would be installed and 1 to 2 week pumping tests completed. The objective of testing is to determine (i) feasibility of pumping the north and south side, (ii) influence of pumping on the Cross Valley seepage and (iii) boundary effects of the aquifer system. Results will be analyzed to assess and design a groundwater collection and water treatment system. The results would provide more reliable estimates of the amount of groundwater to be pumped (for water treatment design) as well as design criteria for a pumping system (depth of screening, interval between wells, pumping rates on south and north side of valley).* "

Note that another component of this study on "attenuation testing" has been split (for administrative purposes) into a separate Task 22H(ii). This report only focuses on the aspect of "groundwater collection". The results of Task 22H(ii) on attenuation testing will be provided by SRK Consulting under separate cover.

1.2 Rationale and Study Objectives

The rationale for this study was described in a memorandum dated May 6, 2005 and entitled "Task 22H(a) – Pump Testing in Rose Creek Valley" (see Appendix A). Briefly, the results of the water and load balance study conducted by Robertson GeoConsultants Inc. (RGC) suggest that current and future tailings seepage is impacting the local groundwater and that groundwater interception may be required in the future to protect Rose Creek (Robertson GeoConsultants Inc., 2006). In 2004, Gartner Lee Limited developed a preliminary design for a groundwater interception system in the Rose Creek Valley aquifer (Gartner Lee Limited, 2004). However, this design was based on modeling alone (no detailed pump test data were available at the time). Furthermore, there is concern that the proposed location (along the crest of the Intermediate Dam) may not be the most suitable location for such an interception system (Appendix A).

The primary purpose of this study is to design a groundwater interception system for the Rose Creek aquifer using pump tests and a numerical model. The specific study objectives may be summarized as follows:

- Determine hydraulic properties (T and S) of the Rose Creek aquifer;
- Assess boundary conditions (valley sides; pond seepage, leakage from RCDC etc) that may influence the performance of a groundwater interception system;
- Assess the influence of aquifer heterogeneity on capture efficiency;
- Design a groundwater interception system at the pre-feasibility level including,
    - Number & location of pumping wells,
    - Required depth of pumping wells;
    - Individual and total pumping rates
- Provide a cost estimate for installation of the proposed groundwater interception system (excluding power supply and pumping to treatment plant).

1.3 Scope of Work

A detailed scope of work for this study was presented in a memorandum, dated July 26, 2005, from Robertson GeoConsultants Inc. (RGC) to Deloitte & Touche Inc. and members of the review panel. This scope of work is reproduced in Appendix A of this report.

The scope of work for Task 22H(i) consists of five main tasks:

- Task 1. Development of Work Plan;
- Task 2. Drilling and Development;
- Task 3. Execution of Pumping Tests;
- Task 4: Test Interpretation & Design;
- Task 5: Costing & Reporting.

Task 1 includes a data review and the development of a preliminary flow model of the lower Rose Creek Valley required for designing the pumping tests (locations and size of pumping wells, pumping rates etc.).

Task 2 includes the drilling of two pumping wells and five monitoring wells at the toe of the Cross Valley Dam in preparation of the pump tests.

Task 3 consists of the execution of several pumping tests at the toe of the Cross Valley, including a 56-hour pumping test on the north side and a 24-hour pumping test on the south side.

Task 4 includes the development of a three-dimensional groundwater flow model for the area and calibration of the model using the pumping test data. The calibrated groundwater flow model is then used to design the groundwater interception system. The model is also used to assess potential future changes to the groundwater flow system (e.g. removal of the Cross Valley Dam and Polishing Pond).

Task 5 includes a costing of the proposed groundwater interception system (at the pre-feasibility level) and reporting of all results (this report).

1.4 Organization of Report

This report summarizes the study on the design of a groundwater interception system to be located downgradient of the Rose Creek Tailings Storage facility.

Section 2 describes the methods and results of the 2005 Drilling Program completed at the toe of the Cross Valley Dam. Section 3 describes the methods and results of the pumping test program carried out in two new pumping wells completed at the toe of the Cross Valley Dam. Section 4 summarizes the construction and calibration of a groundwater flow model for the lower reach of the Rose Creek aquifer. Section 5 describes simulations of groundwater interception scenarios along the toe of the Cross Valley Dam. Recommendations for the design of a groundwater interception design are provided in section 6. Section 7 summarizes the main conclusions and recommendations.

2 DRILLING & WELL INSTALLATION PROGRAM

2.1 Methods

2.1.1 Drilling, Installation and Development of Monitoring Wells

A total of five standard monitoring wells and one multi-level well were installed at the toe of the Cross Valley Dam. Figure 2.1 shows the locations of these new monitoring wells and table 2.1 summarizes pertinent construction details.

The primary purpose of all new monitoring wells was to serve as observation wells during the planned pumping tests (see section 3). Monitoring wells MW1 and MW2 were drilled in close proximity (~5m distance) of the planned pumping well locations and drilling information from these holes also served to finalize the screening interval and screen slot size of the pumping wells (see below). These wells were only drilled to serve as observation wells during pump testing and are not intended for routine monitoring.

Table 2.1

Construction Details of 2005 Monitoring Wells & Pumping Wells.

ID	Location	Easting (m UTM)	Northing (m UTM)	Borehole Depth (m bgs)	Screen Interval[1] (m bTOC)	Stickup[2] (m)	TOC elevation (m NAD27)	SWL[3] (m bTOC)
Pumping Wells								
PW1	North Side of CVD	580,156.7	6,914,285.8	21.08	5.10-6.70 17.29-20.49	0.91	1017.31	2.31
PW2	South Side of CVD	580,097.8	6,914,195.1	16.91	4.96-16.16	0.77	1018.64	3.47
Monitoring Wells								
MW1	5m South of PW1	580,153.6	6,914,282.4	17.67	2.61-9.74 12.83-18.33	0.66	1016.96	1.91
MW2	6m North of PW2	580,100.0	6,914,200.1	16.76	2.97-16.93	0.78	1018.23	3.13
P05-01	50m North of PW1	580,165.0	6,914,334.9	26.52	4.02	0.67	1017.99	2.35
					6.15			2.23
					11.95			1.90
					17.44			1.83
					20.49			1.82
					25.97			1.67
P05-02	25m South of PW1	580,143.5	6,914,264.7	5.18	1.84-5.81	0.93	1016.66	1.86
P05-03	26m South of PW2	580,087.6	6,914,171.0	7.62	4.29-8.47	0.85	1019.79	4.59

1. Multilevel screen intervals presented as midpoint, all screen lengths ~0.15 m.
2. Stickup measured during survey, except PW1 and PW2 (measured with tape).
3. Static water level survey taken September 6, 2005.

Monitoring wells P05-01, P05-02 and P05-03 were drilled at greater distance from the planned pumping wells (25-50m distance). These monitoring wells were placed at strategic locations to complement the existing monitoring network downgradient of the Cross Valley Dam and are intended for future routine monitoring.

All drilling and monitoring well installation was carried out between August 5-7, 2005 under the supervision of Dr. Christoph Wels from Robertson GeoConsultants Inc.. All monitoring wells were drilled by Sonic Drilling Services, a division of Boart Longyear of Alberta using a Nodwell-mounted sonic drill rig and pipe truck (Photo 1). The sonic rig was equipped with a 4 x 6 system (4" or 100mm core barrel and 6" or 150mm casing) that allowed for continuous sampling in 10 ft sections (1 core barrel = 10ft; approximately 3 meters) by advancing the core barrel using ultra-sonic vibrations. Casing was advanced over the core barrel to below the bit to keep the hole open during barrel retrieval. Water was only used during casing advancement to prevent heave between barrel and casing. Water use was kept to the minimum required to advance casing.

Core was recovered in the drill tube and "extruded" into plastic bags, preserving most, if not all, of the natural stratigraphy (Photo 2). Plastic bags were laid out and the core samples logged in the field as it was recovered (Photo 3). Core recovery was generally very good with only minor core losses in P05-02.

All monitoring wells were installed through the 6" casing to prevent collapse of the natural formation. The single monitoring wells (MW1, MW2, P05-02 and P05-03) were completed using 50 mm (2-inch) threaded PVC riser pipes and screens (#10 slot size). The natural formation material (typically clean sand & gravel) was allowed to collapse around the screen to form a natural "gravel pack" by slowly retrieving the casing using sonic vibration. Next, a 10-20 filter sand was emplaced to a height of approximately 0.3 to 1.0m above the top of the screen section, followed by at least 0.3m of ¼" bentonite pellets to seal the screening area.

The multi-level monitoring well P05-01 was completed with six sampling ports using similar construction/installation methods as used earlier for the P03 series of multilevel wells (see Gartner Lee Limited, 2003). Briefly, the sampling tubes consist of continuous 5/8" OD HDPE tubing (standard size sampling tubing). Each screened section was constructed by slotting the HDPE tubing and wrapping it several times with a fine Nitex mesh (Photo 4). All sampling tubes were then fastened to the PVC centre stalk and the entire pre-assembled bundle lowered as one piece into the cased borehole (see Appendix B for more details). The natural formation was allowed to collapse around the multi-level bundle up to 5.5m below ground surface. The upper 5.5m were completed using 10-20 filter sand and ¼" bentonite pellets to provide a filter pack and seal for sampling ports #5 and #6 (see Appendix B for more details).

All monitoring wells were completed at surface by placing a bentonite seal (backfilling at least the upper 0.6m of the bore hole using bentonite chips) and by protecting the PVC casing with a lockable steel protective casing. The specific completion details for individual monitoring wells are summarized in the drilling logs provided in Appendix B.

All monitoring wells were developed after completion to remove any fines from the natural filter pack and the bottom of the PVC casing. Monitoring wells MW1, MW2, P05-02 and P05-03 were equipped with dedicated Waterra^{TM} tubing and standard Delrin^{TM} foot valves (D-25) for development and

sampling. Well development was carried out using a Waterra^{TM} hydrolift pump until field parameters had stabilized and the purged water was clear (Photo 5).

The sampling ports of the multilevel monitoring well P05-01 were developed using a peristaltic pump (Geopump 2) with variable speed control. Purging was accomplished by inserting small-diameter "spaghetti" tubing into the HDPE sampling tubes as close as possible to the sampling port (Photo 6). Again, all sampling ports were purged until the field parameters (pH and EC) had stabilized and the purged water was sufficiently clear for sampling. Table 2.2 summarizes the volumes purged during initial well development and initial readings of water quality.

Table 2.2

Summary of Monitoring Well Development.

Well ID	Volume Purged (liters)	Field pH	Field Specific Conductance (uS/cm)	Comments
MW1	800	7.0	1927	clear
MW2	900	7.2	1230	clear
P05-01 Port #1	150	7.1	1861	clear
P05-01 Port #2	80	7.3	1856	clear
P05-01 Port #3	40	6.9	1632	clear
P05-01 Port #4	20	7.3	1645	clear
P05-01 Port #5	90	7.2	1413	clear
P05-01 Port #6	70	7.1	1531	clear
P05-02	60	6.6	2100	clear
P05-03	100	7.1	n.a.	Some bentonite at bottom; water slightly turbid

The initial sampling of selected monitoring wells (P05-01 and P05-02) for water quality analysis was carried out on September 9, 2005 by Gartner Lee Ltd. as part of the routine monitoring program for the water licence at the Anvil Range Mine using standard sampling protocols and QA/QC procedures (Gartner Lee Ltd., 2006).

2.1.2 Drilling, Installation & Development of Pumping Wells

Two pumping wells (PW1 and PW2) were installed at the toe of the Cross Valley Dam. Figure 2.1 shows the locations of these pumping wells and Table 2.1 summarizes pertinent construction details.

The two pumping wells were intended for two purposes. First, the two wells were to be used to conduct a pumping test on the north and south side of the Rose Creek Valley, respectively. Second,

the two pumping wells should be able to be used for actual recovery of contaminated groundwater in the future, if and when required.

Both objectives influenced the final design of the pumping wells. Preliminary modeling suggested that the pumping rate would have to be >2,000 m^3/day (>400 USGPM) to stress the entire aquifer (Appendix A). In order to achieve such a high pumping rate a large diameter well assembly (8-inch diameter) was selected requiring a nominal 10-inch drill hole. In order to serve as a permanent recovery well in the future, a durable and highly efficient well was required. This second objective was achieved by using stainless steel Type V Johnson screens (continuous slot wire wrap) for well construction and developing a natural filter pack around these well screens.

Drilling, well installation and well development was carried out between August 12-17, 2005 under the supervision of Ms. Laura Findlater from Robertson GeoConsultants Inc.. The two pumping wells were drilled by Cora Lynn Drilling, Fort St John, B.C., using a DR24 Foremost Drill on a tri drive axel Freightliner and a pipe truck (Photo 7). The rig was equipped with a dual rotary drill with Top Head torque of 9,400 foot pounds and a lower Casing Drive torque of 117,000 foot pounds. Air for drilling was supplied by an Atlas Copco air compressor with 900 CFM and 350 PSI. The drill rig was operated by Mr. Bill Wiebe.

The pumping wells were drilled using a 10¾-inch (OD) steel casing with carbide studded casing shoe and a 9-7/8 inch tricone bit (Photo 8). The steel casing was rotated into the ground using the casing driver to isolate the bore hole and control heaving. The tricone bit was advanced with the top drive to drill out the material in front of the casing and retrieve the drill cuttings. The sample was retrieved by injecting air through the drill rods and forcing the drill cuttings up the drill hole in the annulus between the drill rods and the outer casing.

During drilling, the drill cuttings were sampled in 1ft (0.3m) intervals and logged in the field (Photo 9). The observed stratigraphy (in particular at PW1) differed slightly from the near-by SONIC drill holes, hence the well design (total depth, length and slot size of screening intervals) was finalized based on the drill cuttings recovered from the pumping wells.

The well assembly consisted of a 5-ft long tail pipe, the well screen(s) (with slot sizes ranging from #30 to #200) and steel riser pipe to above ground surface. All elements of the well assembly were welded together and have a nominal 8-inch diameter (200mm) (Photo 10). The well construction details are summarized in the drill logs for the pumping wells provided in Appendix B.

Once the bore hole was drilled to the final design depth the drill rods and bit were removed and the well assembly was lowered into the bore hole through the 10-inch steel casing (Photo 11). At PW2, some heaving of silty material into the casing was observed after removal of the drill bit. This sediment was removed from the cased drill hole using a steel bailer prior to well assembly installation. No heaving of sediments into the cased hole was observed at PW1.

Once the well assembly had been lowered into the borehole, the casing was pulled and the natural formation was allowed to collapse around the well screen(s) and riser pipe. The remaining annulus (1-2 ft bgs) was backfilled with bentonite chips to surface. The well casing was fitted with a lockable lid and protected above ground by a 1m x 1m x 0.3m concrete pad.

Both pumping wells were subsequently developed primarily by jetting, surging and pumping with compressed air using a jetting tool (Photo 12). PW2 was first bailed with a steel bailer for 1.5 hours to remove fines that had entered the well string during installation. After the initial bailing the well was developed for 18.5 hours by "jetting" the well screen (cleaning the well screen with compressed air), followed by air surging and air lift pumping to remove any sediment. PW1 did not require bailing and was developed for 8.5 hours using only the air development methods (i.e. jetting, air surging and air-lift pumping).

The airlift yields for the two pumping wells were visually estimated to be about 250 USGPM for PW1 and 300 USGPM for PW2. During development of PW2, air and water was being observed entering the near-by monitoring well MW2 through the screen suggesting some "collapse" of the formation structure within a few meters distance of the pumping well. This observation is consistent with pumping test results which suggested a highly permeable connection between those two wells (see section 3). No such formation collapse was observed at PW1.

By the end of well development, both pumping wells produced clear water suitable for pumping over extended periods of time.

The groundwater pumped from the two pumping wells was first sampled for water quality analysis during the pumping tests carried out in early September, 2005 by Robertson GeoConsultants Inc.. A total of four samples were obtained at various times throughout the pumping tests using an in-line sampling port. The samples were collected in pre-washed 1L sampling bottles and refrigerated on site until all samples had been collected (within about 7 days). A sub-aliquot of each sample was filtered on site through a 0.45µm filter within 48 hours of sampling for dissolved metal analysis. The sub-samples for dissolved metals and total metals were preserved on site using nitric acid provided by the laboratory. For shipment, all samples were placed on ice in a cooler and shipped to ALS Laboratories for analysis.

The complete chemical analyses report (for the PW samples collected as part of this study) is provided in Appendix C.

2.1.3 Survey

All monitoring wells and pumping wells were surveyed by Yukon Engineering Services (YES) in NAD27 (Yukon) coordinates using the GSD95 Geoid. The surveyed locations and geodetic elevations of the top of casing (TOC) of the wells are summarized in Table 2.1.

2.2 Results & Discussion

2.2.1 Hydrostratigraphy

In general, the boreholes intersected glaciofluvial deposits ranging in grading from very coarse sand and gravel with cobbles (GW) to low plasticity silts (ML). In a given borehole, the grading of the sediments often varied considerably over very short depth intervals (often less than 30cm) suggesting a highly chaotic depositional environment characteristic of an outwash deposit (Gartner Lee Limited, 2002). Detailed descriptions of the soil profiles encountered at the various drilling locations are provided in Appendix B.

The pumping wells were screened in highly productive sand & gravel layers. At PW1 (north side) the soil profile was heterogeneous with significant interlayering of poorly graded ("well-washed") sand & gravel layers and silt layers. Highly productive sand & gravel units of significant vertical extent were encountered from 2.7- 6.4m bgs and 14.6-21.0m bgs. The near-by monitoring wells (MW1 and P05-02) showed a similar stratigraphy except for some differences in the thickness of the major silt layers.

In contrast, the stratigraphy further to the north of the pumping well differed significantly. At multilevel well P05-01, the valley sediments were primarily classified as silty sand and/or gravel with only a few layers of well-washed sand and gravel. The higher silt content and poorer sorting of these sediments suggests a different depositional environment (possibly dominated by glacial deposition over fluvial deposition) than observed in the other holes located further towards the center of the Rose Creek valley.

At PW2 (south side), the deposits were more uniform with sand & gravel encountered from 3.0m to 15.8m bgs. This well-washed sand and gravel unit was covered by a thin sequence of silts and sands (alluvium?) and underlain by a sandy silt unit at depth. The near-by observation wells (MW2 and P05-03) generally showed a very similar stratigraphy except for some small differences in the thickness of the cover soils and the productive sand & gravel unit itself.

Figure 2.2 shows a geological cross-section across the Rose Creek Valley, i.e. aligned parallel to the Cross Valley Dam (see insert for alignment of section). The cross-section runs along the proposed alignment of the groundwater interception system, i.e. immediately downgradient of the toe of the Cross Valley Dam in line with the two new pumping wells completed in 2005. The bar diagrams show simplified interpretations of the various drilling logs available in the general vicinity of this cross-section (drilling logs for boreholes with more than 55m offset are shaded lighter). The inferred spatial distribution of the major soil units, as inferred from the individual drilling logs, is shown in colour.

Figure 2.2 illustrates the complexity of the glacio-fluvial deposits in the Rose Creek valley. Note that the 2005 drilling results are generally consistent with earlier drilling information. The soil profile encountered at PW1 is inferred to be representative for much of the northern portion of the valley (from about BPT05-05 to just past PW1) but differs significantly from the soil profile observed further to the north of PW1 (P05-01, BK05-03, P01-11). The soil profile observed at PW2 appears to be representative of much of the southern and central portion of the Rose Creek valley. However, the southern side of the valley is not as well-characterized and the soil profile may possibly differ significantly from the inferred cross-section. Note also that the soil profile observed at P03-09 (some 50m downgradient) differs significantly from that observed at PW2. In other words, the inferred cross-section likely differs significantly perpendicular to this section (in particular in a downgradient direction).

The spatial extent of the till layer and the delineation of the overburden/bedrock contact shown in Figure 2.2 generally agrees with earlier interpretations, i.e. the interpretation of the original field work fro dam design (drilling and seismic survey) carried out by Golders and Associates (Golders, 1979) and the subsequent 3D geological model developed for the Rose Creek Valley by Gartner Lee Ltd (GLL, 2002). Only one of the 2005 drill holes (P05-01) was drilled sufficiently deep to intersect the basal till and the phyllite bedrock. The depth to bedrock at this location (about 24.5m below ground) was within 1-5 meters of the earlier estimates of depth to bedrock.

2.2.2 Groundwater Levels

A complete survey of "static" groundwater levels was taken on September 6, 2005 in all wells available in the reach of interest, i.e. between the Intermediate Dam and downstream of the Cross Valley Dam. Figure 2.3 shows the observed groundwater levels at individual monitoring wells and the inferred groundwater flow field for September 6, 2005. In the case of nested (or multi-level) wells, the geodetic water level elevations were averaged and the direction of the vertical gradient was indicated with an arrow. The upward gradients and the inferred flow field suggest that the area downgradient of the Polishing Pond represents a zone of significant groundwater discharge. This interpretation is consistent with the results of the water and load balance study which indicated a net discharge of about 44-83 L/s of groundwater and toe seepage combined along the seepage channel upstream of the confluence with Rose Creek (Robertson GeoConsultants Inc., 2006).

The groundwater levels are inferred to be elevated along the southern side of the valley due to seepage from the Rose Creek Diversion Canal (in particular downstream of the Cross Valley Dam). However, no groundwater level measurements were available in this area to confirm this interpretation.

At P05-01, the groundwater level in bedrock was about 0.7m higher than in the uppermost layer of the valley sediments. A very similar range in groundwater levels between the bedrock and the upper sediments (0.65m) was observed in the multi-level monitoring well P03-09, located in the center of the valley. These upward vertical gradients in the valley center are believed to be a result of (small) inflows of groundwater through bedrock from the valley sides.

Note that the groundwater levels in the aquifer at the toe of the Cross Valley Pond are significantly lower than the water level in the Polishing Pond (1027.4m on September 7, 2005). Similarly, the groundwater levels in the aquifer at the toe of the Intermediate Dam are significantly lower than in the Intermediate Pond (1046.5m on September 7, 2005). Our interpretation of the groundwater flow field suggests that the Polishing Pond and the Intermediate Pond are "underdrained" due to the large contrast in permeability between the pond sediments/tailings and the underlying aquifer material.

2.2.3 Groundwater Quality

Table 2.3 presents selected results of the initial groundwater quality survey for the new monitoring wells and the two pumping wells (all sampled between September 7-10, 2005). These water quality results were compared to results from the other (existing) monitoring wells downgradient of the Cross Valley Dam to gain insight into the spatial distribution of the contaminants of concern (COC), i.e. in particular sulphate and zinc. Figure 2.4 compares depth profiles of SO4, Mg, Na and Zn in the two multi-level wells P05-01 and P03-09 (September 9, 2005). Figures 2.5a/b show the observed sulphate and zinc concentrations at all available monitoring wells and the inferred spatial extent of these COCs in the Rose Creek aquifer in cross-section.

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Table 2.3

Summary of Initial Groundwater Quality Survey (September 2005).

Well-ID	Date	pH	EC µS/cm	Alkalinity mg/L $CaCO_3$	SO4 mg/l	Ca mg/l	Mg-D mg/l	Na-D mg/l	Zn-D mg/l
Monitoring Wells									
P05-01-1	Sept. 9	7.36	1910	420	723	374	86.9	96.2	0.0141
P05-01-2	Sept. 9	7.46	2000	303	914	443	76.9	52.5	0.0077
P05-01-3	Sept. 9	7.48	1990	318	900	459	78.5	43.8	0.016
P05-01-4	Sept. 9	7.57	2030	325	937	397	76.7	49.8	0.0095
P05-01-5	Sept. 9	7.52	1850	281	842	345	69.2	43.4	0.0080
P05-01-06	Sept. 9	7.45	1860	271	861	333	66.9	43.5	0.0075
P05-02	Sept. 9	7.42	2010	317	895	369	68.1	50.5	0.0264
P05-03	Not sampled in 2005								
Pumping Wells									
PW1	Sept. 10	7.62	1980	322	971	386	73.4	52.6	0.0205
PW2	Sept. 7	7.90	1260	268	470	214	40.3	32.9	0.0083

The following conclusions can be drawn from an inspection of these groundwater quality data:

- At both multilevel well locations (P03-09 and P05-01) the groundwater quality is relatively uniform across the entire depth of the Rose Creek valley sediments suggesting a well-mixed aquifer system with depth; note that groundwater quality in the underlying (weathered) bedrock (at P05-01) differs significantly in major ion composition (e.g. higher Na and lower SO4) suggesting only limited mixing with groundwater flowing within the valley sediments;
- Groundwater quality varies significantly across the valley, with only marginally impacted (dilute) groundwater on the southern side of the valley (at P01-02A/B) and significantly impacted (mineralized) groundwater on the northern side (e.g. at P05-01);
    - Sulphate concentrations showed a gradual increase from south to north with the highest sulphate concentrations (~950-1,000 mg/L SO4) observed at PW1 and near-by monitoring wells (P05-01, P05-02 and P01-11); this spatial pattern suggests an upstream source on the north side and/or dilution from the south;
    - The spatial distribution of dissolved zinc in the aquifer is more uniform, with concentrations ranging from a low of about 0.008 mg/L Zn-D in the southern and central portion of the aquifer and peak concentrations of about 0.02 mg/L Zn-D on the northern side of the aquifer (at PW1 and P05-02);

In summary, groundwater quality downstream of the X Valley Dam shows significant spatial variability with a trend of increasing sulphate and zinc towards the northern side of the valley. This spatial pattern is consistent with trends observed along the Intermediate Dam suggesting that groundwater flow from up-gradient (as opposed to seepage from the X-Valley Pond) is the primary source of groundwater discharging downstream of the X-Valley Dam (Robertson GeoConsultants Inc., 2006).

2.3 Updated Review of Water Quality Time Trends

A detailed review of historical time trends in surface water and groundwater quality in the Rose Creek Valley was presented as part of the water and load balance study for the Rose Creek Tailings Storage Facility (Robertson GeoConsultants Inc., 2006). This section provides a brief update of this review for the reach of interest (downstream of Cross Valley Dam) using more recent water quality data (until end of 2005).

2.3.1 Groundwater Quality Time Trends

Figures 2.6 and 2.7 show the water quality time trends observed at X16 to X18 and P01-01, P01-02 and P01-11, respectively. Figure 2.8 shows water quality depth profiles for selected dates in P03-09. The following observations can be made:

- The recent monitoring data (2005) agree fairly well with earlier years generally
- The groundwater in the northern portion of the aquifer (X18A/B; P01-11) has shown a gradual increase in SO4 and Mg over time; current SO4 concentrations range from ~900 mg/L near the X-Valley Dam at P01-11 to ~500 mg/L at X18 (near the spillway); zinc concentrations have remained in the range of 0.01-0.02 mg/L;
- The groundwater in the southern portion of the aquifer (X16 and P01-02) has not experienced the same increase in ARD products; current SO4 concentrations range from ~100 mg/L near the X-Valley Dam to ~30 mg/L at X16;
- The groundwater in the central portion of the aquifer (at P03-09 and X17) shows intermediate SO4 and Mg concentrations; note that the monitoring well located further downgradient (X17) shows significantly lower SO4 (~40 mg/L) than observed at P03-09 (~300-400 mg/L) suggesting significant dilution by “clean” groundwater;
- Detailed depth monitoring at P03-09 indicates fairly uniform groundwater quality throughout the aquifer with only slightly higher SO4 and Mg concentrations in the upper portion of the aquifer; the most recent survey (September 2005) shows a small increase in selected constituents (incl. Mg, Na and zinc) in the upper portion of the aquifer.

In summary, the highest concentrations of COCs (SO4, Mg, Zn) continue to be observed on the northern side of the Rose Creek Valley, in the immediate vicinity of the Cross Valley Dam. Groundwater quality improves significantly within a few hundred meters from the toe of the X Valley Dam suggesting significant dilution by “clean” groundwater, likely due to seepage from the Rose Creek Diversion Canal.

2.3.2 Surface Water Quality Time Trends

The Intermediate and Cross Valley Ponds represent storage reservoirs, which may receive inflows from surface (e.g. X23 and surface runoff) and/or subsurface (tailings pore water, groundwater). Figure 2.9 shows recent water quality time trends at X4 (Intermediate Pond at spillway) and X5 (Cross Valley Pond or "Polishing Pond" at spillway). The recent time trends can be summarized as follows:

- The water in the Intermediate Pond showed consistently higher concentrations of SO4, Mg and in particular Zn than the Polishing Pond; the much lower zinc concentrations in the Polishing Pond are a result of lime addition resulting in increased pH and zinc precipitation;
- Concentrations of SO4, Mg and zinc in the Intermediate Pond have increased significantly in 2005, with sulphate concentrations of ~1,000 mg/L and zinc concentrations of ~30 mg/L; in contrast, the water quality in the Polishing Pond has remained fairly constant during the same monitoring period;
- Sulphate concentrations in the Polishing Pond continue to be lower than typical groundwater concentrations along the north side of the valley (Figure 2.5a), suggesting that seepage from the Polishing Pond is not the primary source of ARD to the aquifer.

Seepage along the toe of the Cross Valley Dam is monitored at stations X11 (north side), X12 (south side) and X13 (combined seepage). Figure 2.10 shows water quality time trends at these monitoring stations. The following observations can be made:

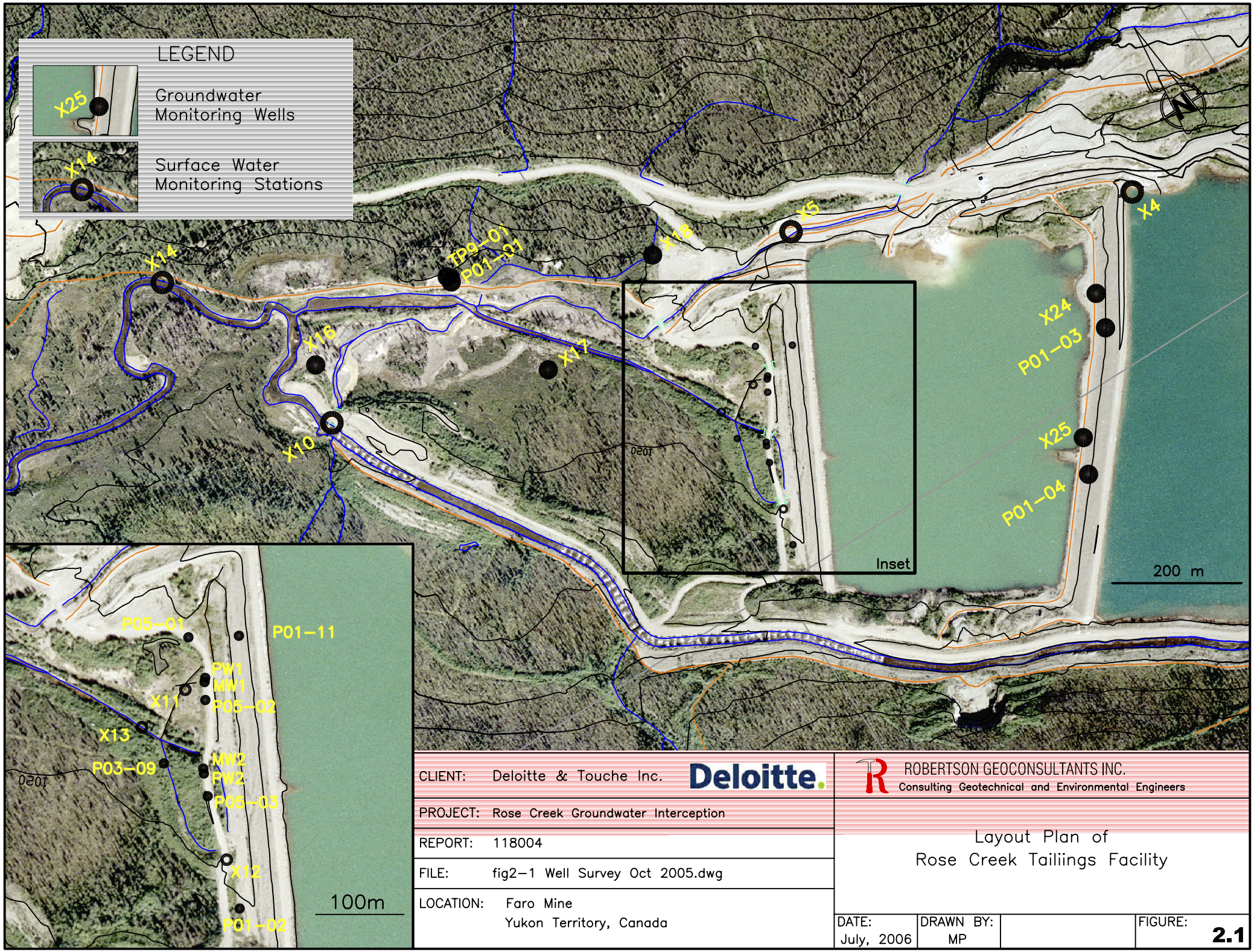
- The north side seep (X11) has shown a consistent increase in SO4 and Mg (but not zinc!) over the last few years with maximum concentrations exceeding 1,000 mg/L SO4 and 80 mg/L Mg; (note, however, that zinc concentrations have remained very low (~0.01 mg/L)); the SO4 and Mg concentrations in X11 are now significantly higher than those observed in the Polishing Pond suggesting significant discharge of impacted groundwater from the north side of the aquifer;

- The south side seep (X12) shows significantly lower concentrations of SO4, Mg and Na than the north side seep with very little, if any change over the observation period, the fact that those concentrations have remained well below those observed in the Polishing Pond would suggest that this seep receives contributions of "clean" groundwater (perhaps leakage from the Rose Creek Diversion);

- The water quality at the central seep monitoring station (X13) has remained fairly constant throughout the observation period; the water quality is representative of a mixture of impacted "north side seepage" and less impacted "south side seepage"; recent seepage flow measurements suggest that north side seepage (X11) represents about 30% of total seepage at X13 with the remaining seepage from the central and southern portion of the Rose Creek Valley (LES, 2005).

In summary, the seep water quality data are consistent with groundwater monitoring data indicating significant differences in groundwater quality between the north and the south side. They also

demonstrate that seepage from the Polishing Pond is not the only source of seepage below the Cross Valley Dam but instead is augmented by significant discharge of local groundwater.



CLIENT:	Deloitte & Touche Inc.
PROJECT:	Rose Creek Groundwater Interception
REPORT:	118004
FILE:	fig2-1 Well Survey Oct 2005.dwg
LOCATION:	Faro Mine Yukon Territory, Canada

ROBERTSON GEOCONSULTANTS INC.
Consulting Geotechnical and Environmental Engineers

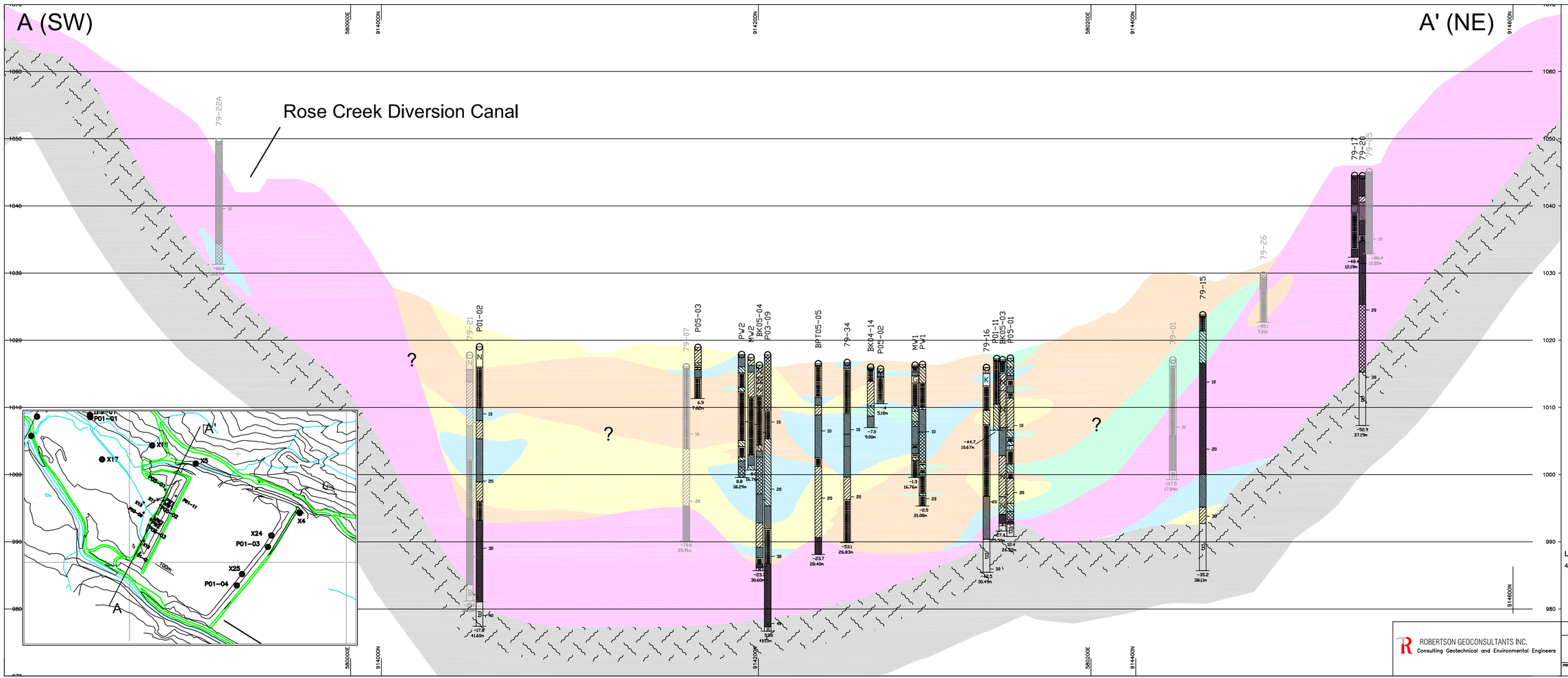
Layout Plan of
Rose Creek Tailiings Facility

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		2.1

A (SW)

A' (NE)

Rose Creek Diversion Canal



LEGEND

- G — Gravel
- Sandy Gravel
- Silt, Sand, and Gravel
- Sand
- Silt and Sand
- Silt
- Till (usually SS)
- B — Bedrock (BR)
- M — Muskeg (MSK)
- N — Not Logged (NL)

- Silt, Silt & Sand, Muskeg
- Sand and Gravel
- Silt, Sand & Gravel
- Sand
- Till
- Bedrock

79-xx — Hole Name

Collar Location
Depth Downhole
Distance Offsection
(+ downstream)
(– upstream)
(at End of Hole)
Length of Hole

Bores >55m
Off Section

Looking Northwest (Downstream)
4x Vertical Exageration

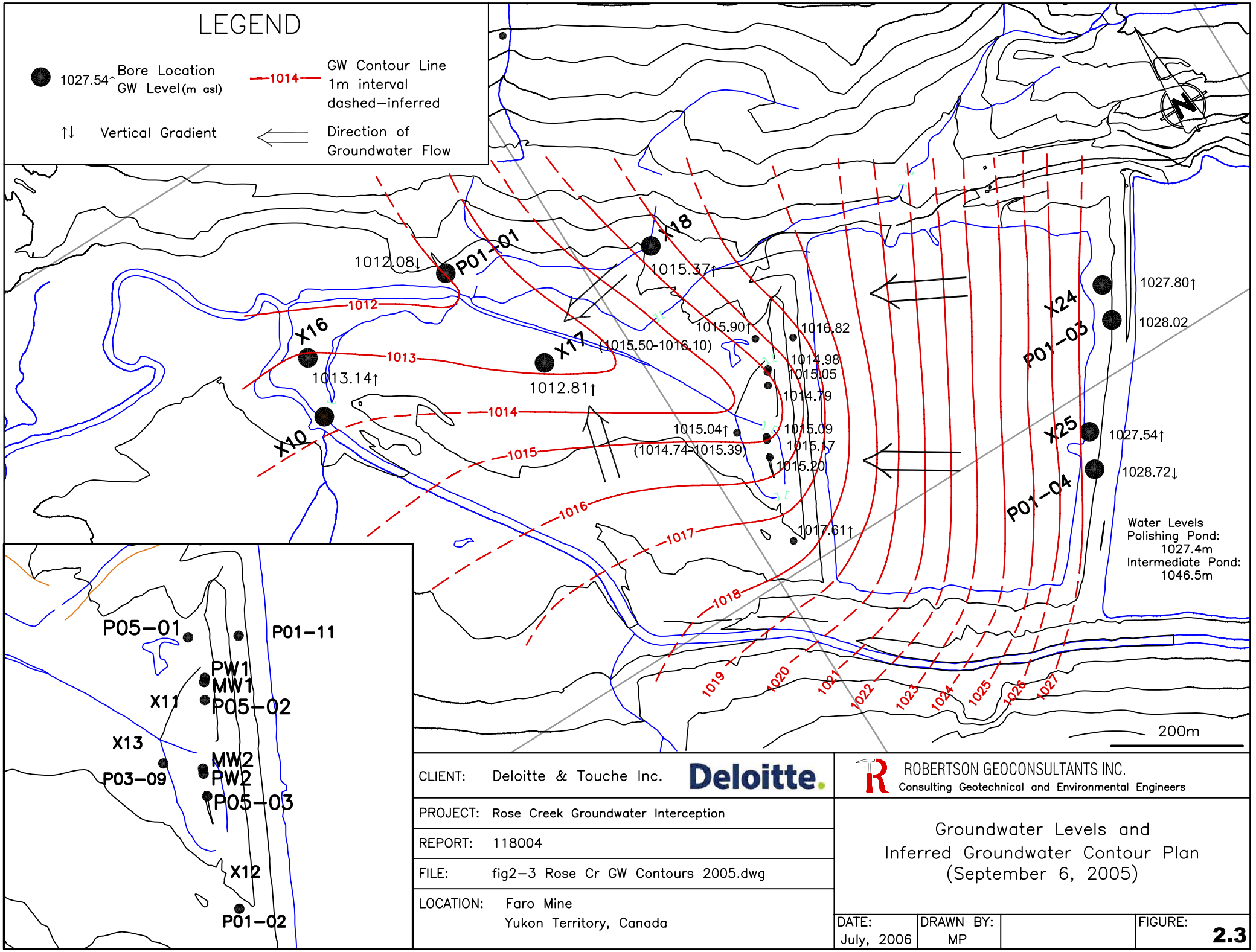
SCALE 1:1000
20 0 20 40
METRES

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FARO MINE SITE

Geological Cross Section
at the toe of Cross Valley Dam

PROJECT NO.	DATE	APPROVED	FIGURE
118004/1	April, 2006		2.2



CLIENT:	Deloitte & Touche Inc.
PROJECT:	Rose Creek Groundwater Interception
REPORT:	118004
FILE:	fig2–3 Rose Cr GW Contours 2005.dwg
LOCATION:	Faro Mine Yukon Territory, Canada

ROBERTSON GEOCONSULTANTS INC.
Consulting Geotechnical and Environmental Engineers

Groundwater Levels and Inferred Groundwater Contour Plan (September 6, 2005)

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		2.3

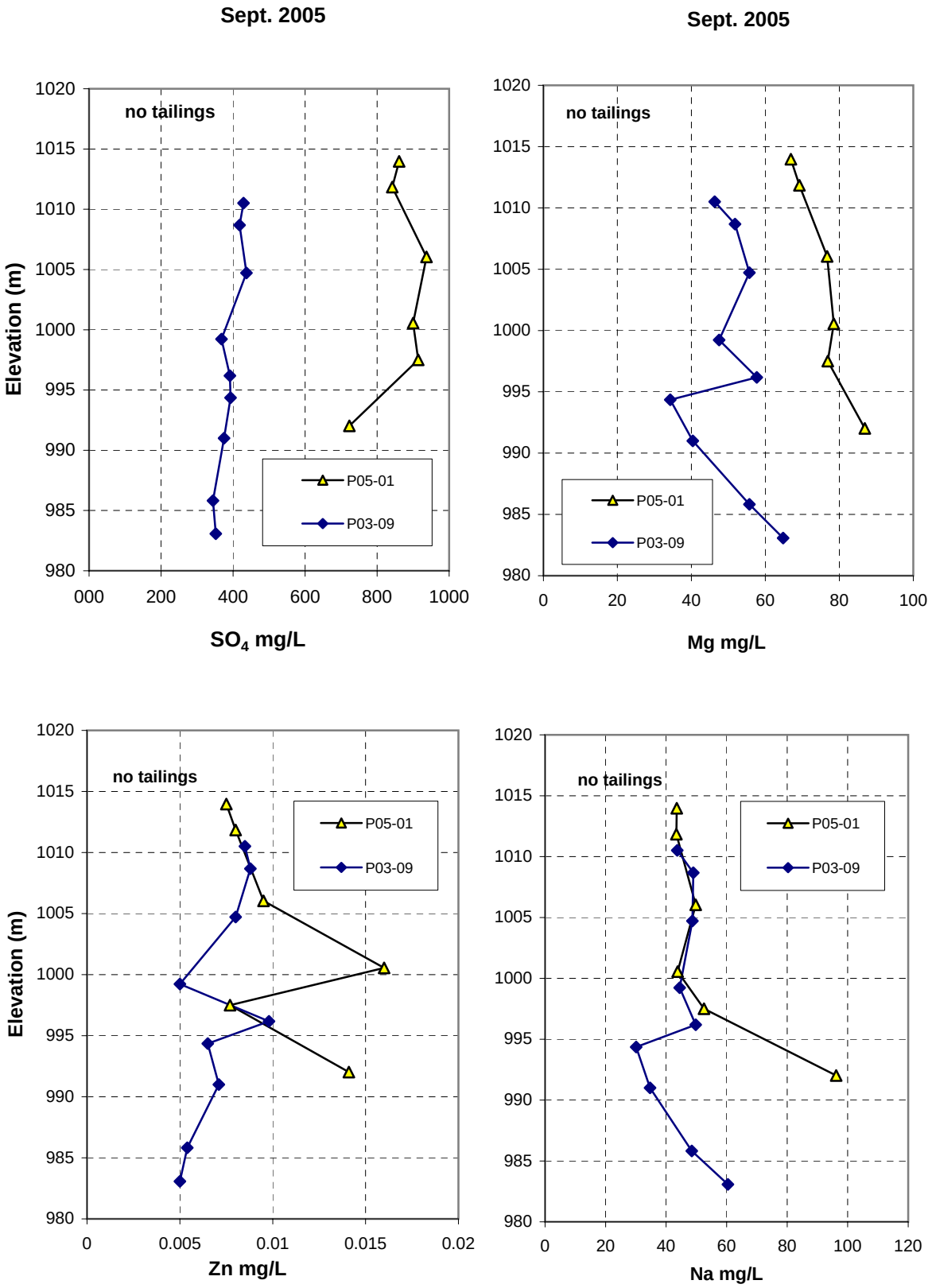


Figure 2.4 Water Quality Depth Profiles for SO_4, Mg, Zn and Na at P05-01 and P03-09.

SW

NE

1030
1020
1010
1000
990
980

79-21
P01-02
79-07
P05-03
PW2
MW2
BK05-04
P03-09
BPT05-05
79-34
BK04-14
P05-02
MW1
PW1
79-16
P01-11
BK05-03
P05-01

127
92.3
470
429
418
437
368
391
393
375
344
352
895
971
904
971
861
842
937
900
914
723
?
?

100
200
300
400
500
600
700
800
900

4x Vertical Exageration

50m

914200N

LEGEND

Borehole Trace

127 Screen Interval & SO_4 Value (mg/L)

700 600 500 SO_4 contour interval (100 mg/L)

CLIENT:	Deloitte & Touche Inc.
PROJECT:	Rose Creek Groundwater Interception
REPORT:	118004
FILE:	fig2-5 cross section geochem.dwg
LOCATION:	Faro Mine Yukon Territory, Canada

ROBERTSON GEOCONSULTANTS INC.
Consulting Geotechnical and Environmental Engineers

Spatial Distribution of SO4 in Rose Creek Aquifer at Cross Valley Dam

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		2.5a

SW

NE

1030
1020
1010
1000
990
980

79-21
P01-02
79-07
P05-03
PW2
MW2
BK05-04
P03-09
BPT05-05
79-34
BK04-14
P05-02
MW1
PW1
79-16
P01-11
BK05-03
P05-01

0.0081
0.0087
0.0083
0.0085
0.0088
0.0080
<0.0050
0.0098
0.0065
0.0071
0.0054
<0.0050
0.0264
0.0205
0.0074
0.0205
0.02
0.01
0.0075
0.0080
0.0095
0.0160
0.0077
0.0141
?
?

6.9
7.62m
-.4
5.18m
-7.0
9.00m
-44.7
10.67m
8.8
18.29m
9.8
16.76m
-1.3
16.76m
-2.5
21.08m
-76.6
25.91m
-23.7
28.40m
-53.1
26.83m
-23.7
30.60m
27.4
25.50m
12.4
26.32m
-48.5
30.49m
-59.0
36.58m
-17.8
41.60m
52.0
41.15m

4x Vertical Exaggeration

50m

914200N

LEGEND

Borehole Trace

0.0087 Screen Interval & Zn Value (mg/L)

0.01 Zn contour interval (0.01 mg/L)

CLIENT: Deloitte & Touche Inc.

PROJECT: Rose Creek Groundwater Interception

REPORT: 118004

FILE: fig2-5 cross section geochem.dwg

LOCATION: Faro Mine
Yukon Territory, Canada

ROBERTSON GEOCONSULTANTS INC.
Consulting Geotechnical and Environmental Engineers

Spatial Distribution of Zn in Rose Creek Aquifer at Cross Valley Dam

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		2.5b

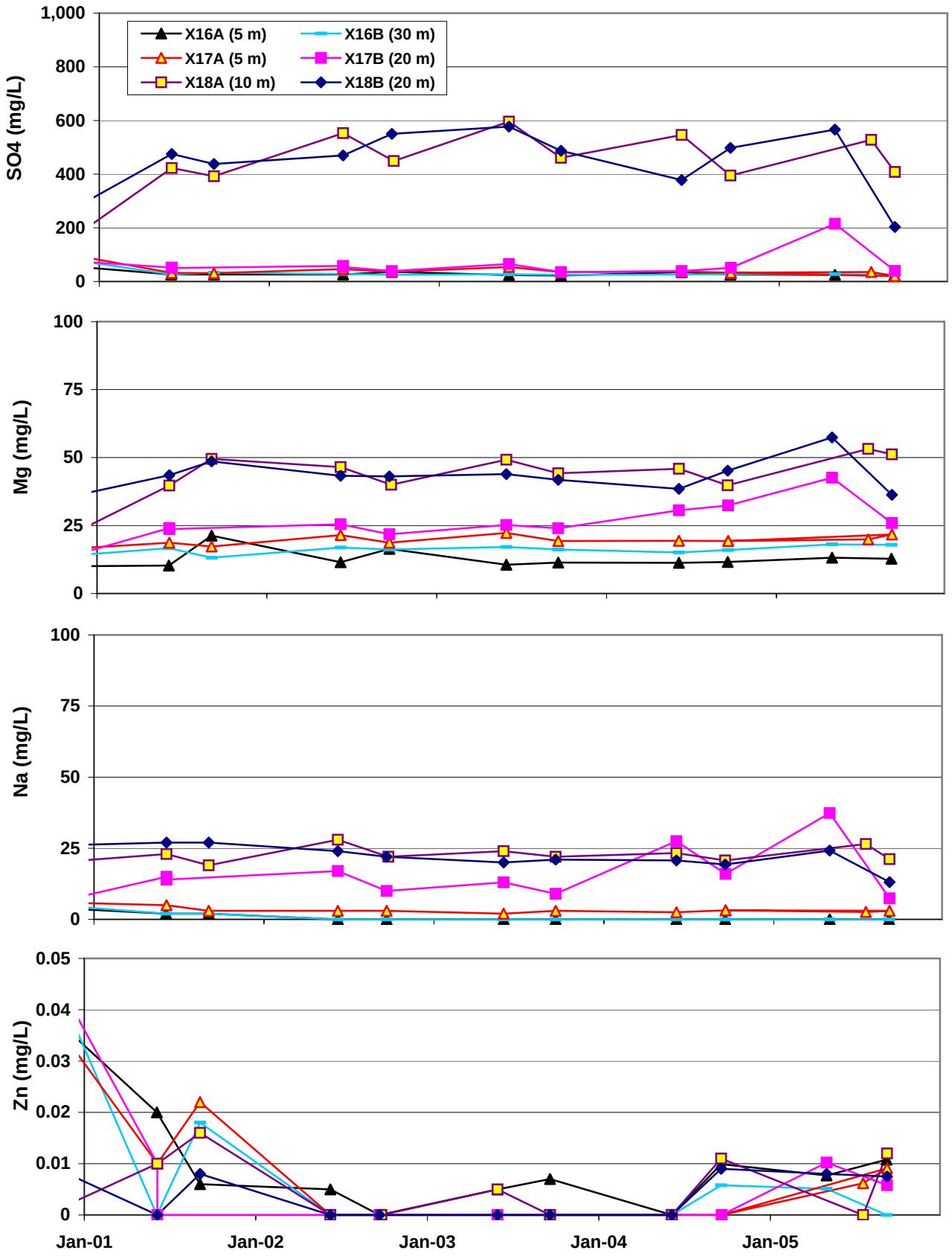


Figure 2.6 Water quality in X16, X17 and X18 (downstream of Cross Valley Dam).

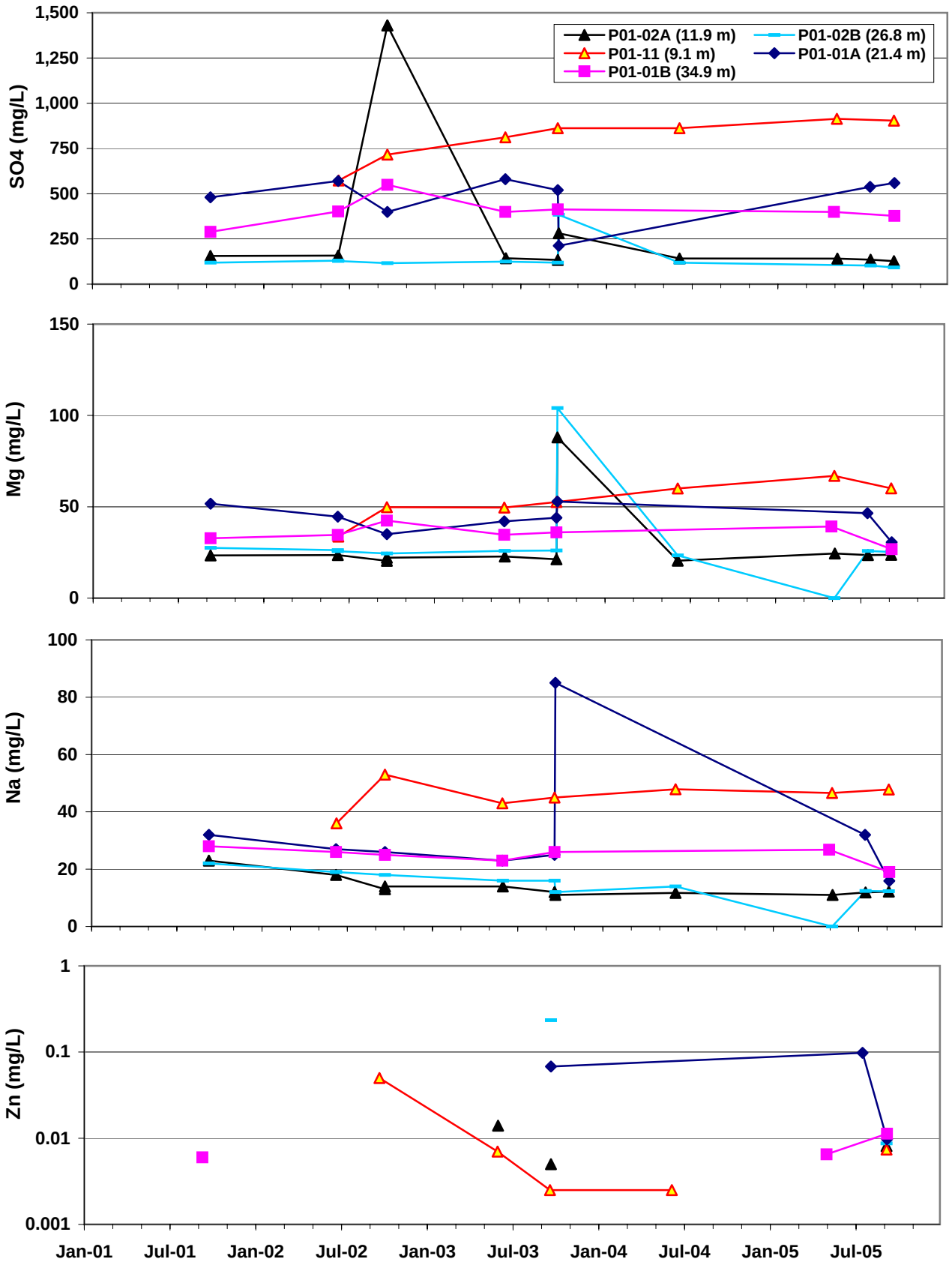


Figure 2.7 Water quality in P01-01, P01-02 and P01-11 (Cross Valley Dam).

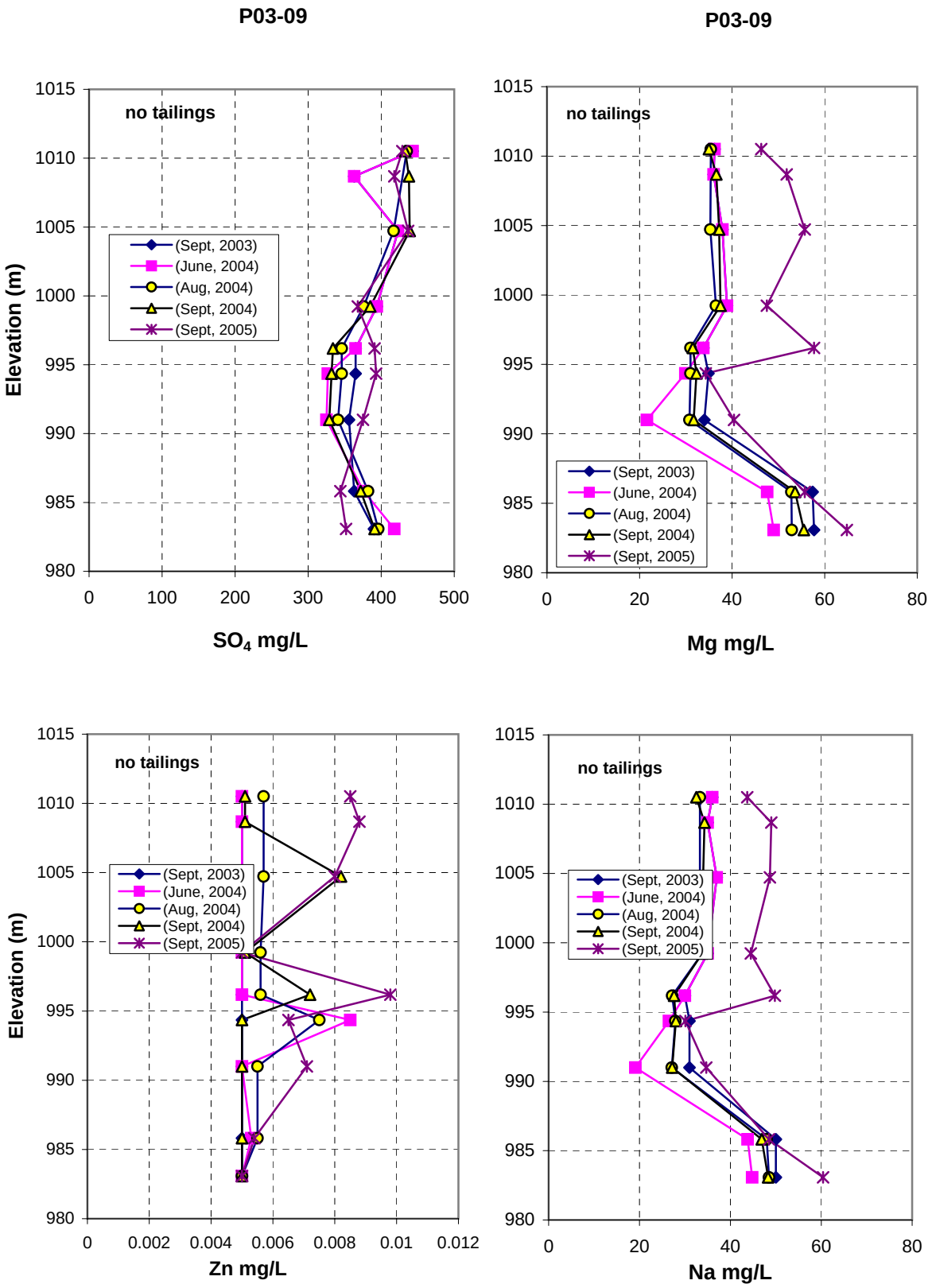


Figure 2.8 Water quality depth profiles of SO_4, Mg, Zn and Na at P03-09.

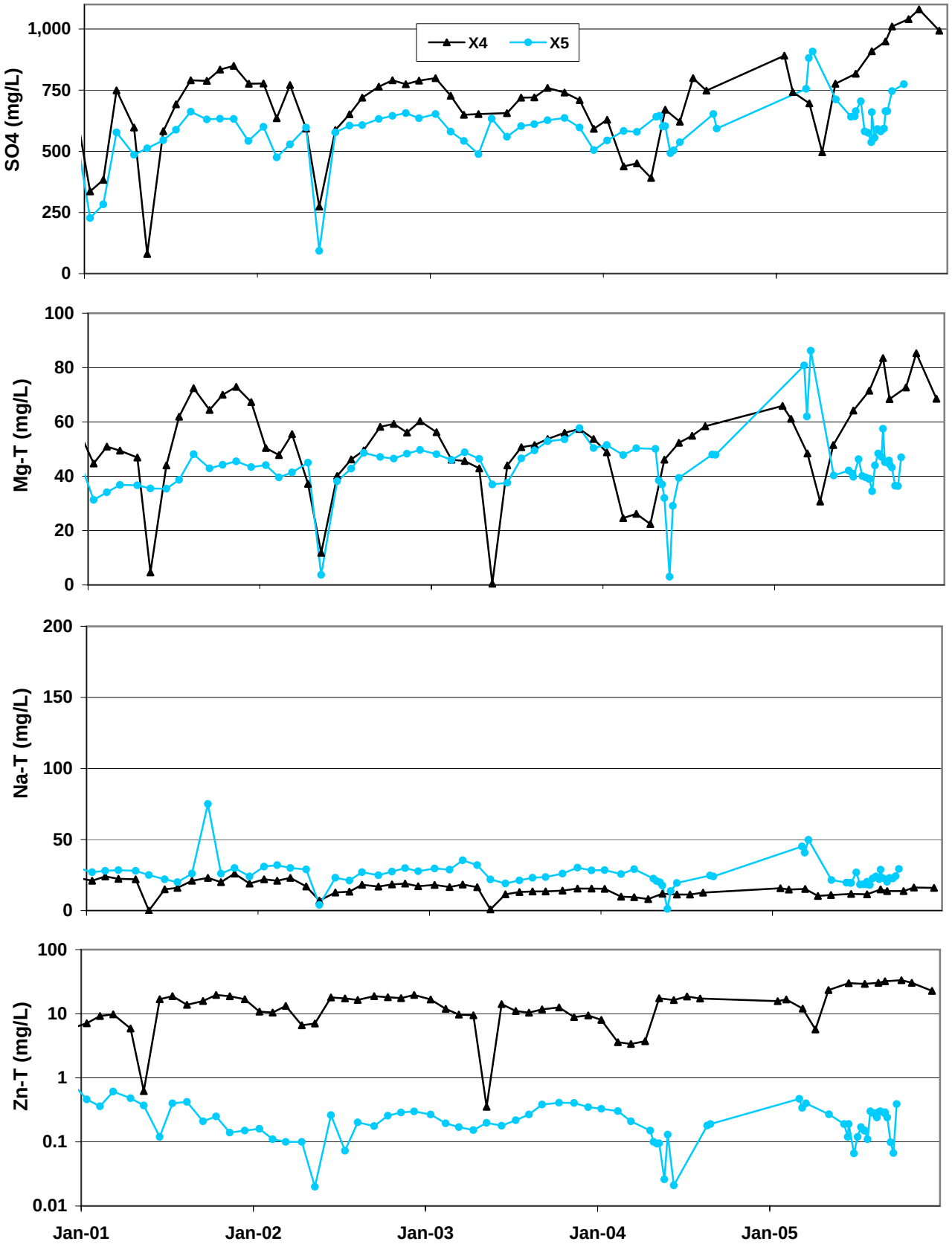


Figure 2.9 Surface water quality at the Intermediate Pond decant (X4) and Cross Valley Pond outflow (X5).

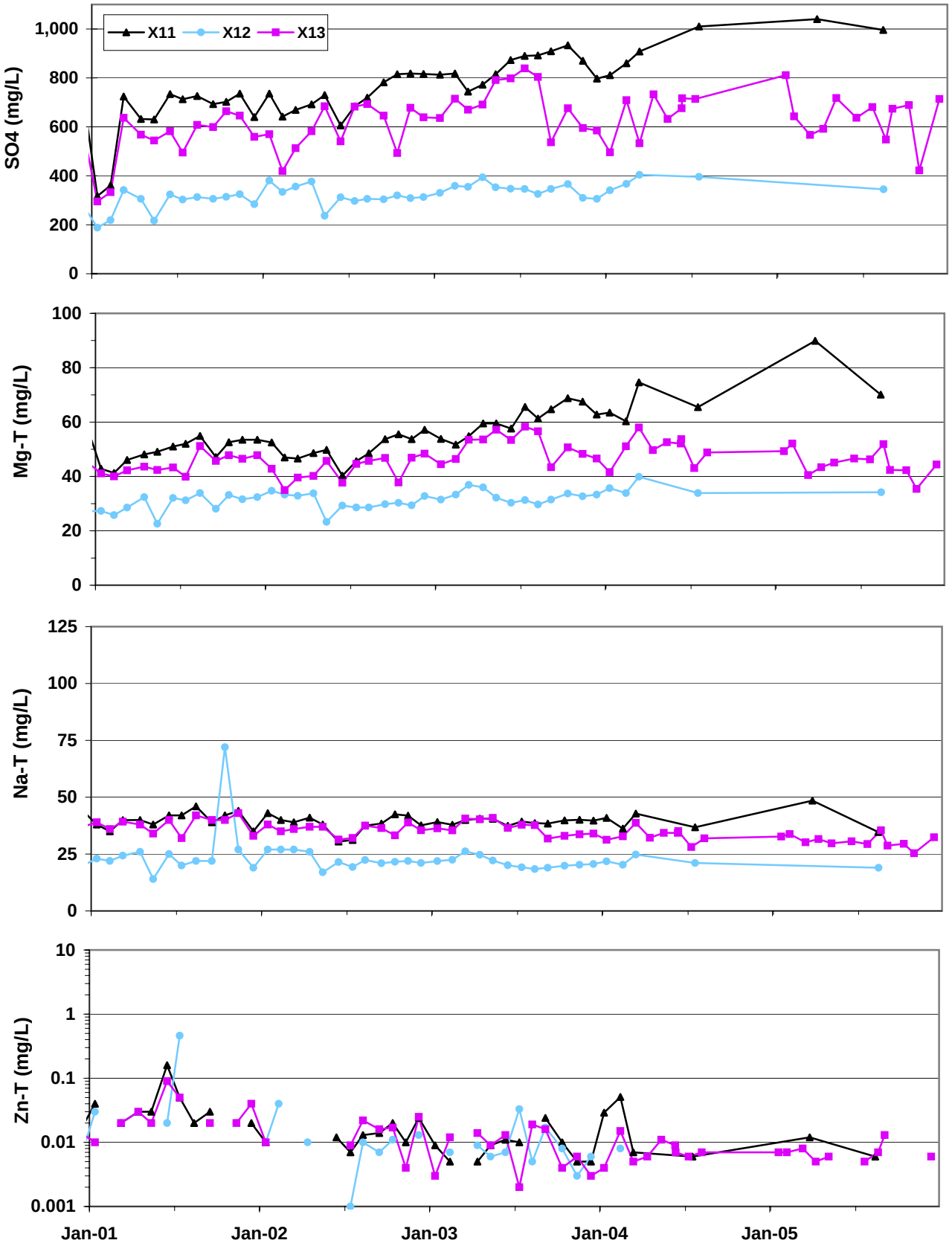


Figure 2.10 Water quality of seepage from the Cross Valley Dam North toe (X11), south toe (X12), and combined seepage (X13).

3 PUMPING TEST PROGRAM

3.1 Methods

A series of pumping tests were carried out between September 6 and September 20, 2005 in the Rose Creek aquifer immediately downgradient of the Cross Valley Dam utilizing the new pumping wells PW1 and PW2.

The pumping tests were performed by staff from Precision Services & Pumps (PSP), Abbotsford, B.C. under the supervision of Laura Findlater from Robertson GeoConsultants Inc.. All pumping tests were carried out using a 6" diameter 3-stage Dempster 25 HP submersible pump (Photo 13). The discharge was routed through a well head assembly (including a digital flow meter, a pressure gauge and a sampling port) (Photos 14-15) and into a 600ft long layflat discharge line. The groundwater was discharged about 25m downstream of X13 in the seepage channel (Photos 16). A geotextile was laid out at the discharge point to minimize erosion in the seepage channel during the tests. The flow rates during the pumping tests were measured using an orifice setup at the end of the discharge line (Photo 17). A manometer was attached to a steel pipe with a face plate and the head readings in the manometer were converted to flow rates using a rating curve for this face plate.

Table 3.1 summarizes the various pumping tests carried out. At both pumping wells, step tests were first carried out to determine the appropriate pumping rate to be used for the subsequent constant discharge (CD) test. Based on these tests, a pumping rate of 500 USGPM (31.5 L/s) was selected for PW1 and a pumping rate of 600 USGPM (37.8 L/s) was selected for PW2.

At PW1, the work plan had called for a 7-day CD test. However, the pump failed after about 56 hours (~2.3 days). Subsequent inspection of the pump indicated that an impeller had jammed due to some coarse sediment (platy texture) that had entered the well screen. The observed sediment intake during the late stages of the PW1 test is very likely due to caving after almost complete desaturation of the upper well screen at that time. The problem with the pump was noted immediately and the pump was turned off within about five minutes. Subsequently, water level recovery was monitored in the pumping well and surrounding monitoring wells until recovery had exceeded 95%. The very short-lived drop in the pumping rate prior to the end of the pumping test is not believed to have an influence on the interpretation of the recovery data (see below).

After full recovery, the pump was replaced (with an equivalent model) and an additional, extended step test carried out to test the well performance and aquifer response at reduced pumping rates. After completion of the first step (2 days at 90 USGPM) the pumping rate was ramped up to the second step (2 days at 195 USGPM) without prior recovery. Note that these step tests did not stress the entire aquifer as much as the earlier 500 USGPM CD test and were therefore not used for interpretation of aquifer properties. However, these step tests provided empirical evidence of the performance of the pumping wells under (reduced) pumping conditions more likely to be used for a seepage interception system (which would use a fence of extraction wells rather than a single well).

At PW2, no problems were encountered during the constant discharge test, and the test was terminated after 24 hours as planned. Again, the recovery in water levels was monitored in the pumping well and the surrounding monitoring wells until the recovery had reached at least 90%.

For a given pumping test, the desired pumping rate was initially obtained using a regulator valve at the well head. During a test, the pumping rate was monitored frequently to ensure a constant discharge. For most of the tests, the pumping rate remained nearly constant requiring only minimal adjustments. Summary sheets documenting field observations of pump operation (including pumping rates, flow adjustments, sediment intake etc) and well response (drawdown) are reproduced in Appendix D.

Table 3.1

Summary of Pumping Tests at Cross Valley Dam

Pumping Well	Type of Test	Date & Time	Duration	Pumping Rate (USGPM)
PW1	Step Test	September 9, 2005 Start: 11:01 am Stop: 17:41 pm	Step 1: 100 min Step 2: 100 min Step 3::100 min Step 4: 100 min	110 269 506 584
	Constant Discharge (w/ recovery)	Start: Sept-10 11:26 am Stop: Sept-12 19:13 pm	56 hours	498
	Step Test (w/ final recovery)	Start: Sept-14 11:51 am Start: Sept-16 12:51 pm Stop: Sept-18 11:46 am	Step 1: 49 hours Step 2::47 hours	90 195
PW2	Step Test	September 6, 2005 Start: 16:36 pm Stop: 20:36 pm	Step 1: 60 min Step 2: 60 min Step 3::60 min Step 4: 60 min	113 258 506 530
	Constant Discharge (w/ recovery)	Start: Sept-7 10:36 am Stop: Sept-8 10:36 am	24 hours	597

Extensive water level monitoring was carried out prior to, during and following the various pumping tests to determine the response of the Rose Creek aquifer to pumping. On September 6, 2005, i.e. prior to all pumping tests, a complete survey of “static” water levels was carried out using all monitoring wells located in the reach between the Intermediate Dam and Rose Creek (see Figure 2.3). For the individual tests, water levels in the pumping well itself and the two nearest monitoring wells (at ~5m and ~25m distance) were monitored using pressure transducers with on-board dataloggers (Solinst Leveloggers M10 and M20) to monitor the detailed drawdown and recovery curves. In addition, water levels were monitored frequently in all monitoring wells in the area (including the pumping well).

A comparison of manual and continuous (logged) water levels indicated small, systematic deviations between those two types of measurements (up to 10 cm). These discrepancies are believed to be a result of changes in the barometric pressure throughout the testing period which are not corrected for in the (non-vented) transducers. Originally, it had been planned to correct the pressure transducer readings using a Solinst Barologger. However, this logger malfunctioned during the test; hence no detailed barometric readings were available for correction. Where required, the manual water level readings were therefore used for analysis and interpretation of the pumping tests.

In addition to water level readings, seepage rates at the toe of the Cross Valley Dam were monitored frequently. For this purpose, staff gauge readings were taken at the various weirs located downstream of the Cross Valley Dam (including stations X11, X12, X13 and weir 3). Staff gauge readings were then converted to flow rates using the updated staff gauge rating curves calibrated a few days earlier by Laberge Environmental Services (LES, 2005).

All monitoring data collected during the various pumping tests (manual readings, staff gauge readings etc.) are provided in EXCEL format in Appendix D (electronic version only).

The groundwater pumped at PW1 and PW2 was sampled four times during their respective CD tests using a sampling port at the well head assembly (Photos 14 and 15). At PW1, samples were collected 1, 17, 24 and 47 hrs into the CD test. At PW2, samples were collected 1, 9, 16 and 24 hrs into the CD test. Field preservation and handling of these samples is described in section 2.3.2.

3.2 Results and Discussion

3.2.1 Step Test Analysis

The analyses of the initial step tests in the pumping wells PW1 and PW2 are summarized in Appendix E. Briefly, the Hantush-Bierschenk method was applied to the step test data to determine linear and non-linear well losses and well efficiency (Kruseman and DeRidder, 1994). The results of this analysis suggested that both pumping wells are very efficient (83% for PW1 and ~100% for PW2), with non-linear well losses much smaller than linear well losses. The high efficiency of these pumping wells is a result of the use of efficient well screens (continuous slot wire wrap), natural filter pack (in coarse sediments) and extensive development of these wells.

3.2.2 Background Trends in Groundwater Levels during Testing Period

Groundwater levels in all monitoring wells, including those located at greater distance from the active pumping well, were monitored throughout the 2-week pumping testing period to determine “natural” time trends in groundwater levels unrelated to the pumping tests. Figure 3.1 shows the observed groundwater level time trends at the toe of the Intermediate Dam. The change in the water level in the Intermediate Pond and the Polishing Pond are also shown for comparison. The groundwater levels at the toe of the Intermediate Dam increased uniformly by about 0.35m between September 6 and 19, 2005 and started to decline thereafter. These time trends are nearly identical to the observed changes in the water level in the Polishing Pond (Figure 3.1). In contrast, the water level in the Intermediate Pond showed a uniform decline in water levels (of about 0.4m) throughout this period.

The changes in pond water levels were caused by increased pumping of water from the Intermediate Pond to the Polishing Pond to reduce the water level in the Intermediate Pond in preparation of the upcoming winter shut-down (Dana Haggar, pers. comm.). These time trends conclusively show that

the groundwater levels at the toe of the Intermediate Dam are controlled by the water level in the Polishing Pond, not the water level in the Intermediate Pond. This observation is consistent with the large difference in absolute (geodetic) water level in the Intermediate Pond and in the underlying aquifer at the toe of the Intermediate Dam (Figure 2.3).

Figure 3.2 shows the observed groundwater level time trends in selected monitoring wells located at the toe of the Cross Valley Dam (P01-2A/B; P01-11 and P05-03) and at greater distance (downgradient) in the Rose Creek valley (X16A/B, X17A/B and X18A/B). The vertical lines indicate the different pumping test periods. As expected, the groundwater levels at the toe of the Cross Valley Dam (CVD) clearly responded to the pumping tests (see sections 3.2.3 and 3.2.4 below for more details). However, these groundwater levels were not affected by the rise in the Polishing Pond level, as demonstrated by the almost complete recovery in most wells to the pre-test levels (September 6 2005) following the final recovery on September 18-19. It is therefore concluded that groundwater levels at the toe of the Cross Valley Dam are not influenced significantly by the water level in the Polishing Pond and did not require any correction for pumping test interpretation.

The more distant monitoring wells (X16, X17 and X18) showed different time trends in groundwater levels. Groundwater levels in X18A/B, located about 250m downstream of the CVD on the north side, showed a small, but significant decrease in groundwater levels after start of pumping in PW1. This decrease is tentatively interpreted to be a result of pumping at PW1. Groundwater levels at X17A/B, located about 400m downstream of the Cross Valley Dam in the center of the valley, did not show any changes throughout the observation period. Similar constant water levels were observed at P01-01A/B located further downgradient (not shown), suggesting that the Rose Creek aquifer downgradient of the immediate vicinity of the Cross Valley Dam (and the influence of the pumping tests) was at steady-state throughout the testing period.

Groundwater levels at X16A/B, located about 800m downstream of the Cross Valley Dam (near the Rose Creek Diversion Canal) showed significantly more variation over time than at X17A/B. The observed time trends at X16A/B are possibly related to the fluctuations in the stage of the near-by RCDC and/or Rose Creek itself. For example, rainfall was observed during the night of September 13/14 coinciding with the main peak in the well hydrograph. The very steady water levels observed at X17A/B (and P01-01) support the conclusion that these water level variations are not caused by the pumping tests at the Cross Valley Dam.

3.2.3 Observed Drawdown and Radius of Influence

3.2.3.1 Test Response at PW1

Figures 3.3 and 3.4 show the drawdown and recovery in the pumping well and selected monitoring wells in response to the 56-hour CD discharge test in PW1 at 500 USGPM (31.5 L/s). Figure 3.5 shows the maximum drawdown and inferred cone of depression in the Rose Creek aquifer at the end of the test (after 56 hours). Figure 3.6 shows the observed flow rates at the seepage monitoring stations throughout the test. The results of this test can be summarized as follows:

- The pumping rate of 500 USGPM produced a maximum drawdown of 4.8m in the pumping well, i.e. completely desaturating the upper well screen;

- The radius of influence of this pumping test extended across the entire width of the Rose Creek aquifer with 0.12-0.17m drawdown observed at the south end of the Cross Valley Dam (P01-02A/B);
- Pumping at PW1 resulted in significant drawdown in all layers of the Rose Creek aquifer, including the underlying bedrock on the north side (port 1 at P05-01) and the underlying till screened in the south-central area (ports 1 and 2 at P03-09)[1];
- Monitoring wells located at the toe of the Intermediate Dam did not show any response to pumping suggesting limited drawdown in the upstream direction;
- The total seepage at the toe of the Cross Valley Dam (at X13) decreased by about 57%; as expected, the seepage reduction was significantly greater on the northern side (78% at X11), i.e. in the area where the pumping test was carried out.

Figures 3.7 and 3.8 show the drawdown and recovery in the pumping well and selected monitoring wells in response to a 4-day step test in PW1, carried out at reduced pumping rates (90 and 195 USGPM, respectively). Figures 3.9 and 3.10 show the maximum drawdown and inferred cone of depression for each step. The results of these tests can be summarized as follows:

- The maximum drawdown in the pumping well PW1 was 0.49m for 90 USGPM (after 48 hours) and 1.05m for 195 USGPM (after 96 hours); the drawdown in the pumping well had not yet reached steady-state at the end of the test (after 4 days);
- As expected, the radius of influence of the pumping well is a function of the pumping rate:
    - At 90 USGPM, the inferred radius of influence for 0.1m and 0.05m drawdown is about 75m and 45m, respectively;
    - At 195 USGPM, the inferred radius of influence for 0.1m and 0.05m drawdown is about 180m and 90m, respectively;
- A pumping rate of 195 USGPM still maintains some drawdown (>0.05m) throughout the aquifer and underlying bedrock at 50m distance (in P05-09);
- A pumping rate of 90 USGPM still maintains some drawdown (>0.05m) throughout the aquifer at 50m distance (in P05-09), but drawdown in the underlying bedrock is marginal (~0.02 m);

3.2.3.2 Test Response at PW2

Figures 3.11 and 3.12 show the drawdown and recovery in selected monitoring wells in response to the 24-hour CD discharge test in PW2 at 600 USGPM (37.7 L/s). Figure 3.13 shows the maximum drawdown and inferred cone of depression in the Rose Creek aquifer at the end of the test (after 24 hours). Figure 3.14 shows the observed flow rates at the seepage monitoring stations throughout the test. The results of this test can be summarized as follows:

- The pumping rate of 600 USGPM only produced a maximum drawdown of 2.1m in the pumping well, i.e. desaturating only the top 0.6m of the well screen;

[1] Note that the deeper ports screened in till at P03-09 showed a greater drawdown than the shallower ports screened in S&G; this would suggest that the upper layers of the aquifer are significantly influenced by leakage (likely by seepage from the Rose Creek Diversion)

- The radius of influence of this pumping test extended across the entire width of the Rose Creek aquifer with 0.15m drawdown observed at the north end of the Cross Valley Dam (P01-11);
- Pumping at PW2 resulted in significant drawdown in all layers of the Rose Creek aquifer, including the underlying till in the south-central area (ports 1 and 2 at P03-09) and the underlying bedrock on the north side (port 1 at P05-01);
- Monitoring wells located at the downstream toe of the Intermediate Dam did not show any response to pumping suggesting limited drawdown in the upstream direction.

In summary, both pumping wells installed at the toe of the Cross Valley Dam are screened in highly productive sand & gravel layers and are capable of pumping significant quantities of groundwater. Groundwater yields in the south-central portion of the aquifer are generally higher than on the north side, sustaining pumping rates in excess of 600 USGPM compared to an estimated 300-400 USGPM on the north side. The pumping tests clearly indicated that groundwater pumping from extraction wells is a viable control strategy for intercepting contaminated groundwater in the Rose Creek Valley.

3.2.4 Analytical Interpretation of Pumping Tests

The pumping test data were first interpreted using analytical solutions to obtain initial estimates of transmissivity (T) and storage coefficient (S) for the Rose Creek aquifer. For the purpose of this exercise, the drawdown data were analysed using the Cooper-Jacob method and the recovery data were analysed using the Theis method. Both analytical solutions assume a confined aquifer with "ideal properties" (infinite extent, no leakage). It is recognized that these conditions are not strictly met for the Rose Creek aquifer. Nevertheless, these analyses provided initial estimates of aquifer properties and thus guidance for subsequent calibration of a groundwater flow model for the area (see section 4).

3.2.4.1 Constant Discharge Test at PW1

Figure 3.15 shows the "normalized" drawdown at the pumping well and various monitoring wells for the 56 hour CD test at PW1 (500 USGPM). In this plot, the drawdown is plotted against t/r^2 where t equals the time since pumping started and r equals the distance from the pumping well, to account for the reduced drawdown in monitoring wells located at a greater distance from the pumping well. In an ideal aquifer, all drawdown data would fall on a straight line using this normalized drawdown plot (Kruseman & de Ridder, 1994).

The large scatter in the normalized drawdown plot clearly indicates that the Rose Creek aquifer is not an "ideal aquifer", likely due to heterogeneity within the aquifer and/or leakage from semi-confining layers. An inspection of Figure 3.15 indicates two types of pumping test responses (i) a steeper drawdown in the vicinity of the pumping well (say < 50m radius) and (ii) a reduced rate of drawdown in more distant wells (>> 50m radius). A similar pattern was observed during the pumping test at PW2 (see below). Because of these non-ideal aquifer conditions only the drawdown in the pumping well and the nearest monitoring wells was interpreted using analytical solutions. The drawdown in the more distant monitoring wells was analysed during subsequent transient calibration of the groundwater flow model (see section 4).

Table 3.2 summarizes the estimated T values (in m^2/day) and S values (m/m) for the pumping well PW1 and the two nearest monitoring wells. The best fit analysis to the individual drawdown and recovery data are summarized in Appendix F1. The drawdown data showed a distinct change in slope (increase in drawdown) after about 1,400 min (Appendix F1). This increase in drawdown is likely caused by the desaturation of the upper screen (about 70% desaturated at this time). At later times, the majority of water was likely extracted from the deeper screen, causing an increase in the rate of drawdown. The "early" drawdown data are therefore believed to be more representative of the entire aquifer system. The Cooper-Jacob analysis of the early drawdown data suggests an average transmissivity of ~1,000m^2/day in vicinity of PW1. Assuming an average aquifer thickness of about 25m (near PW1), this T estimate would represent an effective hydraulic conductivity of ~ $5x10^{-4}$ m/s for the glacio-fluvial sediments in the northern portion of the Rose Creek valley.

Using the early drawdown data, the estimated storage coefficient values ranged from 0.02 to 0.04. These storage coefficients are at the upper end of typical values for confined aquifers and much lower than typical (specific yield) values for unconfined sand & gravel aquifers. The observed intermediate storage values are interpreted to be a composite result of pumping an unconfined upper layer (screened by the upper well screen) and a confined lower layer (screened with the lower well screen). Leakage effects may also influence the interpretation of the storage coefficient.

Table 3.2

Initial estimates of aquifer properties (near PW1) using analytical solutions.

Well ID	Distance from Pumping Well (m)	Drawdown - Cooper and Jacob			Recovery - Theis	
		Early		Late	Early	Late
		Storage Coeff. (unitless)	T (m^2/day)	T (m^2/day)	T (m^2/day)	T (m^2/day)
PW1	0	-	1047	206	1010	379
MW1	4.7	0.044	1023	500	1352	402
P05-02	25	0.021	1037	425	-	369
Average		**0.0326**	**1036**	**377**	**1181**	**383**

Notes:
Pumping Well PW1
Pumping Rate 498 USgpm

The analysis of the recovery data provided similar estimates of transmissivity as those determined from the drawdown data (Table 3.2). The recovery data also showed a clear change in slope when plotted in t/t' space (see Appendix F1) resulting in different T estimates for early and late recovery data (Table 3.2). Again, this non-ideal response is likely a reflection of the heterogeneity in the aquifer, emphasized by the desaturation of the upper well screen during the later stages of the test. A more detailed analysis of the pumping test data (using a groundwater flow model) was required to interpret these non-ideal test responses (see section 4).

3.2.4.2 Constant Discharge Test at PW2

Figure 3.16 shows the "normalized" drawdown at the pumping well and various monitoring wells for the 24 hour CD test at PW2 (600 USGPM). The normalized drawdown plot for the PW2 CD test shows an even greater scatter than the PW1 test response suggesting that the south-central portion of the aquifer is also highly "non-ideal". Again, a steeper drawdown is generally observed within the vicinity of the pumping well compared to at the more distant monitoring wells. This consistent pattern in both tests (with pumping at opposite sides of the aquifer) clearly indicates that the far-field response is significantly influenced by non-ideal behaviour and cannot be easily interpreted using simple analytical solutions. Because of these non-ideal aquifer conditions only the drawdown in the pumping well and the nearest monitoring wells was interpreted using analytical solutions. The drawdown in the more distant monitoring wells was analysed during subsequent transient calibration of the groundwater flow model (see section 4).

Table 3.3 summarizes the estimated T values (in m^2/day) and S values (m/m) for the pumping well PW2 and the two nearest monitoring wells. The best fit analysis to the individual drawdown and recovery data are summarized in Appendix F2. At PW2, the drawdown data showed a fairly uniform slope (in log (t) space) justifying the use of a single best fit line (Appendix F2). The Cooper-Jacob analysis of the drawdown data suggests an average transmissivity of ~1,500m^2/day in vicinity of PW2. Assuming an average aquifer thickness of about 25m (near PW2), this T estimate would represent an effective hydraulic conductivity of ~7x10^{-4} m/s for the glacio-fluvial sediments in the south-central portion of the Rose Creek valley.

Interpretation of the drawdown data for the PW2 test suggests storage coefficient values ranging from 0.0003 to 0.0015. These S values are within the reported range of storage coefficients for confined aquifers, albeit at the upper end of the scale (Freeze and Cherry, 1979). Again, non-ideal aquifer conditions (such as leakage and/or partial desaturation of the confined aquifer) may have contributed to these high storage values.

Table 3.3

Initial estimates of aquifer properties (near PW2) using analytical solutions.

Well ID	Distance from Pumping Well (m)	Drawdown - Cooper and Jacob		Recovery - Theis	
				Early	Late
		Storage Coeff. (unitless)	T (m^2/day)	T (m^2/day)	T (m^2/day)
PW2	0	-	1481	1980	721
MW2	5.6	0.0015	1507	2250	742
P05-03	26	0.0003	1533	2697	579
Average		**0.0009**	**1507**	**2309**	**680**

Notes:
Pump Well PW2
Pump rate (GPM) 597

The recovery data for the PW2 test showed a similar non-ideal response as observed at PW1 with a flatter slope during the early recovery compared to the late recovery (Appendix F2). Again, more in-depth analysis will be required to determine the cause for this non-ideal response. Nevertheless, the estimated T values for the early and late recovery data bracket the estimated T value for the drawdown data and provide a reasonable upper and lower bound for the aquifer transmissivity in vicinity of PW2.

3.3 Initial Loading Estimates

Initial estimates of groundwater flow through the aquifer beneath the Cross Valley Dam and the associated contaminant loads were obtained based on the initial interpretation of the pumping test data. These initial estimates provided initial calibration targets for the groundwater flow model (see section 4).

3.3.1 Estimates of Groundwater Flow

The groundwater flow rate (Q) in the Rose Creek aquifer was estimated using Darcy's Law:

Q = T * aquifer width * hydraulic gradient

This Darcy calculation was carried out for the north and south side of the Rose Creek aquifer which were shown to differ significantly in aquifer transmissivity. In order to bracket the likely range of groundwater flow we used an average T estimate for the early and late recovery, respectively.

Table 3.4 shows the results of these initial groundwater flow calculations. These Darcy calculations suggest that the groundwater flow beneath the Cross Valley Dam may range from 45-147 L/s with an estimated average of 96 L/s. These flow estimates agree very well with earlier groundwater flow estimates for this reach (95.4 - 97.3 L/s at Reach 6) obtained using the water and load balance model (Robertson GeoConsultants Inc., 2006).

3.3.2 Estimates of Contaminant Load

The loading of sulphate and zinc in the Rose Creek aquifer (downstream of the Cross Valley Dam) was estimated as follows:

Load = Q x C

Where Q is the volumetric flux of groundwater and C is the concentration of a given solute. The groundwater pumped from PW1 and PW2 was sampled several times throughout the CD pumping tests and analysed in a certified laboratory. As expected, the groundwater pumped from the north side (PW1) showed about two times higher concentrations of sulphate and zinc than groundwater pumped from the south-central side[2]. Therefore, loading calculations were carried out separately for the north and south side.

[2] Groundwater quality in both pumping wells remained very stable throughout the pump tests

Table 3.4 shows the observed (average) sulphate and zinc concentrations in PW1 and PW2 and the estimated loading in the Rose Creek aquifer. The current sulphate load in groundwater is estimated to be about 1,840 t/yr (±1,000 t/yr) and the zinc load is estimated to be about 30 kg/yr (±15 kg/yr). These loading estimates are somewhat higher than those estimated using the water and load balance model (e.g. sulphate loads in groundwater at reach 6 ranged from 1121 – 1246 t/yr, Robertson GeoConsultants Inc., 2006). One potential reason for this discrepancy could be that the assumed sulphate and zinc concentrations used here (Table 3.4) are not representative of all groundwater, in particular along the south side of the aquifer, where leakage from the RCDC provides a source of dilution (Figure 2.5a/b).

Table 3.4

Initial estimates of groundwater flow and contaminant loading in Rose Creek aquifer (downstream of Cross Valley Dam).

	North Side	South Side	Combined
Transmissivity (m^2/d)[1]			
Late Data	383	680	-
Early Data	1,181	2,309	-
Average	782	1,495	-
Width of Valley (m)	196	244	-
Average Gradient[2]	0.016	0.016	-
Flow (L/s)			
Minimum	14	31	**45**
Maximum	43	104	**147**
Average	28	67	**96**
Sulphate Conc (mg/L)[3]	944	468	**-**
Sulphate Load (tonnes/yr)			
Minimum	413	453	**866**
Maximum	1,275	1,538	**2,813**
Average	844	995	**1,840**
Zinc Conc (mg/L)[3]	0.0176	0.0069	**-**
Zinc Load (tonnes/yr)			
Minimum	0.008	0.007	**0.014**
Maximum	0.024	0.023	**0.046**
Average	0.016	0.015	**0.030**

1. Based on Theis analysis of recovery data from pump tests
2. Gradients calculated from September 6, 2005 static water levels.
3. Average values calculated from 4 samples taken at each pumping well during pump tests.

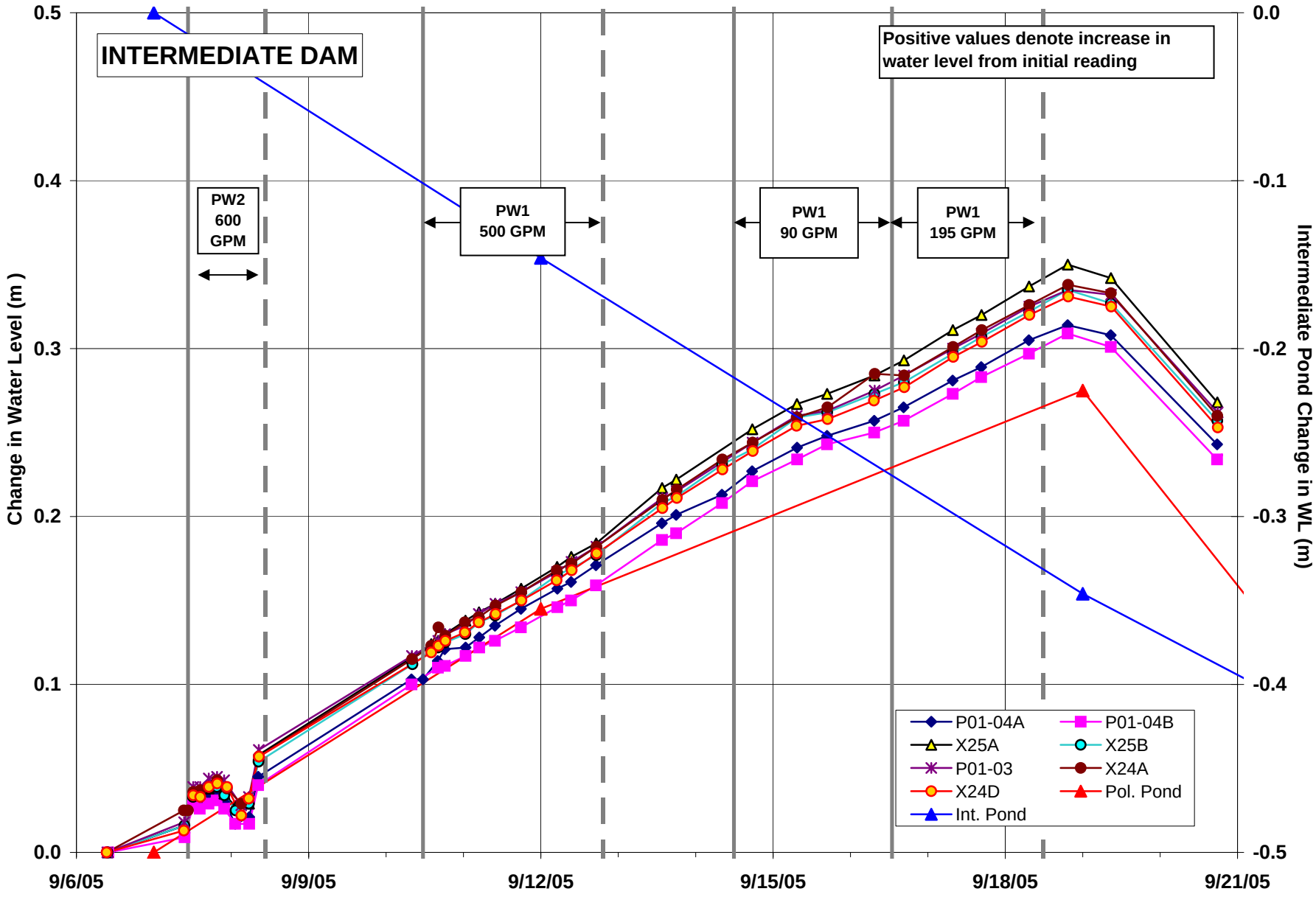


Figure 3.1. Groundater level time trends at toe of Intermediate Dam

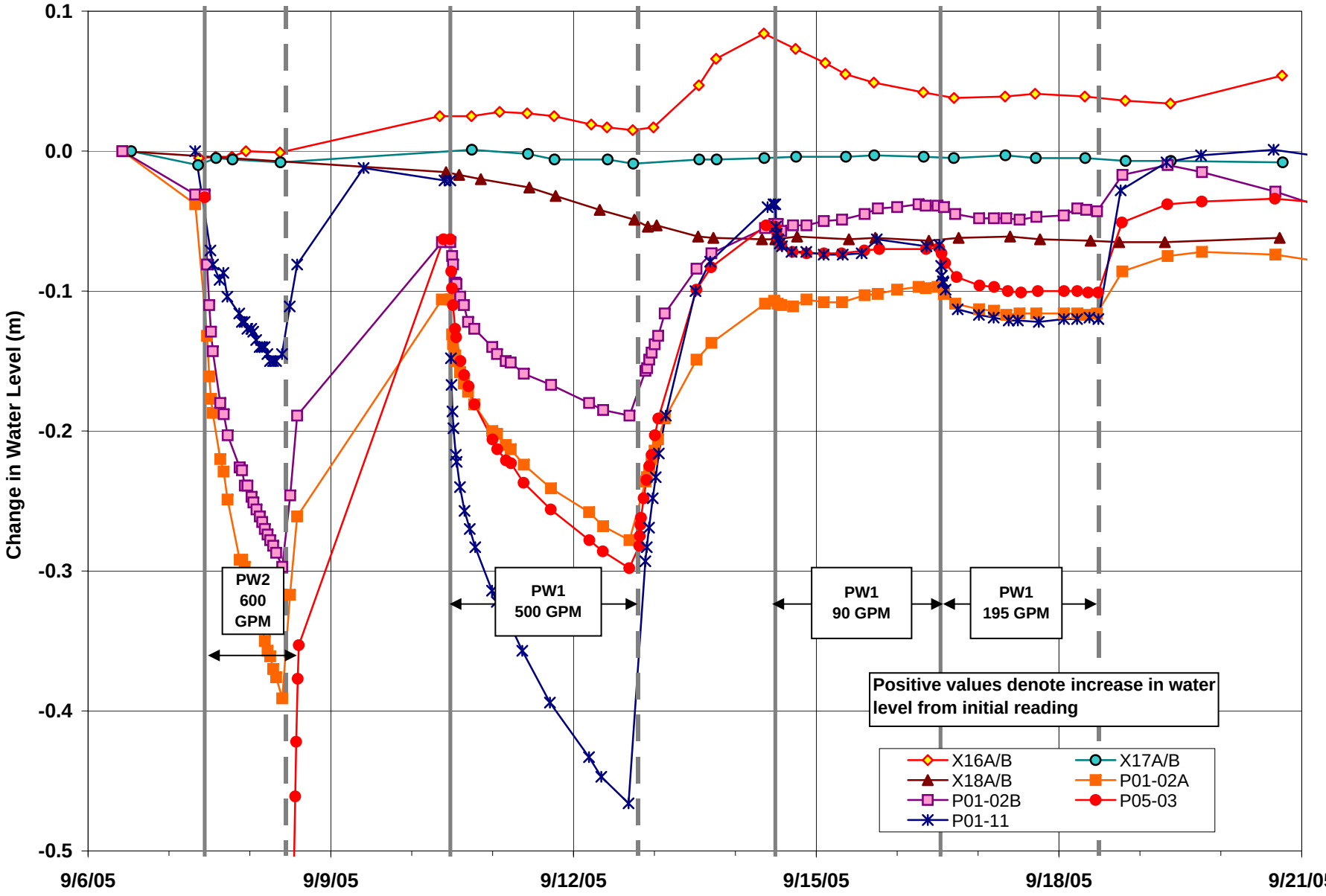


Figure 3.2. Groundwater level time trends downstream of the Cross Valley Dam.

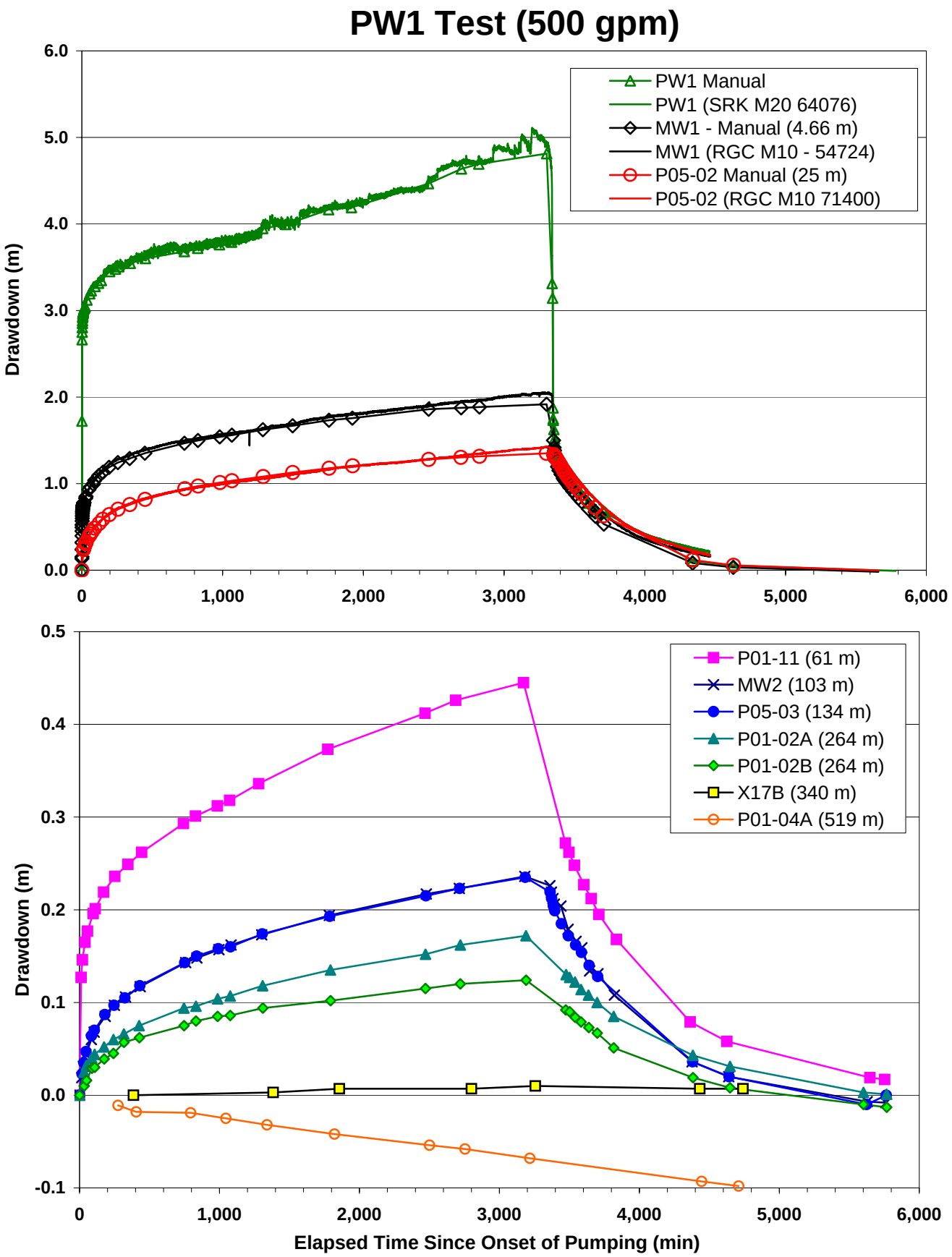


Figure 3.3. Drawdown and recovery observed at conventional wells during 56-hr CD pump test at PW-1 (500-USGPM).

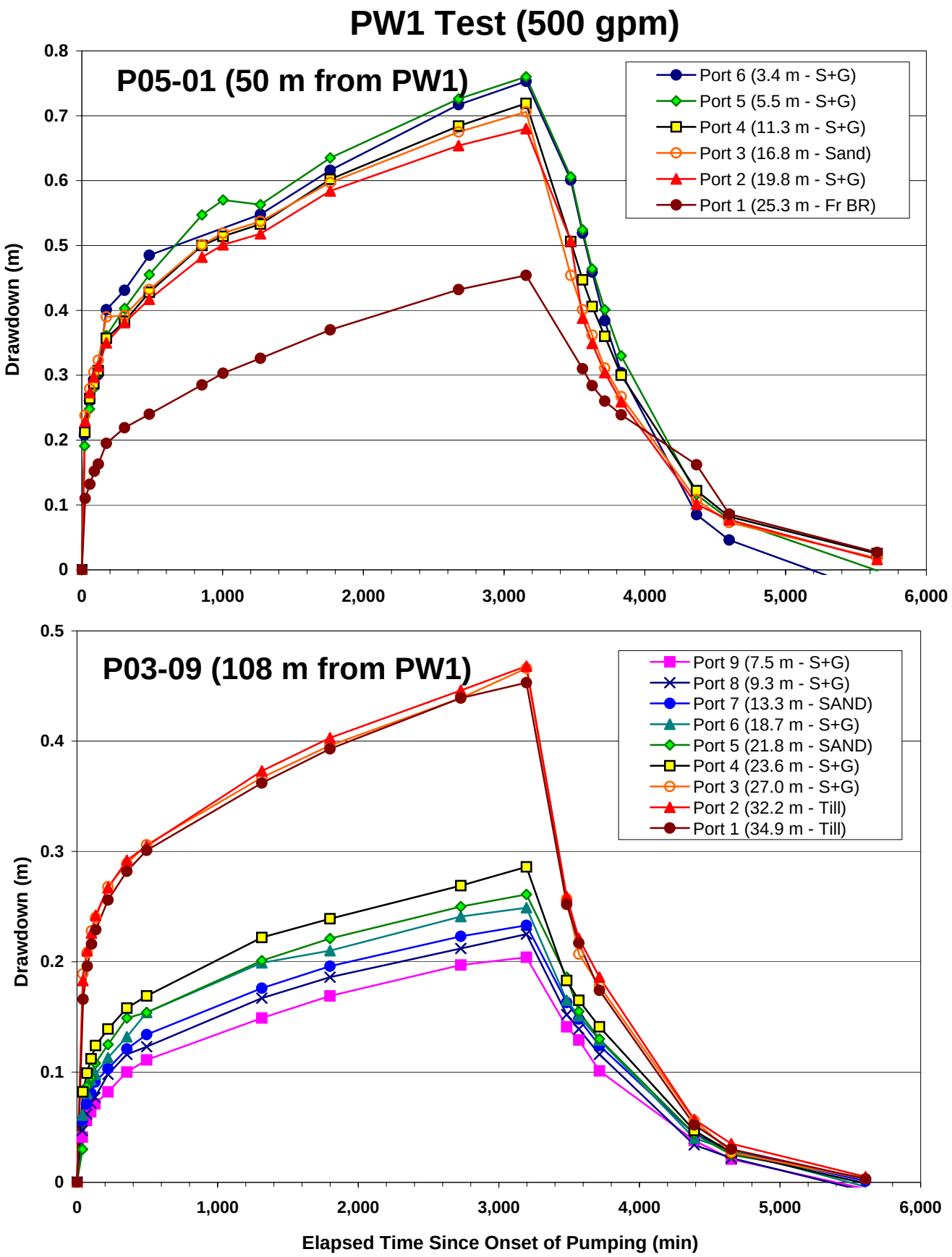
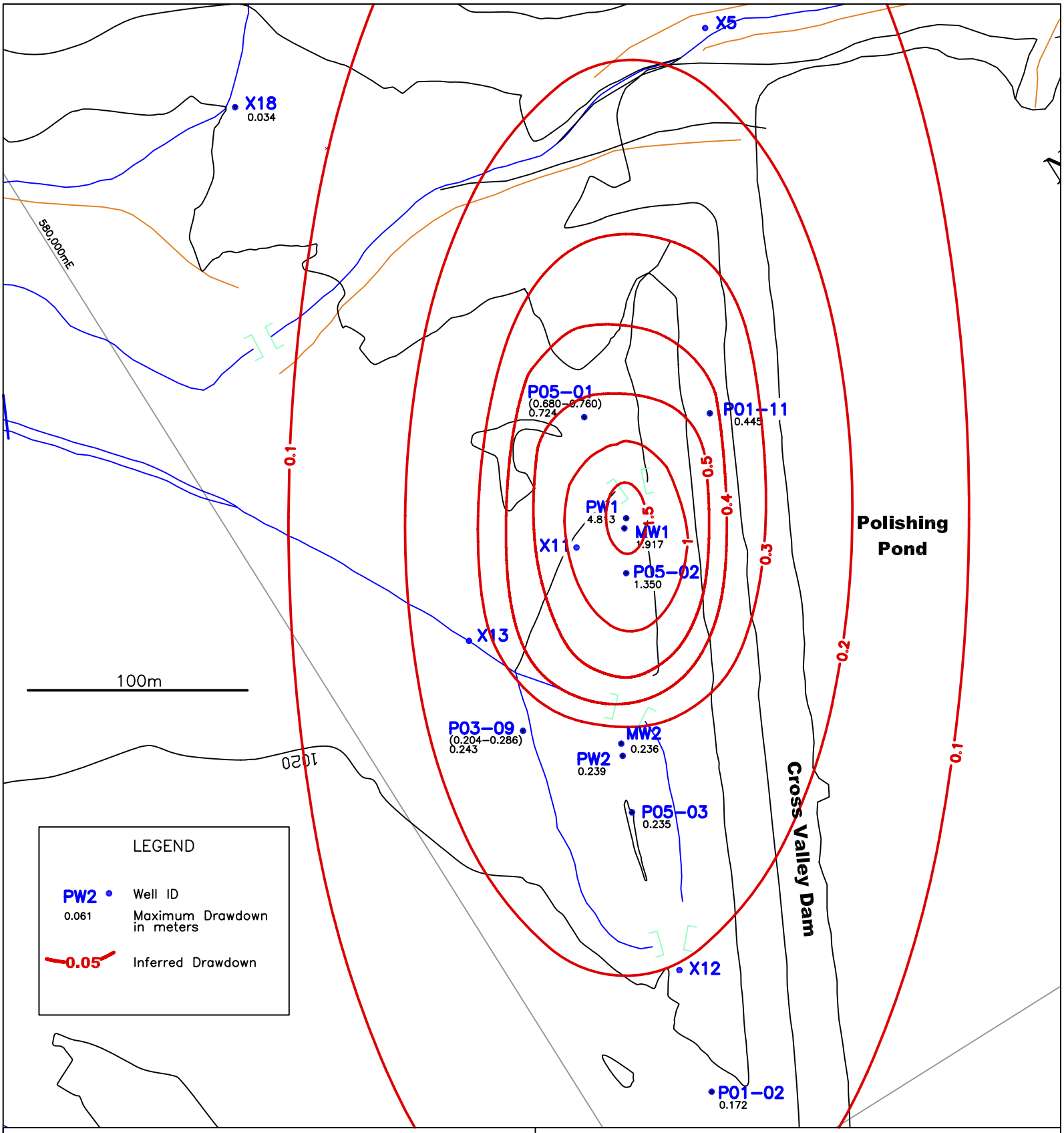


Figure 3.4. Drawdown and recovery measured at multilevel wells during 56-hr CD pump test at PW-1 (500-USGPM).



CLIENT: Deloitte & Touche Inc.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers
PROJECT: Rose Creek Groundwater Interception	Maximum Drawdown and Inferred Cone of Depression at end of 56–hr CD Pump Test at PW–1 (500 USGPM)
REPORT: 118004	
FILE: 2005 Hi Flow Pump Tests.dwg	
LOCATION: Faro Mine, Yukon Territory, Canada	

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		3.5

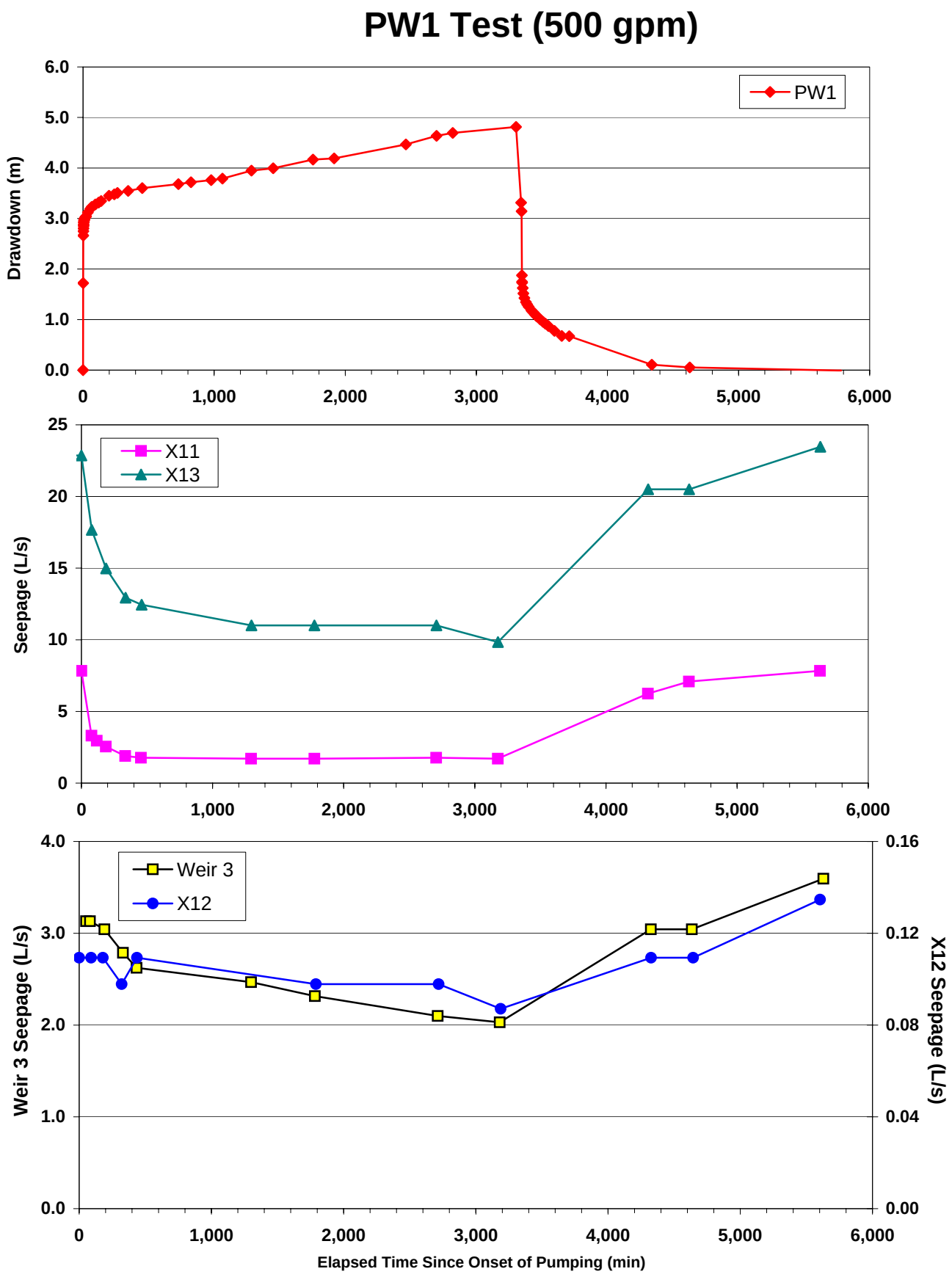


Figure 3.6. Observed seepage rates at toe of Cross Valley Dam during 56-hr CD pump test at PW-1 (500-USGPM).

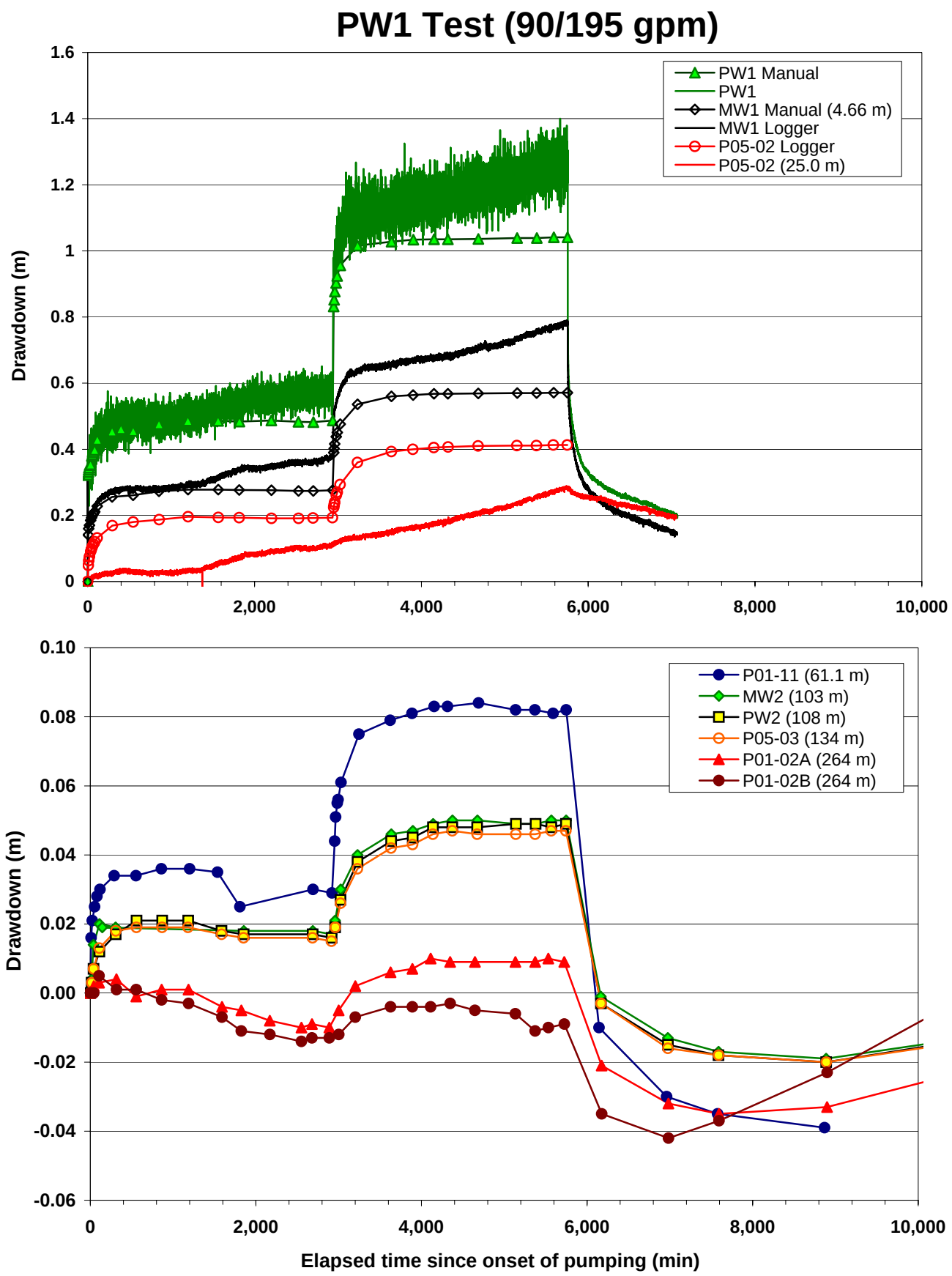


Figure 3.7. Observed drawdown and recovery at conventional wells during 4-day step test at PW-1.

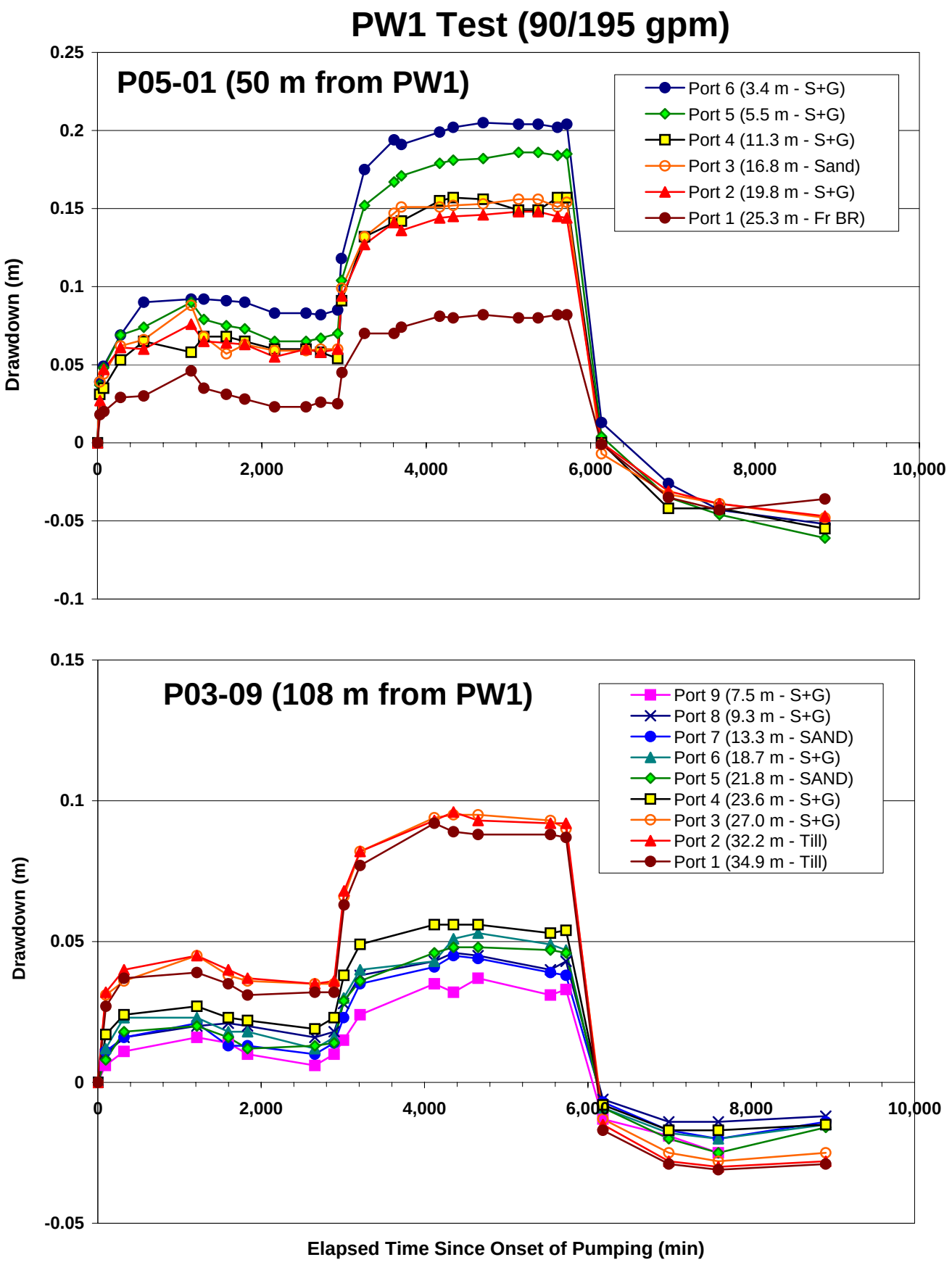
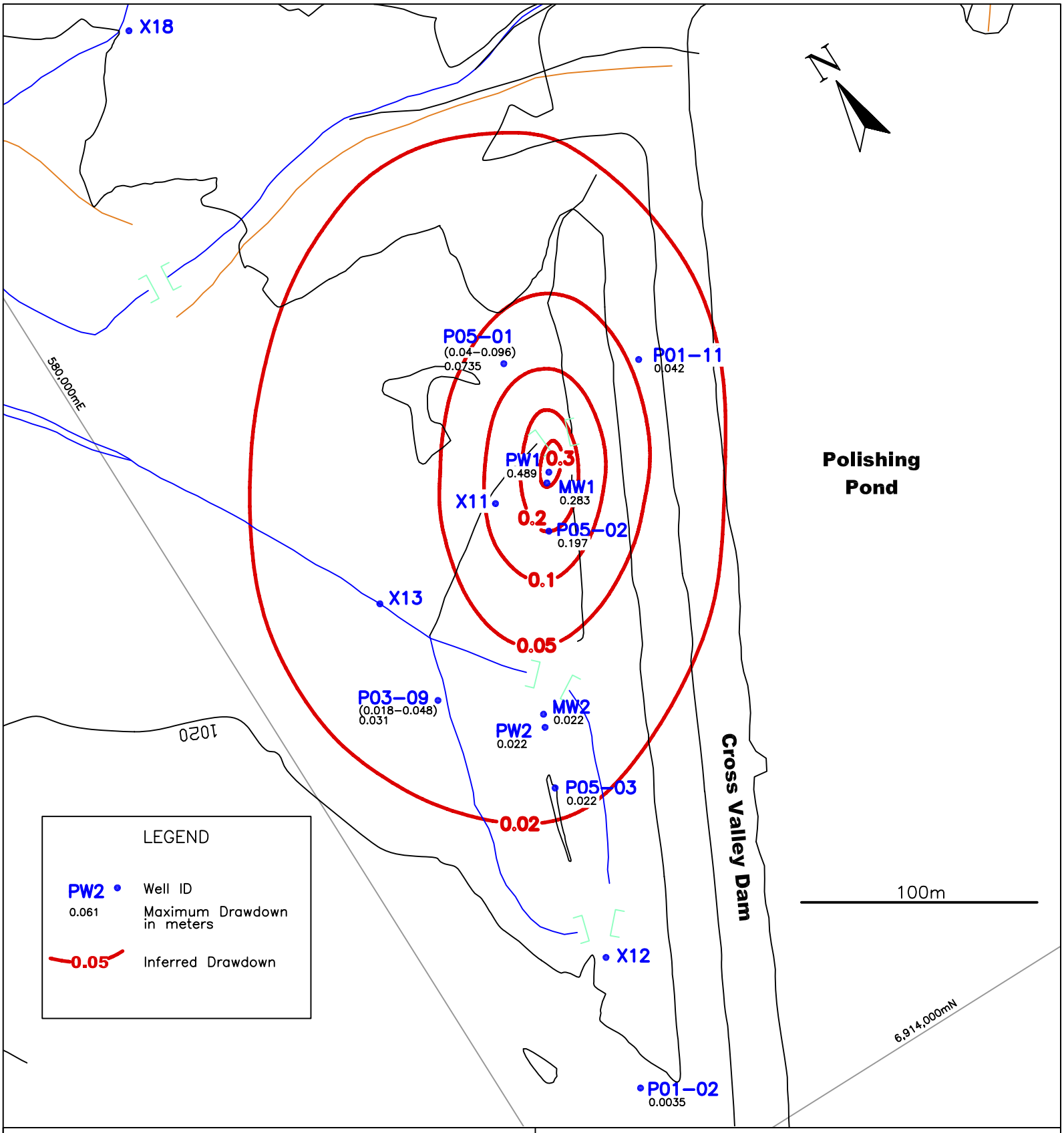
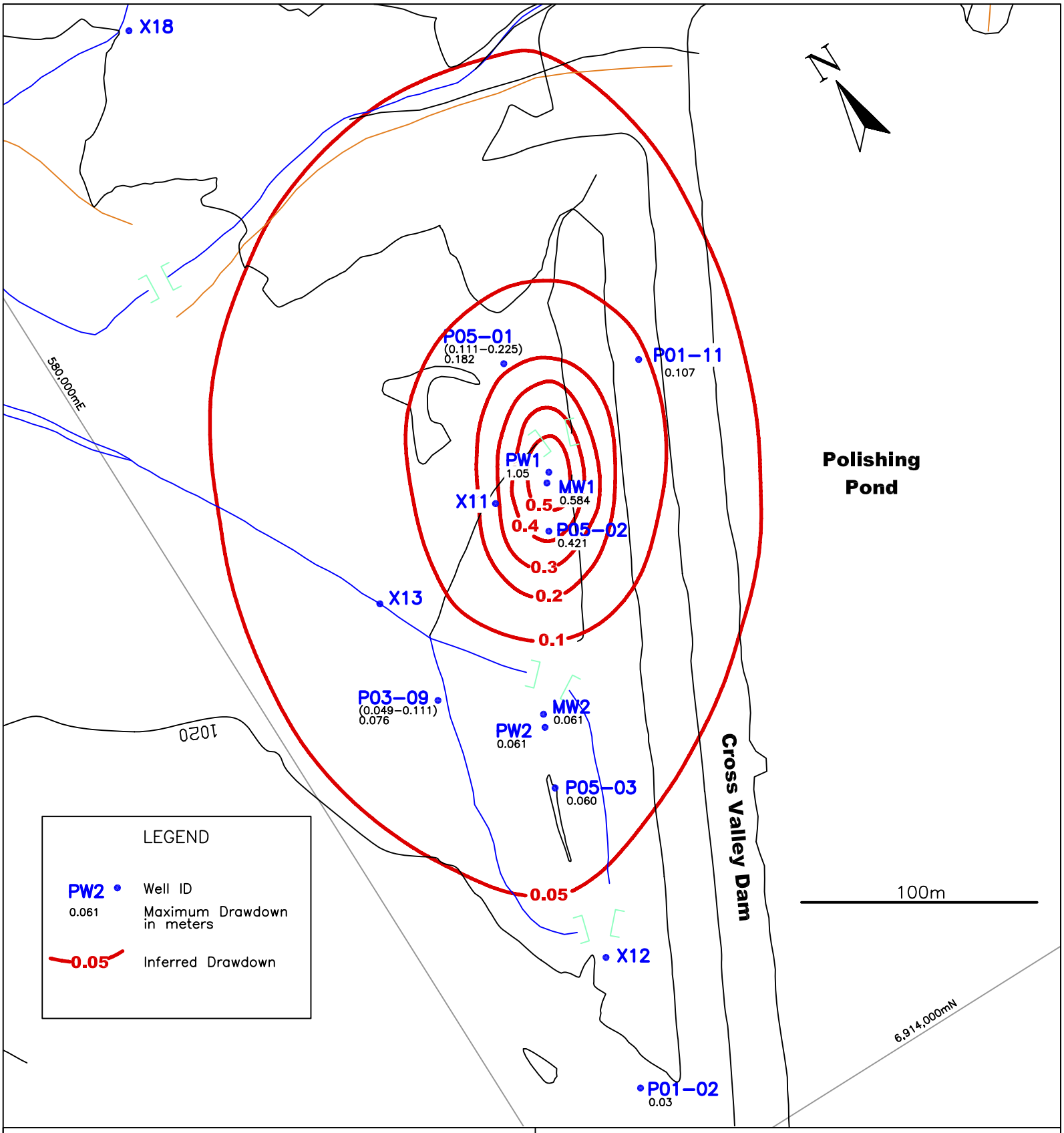


Figure 3.8. Observed drawdown and recovery at multi-level wells during 4-day step test at PW-1.



CLIENT: Deloitte & Touche Inc.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers
PROJECT: Rose Creek Groundwater Interception	Maximum Drawdown and Inferred Cone of Depression at end of 1st Step in PW–1 (48 hrs. at 90 USGPM)
REPORT: 118004	
FILE: 2005 GW contours.dwg	
LOCATION: Faro Mine, Yukon Territory, Canada	

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		3.9



CLIENT: Deloitte & Touche Inc.	Deloitte.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers	
PROJECT: Rose Creek Groundwater Interception		Maximum Drawdown and Inferred Cone of Depression at end of 2nd Step in PW–1 (48 hrs. at 195 USGPM)	
REPORT: 118004			
FILE: 2005 GW contours.dwg			
LOCATION: Faro Mine, Yukon Territory, Canada			
DATE: July, 2006	DRAWN BY: MP		FIGURE: 3.10

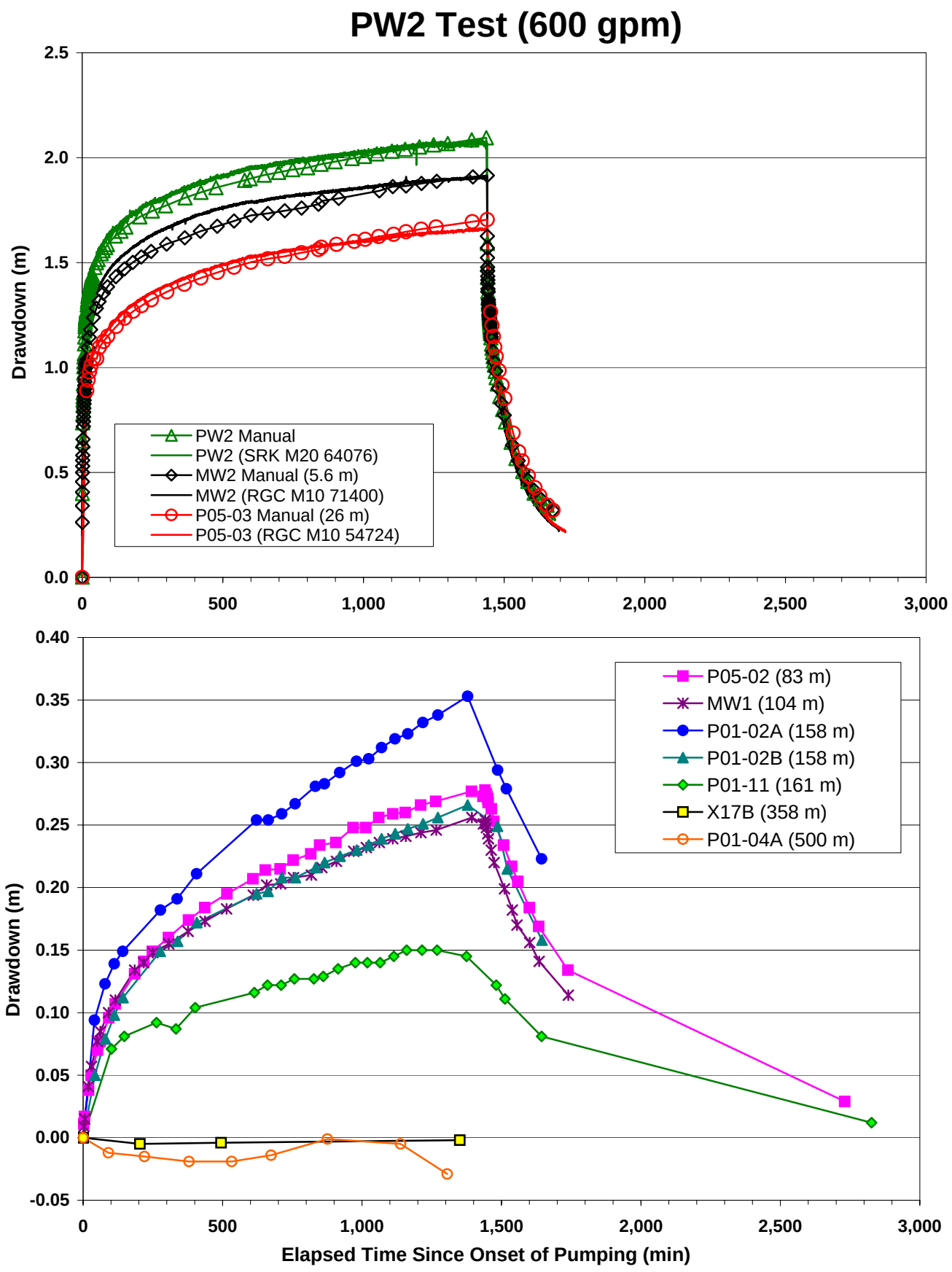


Figure 3.11. Drawdown and recovery observed at conventional wells during 24-hr CD pump test at PW-2 (600-USGPM).

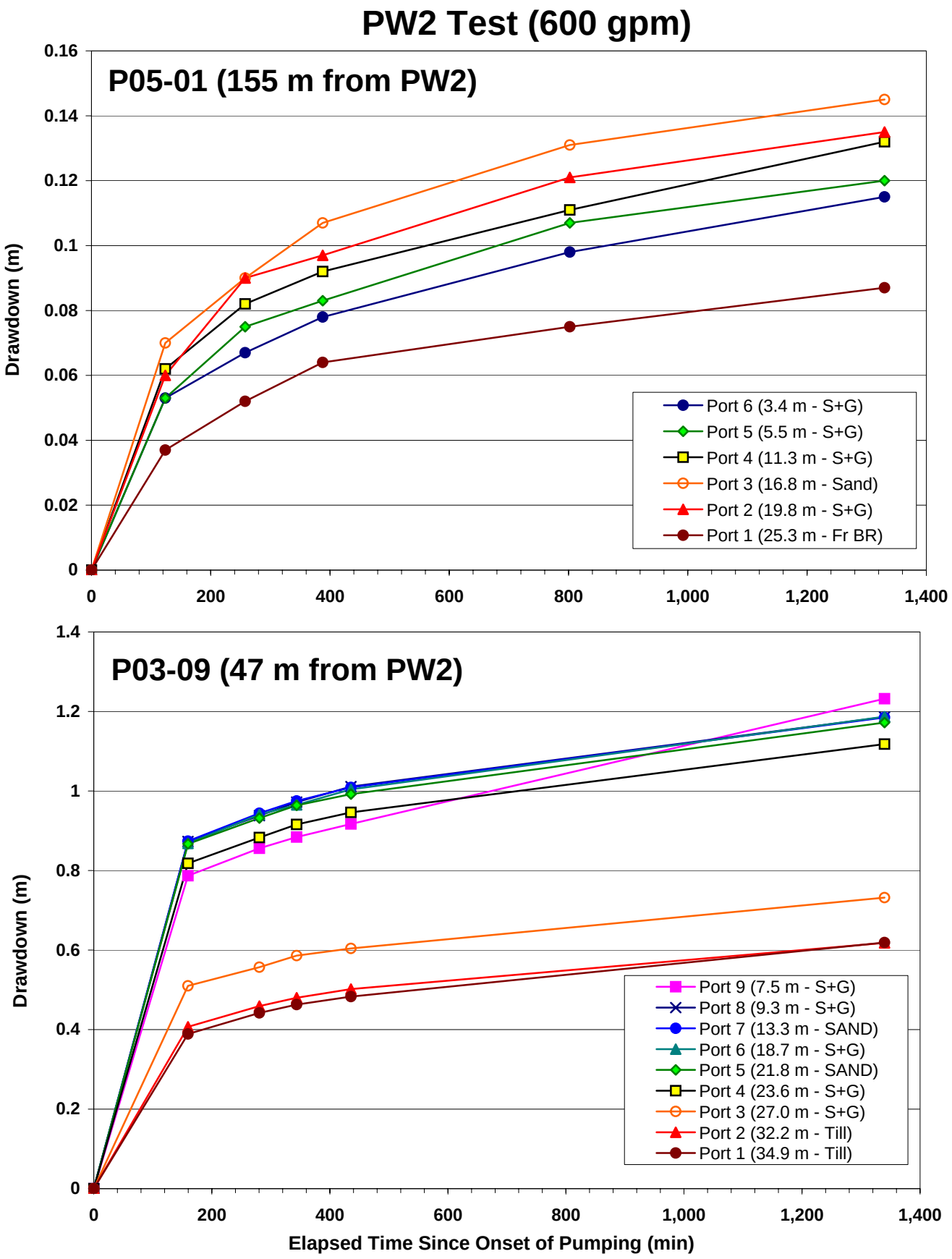
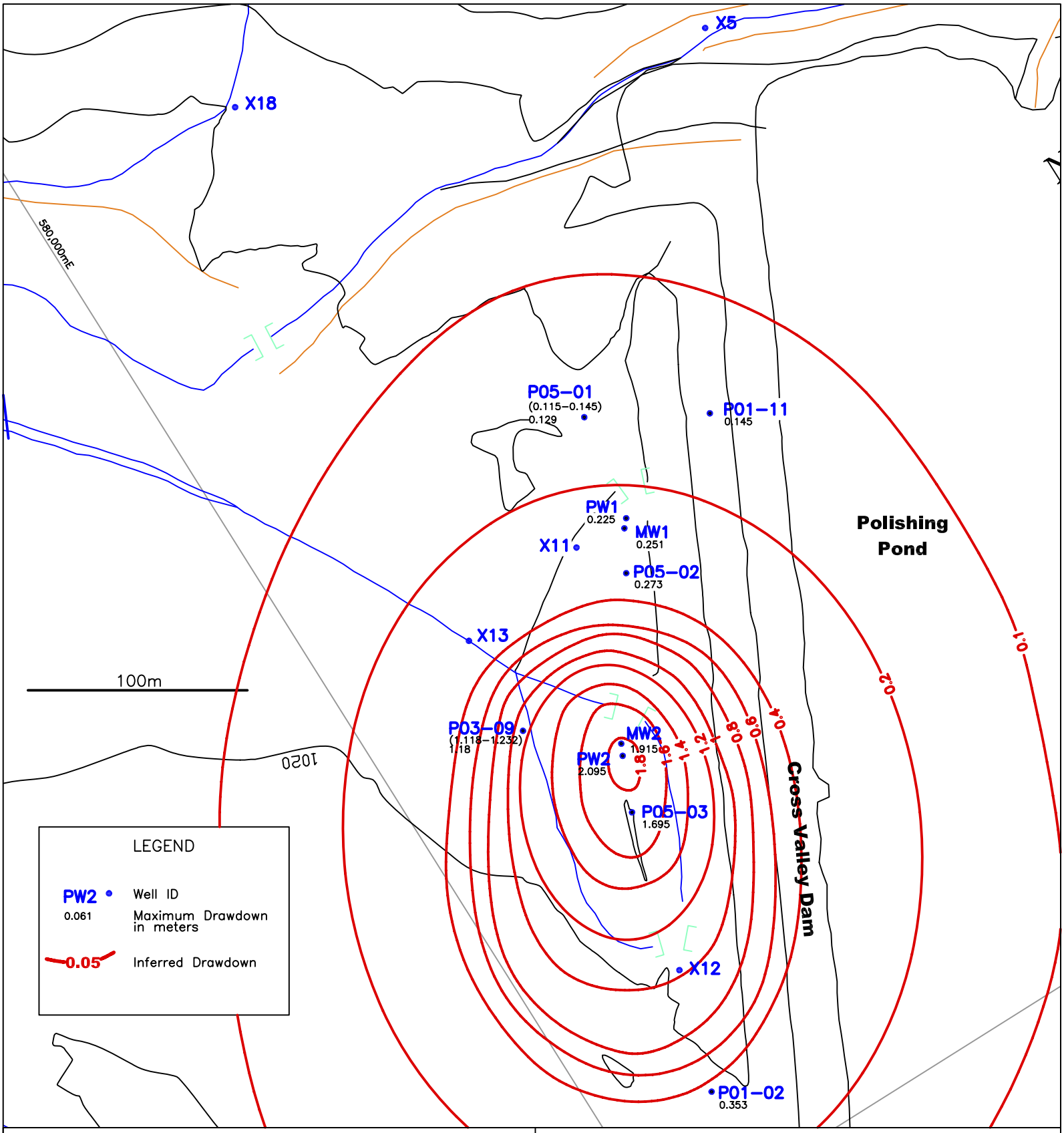


Figure 3.12. Drawdown observed at multi-level wells during during 24-hr CD pump test at PW-2 (600-USGPM).



CLIENT: Deloitte & Touche Inc.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers
PROJECT: Rose Creek Groundwater Interception	Maximum Drawdown and Inferred Cone of Depression at end of 24–hr CD Pump Test at PW–2 (600 USGPM)
REPORT: 118004	
FILE: 2005 Hi Flow Pump Tests.dwg	
LOCATION: Faro Mine, Yukon Territory, Canada	

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		3.13

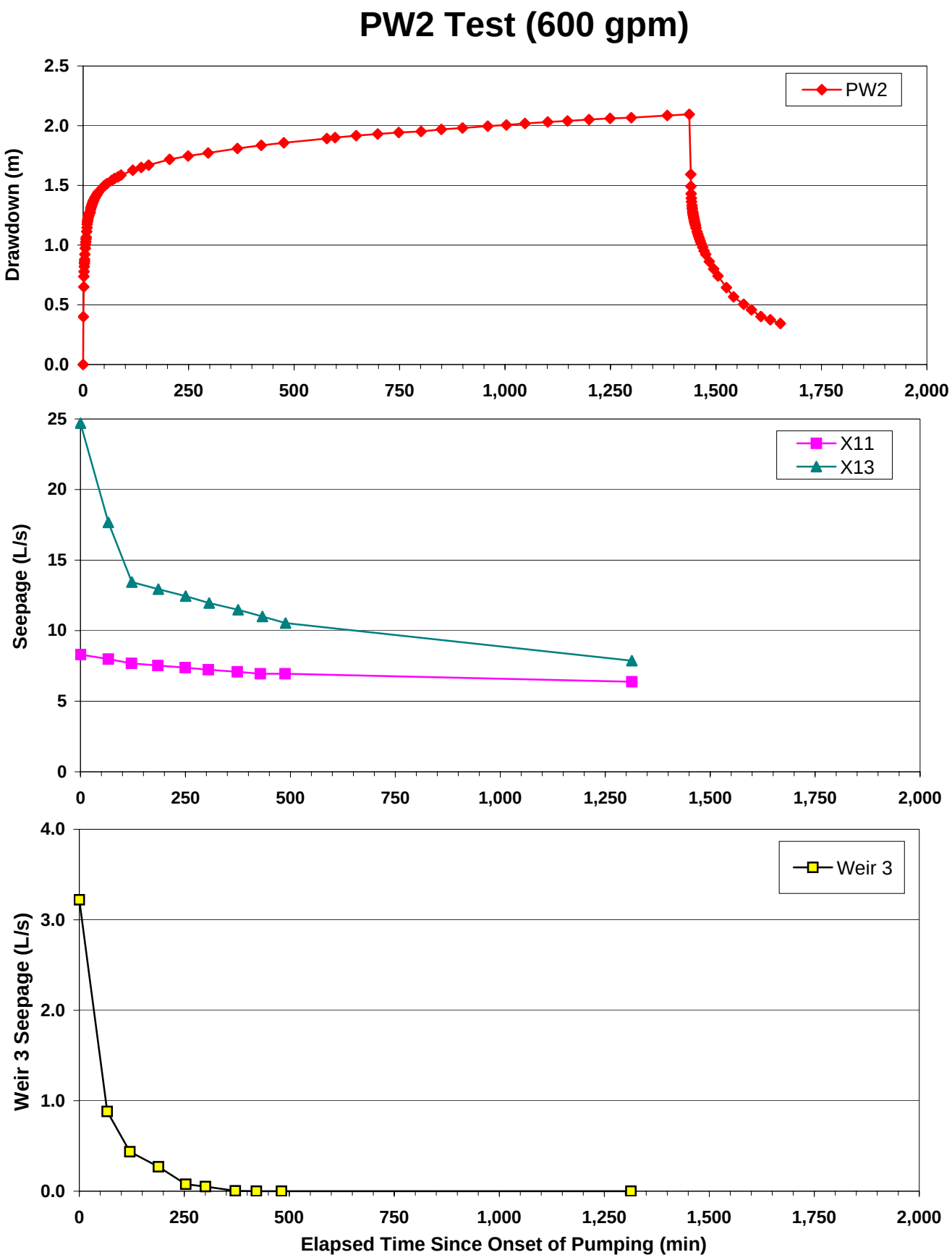


Figure 3.14. Observed seepage rates at toe of Cross Valley Dam during 24-hr CD pump test at PW-2 (600 USGPM).

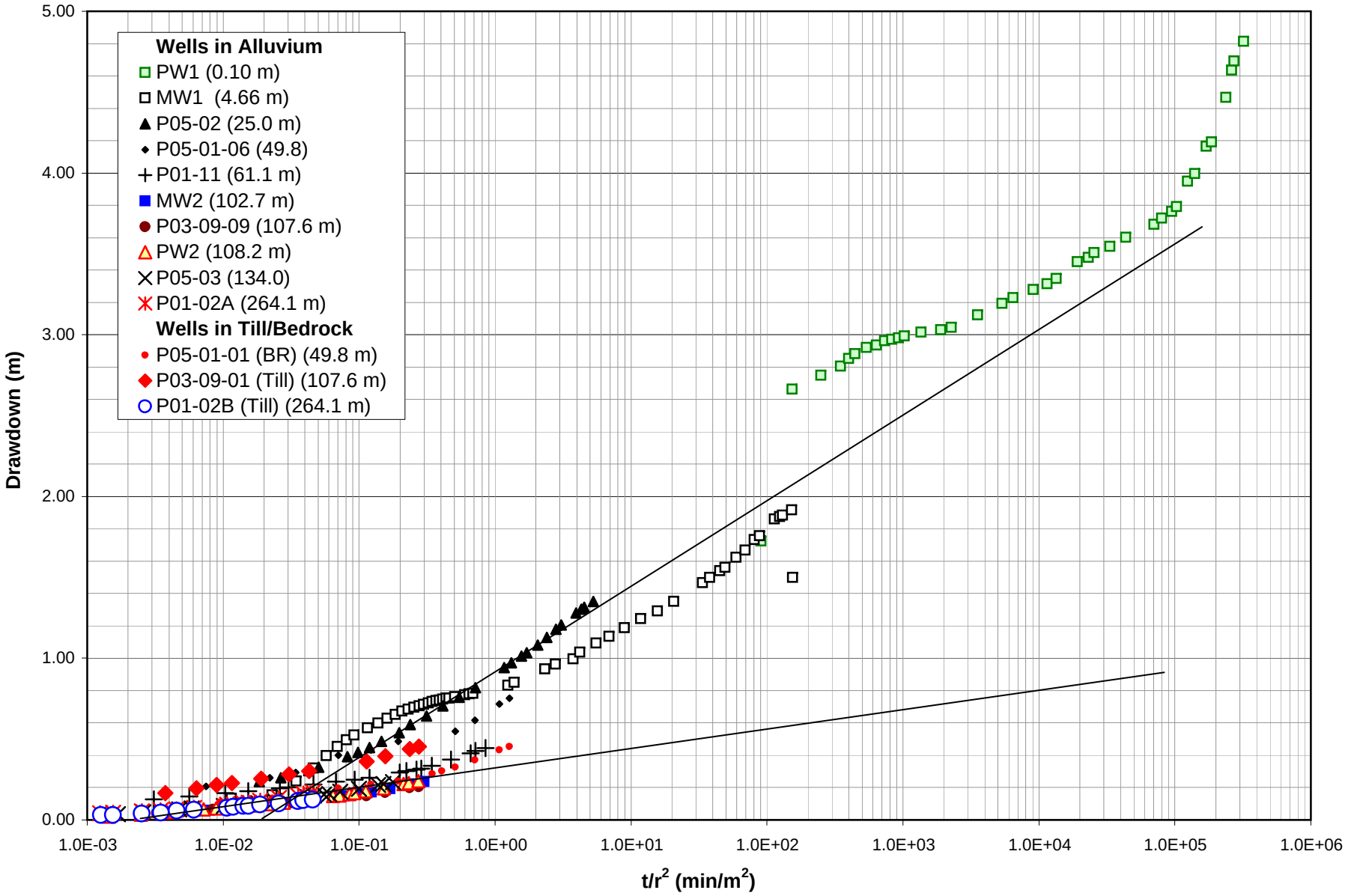


Figure 3.15. "Normalized" drawdown at the pumping well and various monitoring wells for the 56-hr CD test at PW-1 (500-USGPM).

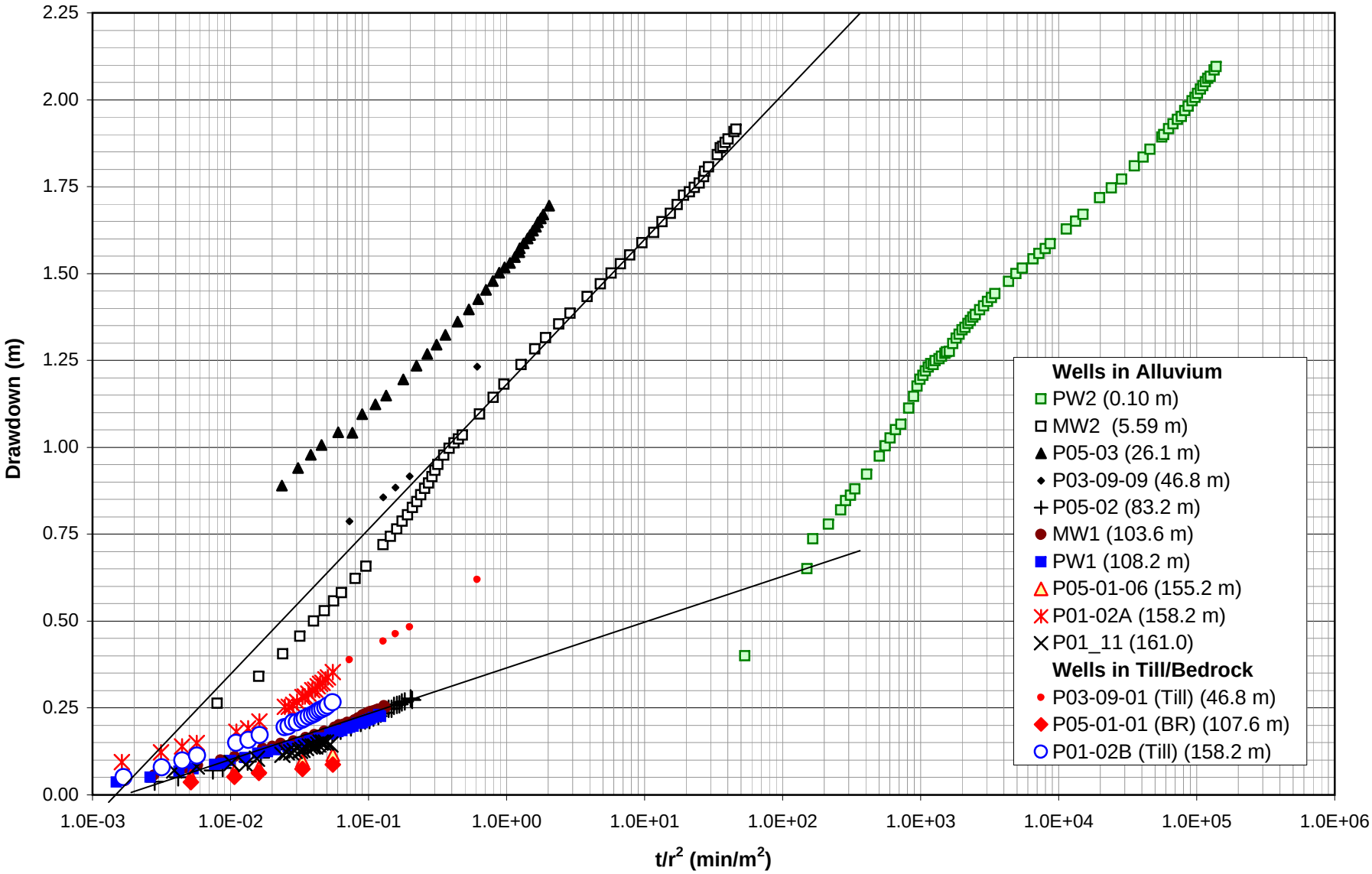


Figure 3.16. "Normalized" drawdown in pumping welll and various monitoring wells for the 24-hr CD test at PW-2 (600-USGPM).

4 CALIBRATION OF GROUNDWATER FLOW MODEL

4.1 Objectives

A three-dimensional groundwater flow model was developed and calibrated for the Rose Creek aquifer (in the vicinity of the Cross Valley Dam). The objectives of this modeling exercise were as follows:

- Determine hydraulic properties (T and S) and groundwater flow rates for the Rose Creek aquifer (near the Cross Valley Dam);
- Assess boundary conditions (valley sides; pond seepage, leakage from RCDC etc) that may influence the performance of a groundwater interception system;
- Assess the influence of aquifer heterogeneity on capture efficiency;
- Design a groundwater interception system at the pre-feasibility level including,
    - Number & location of pumping wells,
    - Required depth of pumping wells;
    - Individual and total pumping rates

This section describes the development and calibration of the groundwater flow model for the Rose Creek aquifer. The use of the calibrated groundwater flow model for the design of a groundwater interception system is described in section 5.

4.2 Conceptual Model

A generalized conceptual model of groundwater flow in the Rose Creek valley was developed based on the results of the 2005 field investigation (section 3), an earlier water and load balance study (Robertson GeoConsultants Inc., 2006) and earlier groundwater modeling studies (Robertson GeoConsultants Inc., 1996; Gartner Lee Limited, 2002, 2004). The conceptual hydrogeological model for the Rose Creek aquifer in the general vicinity of the Cross Valley Dam may be summarized as follows:

- The Rose Creek valley sediments represent a highly productive aquifer with groundwater flow generally flowing down valley, i.e. parallel to the valley axis (in a westerly direction); however, the construction of the “Down Valley Scheme” has significantly altered the local hydrogeology in the study area: seepage from the Intermediate Pond, Polishing Pond and Rose Creek Diversion Canal (RCDC) are now the primary sources of recharge to the aquifer and drainage ditches excavated downstream of the Cross Valley Dam represent the primary areas of groundwater discharge to surface;

- The local groundwater system can be subdivided into the following hydrostratigraphic units:

    - Mine waste material (Intermediate tailings; Polishing Pond sediments and earth dam materials);

    - Glacio-fluvial sediments in the valley center;

    - Colluvial sediments in selected areas (fans or terraces extending into the valley center)

- Basal till in the valley center,
- Till-like soils on the valley sides;
- Slightly weathered to fresh bedrock underlying all overburden sediments (in valley center and hill sides)

In general, the majority of groundwater flow occurs in the highly permeable glacio-fluvial sediments and colluvial sediments in the valley center. Groundwater flow in the underlying basal till and till-like soils on the valley sides and underlying bedrock is of much lower magnitude. The overlying mine waste units act primarily as conduits for seepage from the Intermediate Pond and Polishing Pond into the aquifer.

- Infiltration of snowmelt and summer/fall precipitation on the valley sides and the valley center (where not covered by mine waste) provide a seasonal source of recharge to the Rose Creek aquifer; however, this source of recharge is believed to be small compared to the seepage from the dam structures and leakage from the RCDC in the study area.
- The Rose Creek is hydraulically connected to the aquifer (in its natural streambed d/s of the Cross Valley Dam) and represents a secondary area of groundwater discharge to surface.

The numerical representation of this conceptual model is discussed in more detail below.

4.3 Numerical Methods

Groundwater flow was simulated using the finite difference groundwater flow code MODFLOW96, developed by the USGS (Harbaugh and McDonald, 1996) in conjunction with the pre-post processor GMS V3.1 (EMRL, 2000), which is commercially available and licensed to Robertson GeoConsultants Inc..

4.3.1 Model Approach

As per the scope of work for this study, the groundwater flow model was used to simulate groundwater flow conditions encountered during the pumping test period (September 2005). A simulation of high flow conditions (spring freshet) and/or transient groundwater flow (temporal trends throughout the year) was beyond the scope of this study.

The analysis of groundwater flow in the Rose Creek aquifer is based on the following set of assumptions:

- At the scale of the analysis the aquifer system can be grouped into six hydrostratigraphic units: (i) mine waste material, (ii) glacio-fluvial sediments in the valley center; (iii) colluvial sediments (in selected areas) (iv) till in the valley center (v) slope till on the valley sides; (vi) slightly weathered to fresh bedrock underlying all overburden sediments;
- The various hydrostratigraphic units can be represented by a single model layer of variable thickness (based on a geological model) with representative hydrogeologic properties (i.e. hydraulic conductivity, specific yield, storativity);
- Water movement in all hydrostratigraphic units (including bedrock units) follows Darcy's Law and can be modeled using the "equivalent porous medium" approach, i.e. using average (or

"effective") hydraulic properties; each hydrostratigraphic unit is homogeneous and isotropic (for a given sub-region);

- In the absence of pumping, groundwater flow in the Rose Creek aquifer can be approximated assuming steady-state conditions, i.e. changes in groundwater storage are negligible and groundwater flow is sustained by inflow from upstream sources and seepage/leakage from ponds and diversions; furthermore, recharge from the valley sides and the natural valley floor (uncovered) is sufficiently small that it can be ignored for the analysis of groundwater flow in September 2006
- The Intermediate Pond and Polishing Pond can be represented as a constant head boundary (equivalent to the water level of the pond), implying that the hydraulic head in the underlying tailings/sediments and the overlying water column is equal;
- The Rose Creek stream channel (and drainage ditches) can be represented as a drain arc; this assumption implies that these surface water channels represent gaining streams along their entire length in the model.

4.3.2 Model Setup

Figure 4.1 shows the model domain used for the groundwater flow model superimposed on the topographic map of the study area. The model domain covers a surface area of about 1.5 km^2 and extends from the Intermediate Pond to just upstream of the confluence of Rose Creek with Next Creek. The model domain is bounded on the north side by the Interceptor Ditch and to the south by the RCDC.

The model domain was discretized into 78 rows by 157 columns with a tighter spacing (minimum cell size of 5m x 5m) in the vicinity of the pumping wells at the toe of the Cross Valley Dam. The finite difference grid includes six model layers representing different hydrostratigraphic units.

The lateral and vertical extent of the major hydrostratigraphic units (mine waste, sediments, till and bedrock) were inferred from a 3D geological model of the Rose Creek valley. The geological model used here is based on the geological model developed by Gartner Lee Limited (GLL) for the 2001 groundwater model (GLL, 2002). For the purpose of this study, this model was extended further downstream (to include the entire model domain) and was slightly modified using the most recent drilling information in the vicinity of the Cross Valley Dam (see figure 2.2).

Model layer 1 represents anthropogenic material (i.e. tailings, PP sediments and earth dam materials). This layer is only active in those parts of the model domain where mine waste and associated earth dam materials are present. The top and bottom elevations of model layer 1 represent the current ground surface and pre-mining ground surface and were inferred from the updated geological model.

Model layers 2 to 4 represent primarily glacio-fluvial sediments but also include colluvial sediments (see below). The top and bottom elevations of this sediment package were also inferred from the updated geological model. Note that these sediments reach maximum thicknesses of up to 30m in

the center of the valley but "pinch out" along the hill sides[3] (Figure 2.2). This sediment package was subdivided into three model layers (layers 2 to 4) to better represent three-dimensional flow and to allow the simulation of partially penetrating pumping wells. The top elevations of layers 3 and 4 were assumed to be constant at 1010 m amsl and 1000 m amsl, respectively.

Model layer 5 represents till-like material and the top and bottom elevations of this unit were inferred from the updated geological model. For the purpose of this study, the till unit was further subdivided into "slope till" on the hill sides and "basal till" in the valley center (see below).

Model layer 6 represents slightly weathered/fresh bedrock which underlies the till unit across the entire model domain. The top elevation of this layer was again inferred from the updated geological model. The bottom elevation of the bedrock layer slopes from 933m AMSL at the upstream boundary to 914m AMSL at the downstream boundary. This way, the thickness of the bedrock unit was at least 50m in the valley center. This depth is considered adequate to account for deep bedrock flow in the Rose Creek Valley.

The model domain was subdivided into different model regions representing different mine waste units and/or hydrogeological areas assumed to have different hydraulic properties. For each region and model layer, an initial set of hydraulic parameters (K and S) was specified. Table 4.1 summarizes the initial estimates of hydraulic parameters for each unit which were either based on earlier work or based on the 2005 pumping test. Figures 4.2 to 4.4 show the different model regions in layers 1 (mine waste), 2-4 (sediments) and 5 (till).

Model layer 1 includes the intermediate tailings, Intermediate Dam material, Polishing Pond sediments and Cross Valley Dam material. Note that the earth dams were simulated using "effective" hydraulic parameters. A more detailed representation of the dam structure was beyond the scope of this study. The hydraulic conductivity of the tailings and PP sediments was later adjusted during model calibration. Model calibration suggested that there would be more seepage from the Polishing Pond on the northern side. An area with higher K was therefore later introduced into the model (see "window" in Figure 4.2).

Model layers 2 to 4 represent primarily glacio-fluvial sediments. Although these sediments are highly heterogeneous at the borehole scale this small-scale heterogeneity was not simulated in the model. Instead, these sediments were subdivided into sub-regions only to the extent required for model calibration (in particular the transient calibration using the pumping test data). These sub-regions are somewhat arbitrary in a sense that they are not based on a 3D block model of the sediments inferred from borehole logs. Model layers 2 to 4 also included two smaller zones referred to as "colluvium" (Figure 4.3). The southern zone represents a well-defined colluvial fan which slopes from a tributary valley towards the Rose Creek Valley (BGC, 2005). The northern area has been mapped as a glaciofluvial terrace with some morainal veneer (Klohn Leonoff, 1981). Initially, this terrace area was assumed to have the same hydraulic properties as the glacio-fluvial sediments (Table 4.1). However, model calibration suggested that this area has a lower effective hydraulic conductivity more representative of colluvium.

[3] Note that mine waste (layer 1) directly overlies till in a few peripheral areas of the model domain. In those areas, model layers 2-4 were given a nominal (very small) thickness and given the hydraulic properties of the underlying till material (layer 5) to provide continuity

Table 4.1

Initial Estimates of Hydraulic Parameters

Hydrostratigraphic Unit	Model Layer	Model Sub-region	Initial Model Input Parameters		Source
			K (m/s)	S (m/m)	
Intermediate Tailings	1	IT	2.0E-08	-	GLL, 2002
Intermediate Dam	1	ID	3.4E-07	-	GLL, 2002
Polishing Pond Sediments	1	PP	3.0E-07	-	GLL, 2002
Cross Valley Dam	1	CVD	1.0E-07	-	GLL, 2002
Glacio-fluvial Sediments	2 - 4	GFS-N	4.9E-04	0.0326	Pumping test at PW-1
	2 - 4	GFS-S	7.0E-04	0.0009	Pumping test at PW-2
	2 - 4	GFS regional-E	1.5E-04	-	GLL, 2002
	2 - 4	GFS regional-W	5.0E-04	-	GLL, 2002
Colluvium	2 - 4	COL-N	3.0E-07	-	GLL, 2002
	2 - 4	COL-S	6.0E-05	-	slug test at DH81-K5 (KL, 1981)
Basal Till	5	TB	3.0E-07	-	GLL, 2002
Slope Till	5	TS	1.0E-05	-	GLL, 2002
Bedrock	6	BR	2.0E-07	-	GLL, 2002

Model layer 5 represents all till-like material. As outlined above, the till unit was further subdivided into “slope till” on the hill sides and “basal till” in the valley center (Figure 4.4). Conceptually, the basal till is highly consolidated and therefore less permeable material, whereas the slope till represents less consolidated and therefore more permeable material (Table 4.1). The major break in topography from the valley floor to the hill side was used to determine the transition from basal till to slope till.

Figures 4.5 to 4.7 show the boundary conditions for model layers 1 to 3. Model layer 1 is only active within the foot print area of the major mine waste units. Most mine waste units are bounded on all sides by no-flow boundaries. The Intermediate Pond (IP) and Polishing Pond (PP) are represented by constant head nodes. In other words, the pond may either receive inflow from the aquifer (e.g. on the upstream side of the PP) or provide seepage to the aquifer (e.g. on the downstream side of the PP).

Seepage from layer 1 infiltrates into model layer 2 (below natural ground). Initial simulations showed that seepage faces did not develop at the toe of the dams, hence no drain nodes were required in layer 1.

The external boundaries of model layers 2 to 6 are essentially the same with constant head boundaries on the upgradient and downgradient sides and no-flow boundaries on the valley sides. The constant head boundaries simulate the underflow entering the model domain on the upgradient side and underflow leaving the model domain on the downgradient side. The no-flow boundaries on the valley sides imply that groundwater recharge from the valley sides is negligible. Initial sensitivity runs using constant head boundaries (to simulate recharge from the valley sides) supported the use of this simplifying assumption.

The internal sources/sinks to the model included:

- Rose Creek Diversion Canal

- Rose Creek (natural streambed); and
- Drainage ditches along the toe of the Cross Valley Dam (to confluence with Rose Creek);

The Rose Creek Diversion Canal was represented using constant head and river nodes in model layers 2-6. Downstream of the Cross Valley Dam, where the RCDC diverges from the valley side, the elevation of the RCDC was significantly higher than groundwater level in the valley suggesting potential leakage from the RCDC. River nodes were assigned to layer 2 in this reach where there was reason to believe that the RCDC might be perched. In this area the rate of leakage is a function of the river conductance. The magnitude of the river conductance was initially set equal to the hydraulic conductivity of the geologic unit, but was subsequently modified (reduced) during model calibration. Where the RCDC flows along the side of the valley (southeast margin of the model), it was simulated as river nodes and constant heads in the till and bedrock (layers 5 & 6), implying that the head in the channel has equilibrated across the hydrostratigraphic units. The constant heads were introduced in layer 6 to improve calibration of head measurements near the valley sides (P01-02).

The Rose Creek itself was simulated using drain nodes in layer 2 with the drain elevation set equal to the elevation of the creek bed. This way, groundwater discharges to the creek (drain) only if the hydraulic head is greater than the creek bed and ceases when the water table falls below the creek bed. This approach is consistent with previous streamflow measurements which had suggested that Rose Creek is a net gaining stream along the reach downstream of the Cross Valley Dam (Robertson GeoConsultants Inc., 2006). The drain bed conductance was set sufficiently high so that the hydraulic conductivity of the aquifer controlled discharge into the stream rather than the streambed conductance.

The drainage ditches located downstream of the Cross Valley Dam were also simulated using drain nodes in layer 2. Again, this approach is consistent with previous streamflow measurements which had suggested that these ditches receive significant groundwater discharge (Robertson GeoConsultants Inc., 2006). Note that no detailed spot elevations were available for the drains, hence the drain elevations had to be estimated based on available (digital) surface topography maps. For the purpose of this study, the drain conductance was again set equivalent to the hydraulic conductivity of the underlying aquifer material.

4.3.3 Model Calibration

The groundwater flow model was first calibrated assuming steady-state conditions. This initial calibration was then followed with a transient calibration using the results of the two pumping tests. In general, the steady-state calibration focused on the entire model domain whereas the transient calibration focused on the immediate vicinity of the CVD. As might be expected, several iterations between steady-state and transient calibration were required before a satisfactory match with all data was achieved.

The steady-state model was calibrated using the detailed groundwater level survey carried out on September 6, 2005 in the reach of interest. The monitoring wells and observed groundwater elevations used for steady-state calibration are listed in Table 4.2. The inferred groundwater flow field is illustrated in Figure 2.3. In addition, the steady-state model was calibrated against various seepage estimates. Unfortunately, seepage estimates for early September 2005 were only available for

selected stations at the toe of the CVD (X11, X12, and X13). These seepage estimates were on

Table 4.2

Observed groundwater levels (September 6, 2005) used for calibration of steady-state model.

Well ID	DTW (m bTOC)	Water Level (m bgs)	Water Level (m amsl)
Intermediate Dam			
X24A	5.35	4.54	1027.76
X24C	5.23	4.53	1027.77
X24D	5.13	4.43	1027.87
X25A	4.62	3.94	1027.46
X25B	4.42	3.79	1027.61
P01-03	4.20	3.64	1028.02
P01-04A	2.91	2.40	1028.99
P01-04B	3.43	2.93	1028.46
Cross Valley Dam			
P01-11	1.01	0.46	1016.82
P05-01-01	1.67	1.07	1016.25
P05-01-02	1.82	1.22	1016.10
P05-01-03	1.83	1.23	1016.09
P05-01-04	1.90	1.29	1016.02
P05-01-05	2.23	1.62	1015.69
P05-01-06	2.35	1.75	1015.57
PW1	2.32	1.41	1014.98
MW1	1.92	1.26	1015.05
P05-02	1.87	0.94	1014.79
MW2	3.13	2.36	1015.09
PW2	3.47	2.70	1015.17
P05-03	4.60	3.75	1015.20
P01-02A	2.11	1.41	1017.61
P01-02B	0.57	-1.01	1020.04
P03-09-01	2.78	2.09	1015.74
P03-09-02	3.01	2.32	1015.51
P03-09-03	3.12	2.43	1015.39
P03-09-04	3.38	2.69	1015.14
P03-09-05	3.41	2.72	1015.10
P03-09-06	3.43	2.74	1015.08
P03-09-07	3.57	2.88	1014.95
P03-09-08	3.61	2.92	1014.91
P03-09-09	3.77	3.08	1014.74
Regional Bores d/s of Cross Valley Dam			
X16A	3.63	2.81	1013.10
X16B	3.96	2.74	1013.17
X17A	2.17	1.53	1012.77
X17B	2.72	1.44	1012.86
X18A	4.40	3.78	1015.19
X18B	4.11	3.43	1015.54
P01-01A	4.02	3.26	1012.12
P01-01B	3.82	3.34	1012.04

Table 4.3

Observed seepage rates used for steady-state calibration.

Station ID	Description	Calibration Target[1,2] (L/s)
X11	North side CVD toe seepage	8.3 – 10.2
X12	South side CVD toe seepage	0.1 – 0.3
X13	Combined CVD toe seepage	25 – 31
CVS1	Incremental CVD seepage (between X13 and just u/s of confluence with Rose Creek)	41 – 51
RCDC	Incremental seepage loss from RCDC between Intermediate Dam and X10	40 - 200[3]
Rose Creek	Streamflow gain in Rose Creek from X10 to downgradient model domain	43 -175[3]

Notes:
1. Positive values (black font) indicate a net gain in streamflow (groundwater discharge)
2. Negative values (red font) indicate a net loss of streamflow (recharge to groundwater)
3. lower limit based on April 2005 measurement

average 20% lower than those observed during a detailed streamflow survey carried out on September 19/20, 2004. For model calibration, the observed ranges of seepage rates for those two dates were used as calibration targets (see Table 4.3). For station CVS1 (not surveyed in September 2005), the lower target rate was pro-rated assuming a 20% lower flow rate than observed in September 2004.

No streamflow measurements were taken along the RCDC and Rose Creek in September 2005. For those reaches, very different seepage losses (RCDC) and gains (Rose Creek) were observed in two different surveys carried out as part of the Rose Creek Water and Load Balance Study (Robertson GeoConsultants Inc., 2006). It is unclear whether these differences are due to different flow conditions (the streamflow in Rose Creek in April 2005 was only about 50% of the stream flow in September 2004) or whether they are due to measurement errors. Because of the uncertainty in these measurements, the (significant) range of measurements was used as calibration targets.

It should be emphasized that different methods were used to arrive at these seepage estimates. The flow rates for stations equipped with weirs are considered more accurate and are shaded in yellow. These estimates are considered much more reliable and every attempt was made during steady-state model calibration to meet these targets. The other seepage targets are more uncertain and they were only used as a general guide during model calibration.

For transient calibration, the results of both the 56-hour pumping test in PW-1 (north side) and the 24-hr pumping test in PW-2 (south side) were utilized. For each transient calibration, the simulated drawdown and recovery in the pumping well and eight selected monitoring wells were compared to the drawdown/recovery curves observed in the respective wells during the pumping test. In addition, the simulated change in the seepage rates at the toe of the Cross Valley Dam (primarily at X11 and X13) were compared to the observed changes in seepage rates to further constrain the model.

In general, all model calibration was carried out by trial-and-error, i.e. model parameters were varied within reasonable constraints (based on available field data and experience of the author) and the resulting flow solution was compared to the calibration targets. During steady-state calibration the goodness-of-fit of the simulated versus observed groundwater levels was evaluated by means of linear regression analysis. The primary objective of model calibration was to minimize the scatter (increase r^2 and reduce RMS) and to obtain an unbiased match of the simulated and observed water levels (i.e. a linear regression with zero intercept and slope=1.0). For the purpose of this study, a steady-state model calibration was deemed acceptable if the RMS was less than 1m.

No rigorous statistical analysis was used for assessing the goodness-of-fit with the seepage calibration targets (Table 4.3). Instead, the goodness-of-fit was judged on a case-by-case basis considering the uncertainty in the individual seepage estimates. In general, calibration focused on those seepage estimates for the CVD toe seepage (X11 and X13), which are considered reliable. For those calibration targets, model calibration was deemed acceptable if the simulated seepage rate fell within the observed target range. The other seepage rates only provided a general guideline and no specific calibration criteria were set.

For the transient calibration a statistical analysis of the goodness-of-fit of simulated versus observed drawdown/recovery was beyond the scope of this study. Instead, the simulated and observed drawdown/recovery curves were plotted on the same graph for visual comparison. While the primary emphasis was on matching the maximum drawdown, the shape of the drawdown and recovery curves was also used for assessing the “goodness-of-fit”.

A deliberate attempt was made to keep the complexity of the model to a minimum in order to avoid over-parameterization and to develop a transparent, robust model. In other words, our initial parameter estimates of K and S were only modified where it resulted in significant improvement of model calibration. Similarly, specific sub-zones (such as sub-regions in the glacio-fluvial sediments) were only introduced if their use resulted in a notable improvement in model calibration (with respect to water levels and/or seepage).

Note that a simulation of vertical heterogeneity within a given hydrostratigraphic unit (e.g. glacio-fluvial sediments) was beyond the scope of this study. In other words, each hydrostratigraphic unit was assumed to be isotropic.

4.4 Results

4.4.1 Steady-State Calibration

Table 4.4 lists the simulated groundwater levels for the 31 observation points and the computed residuals (deviation from observed value) for the calibrated model. Figure 4.8 shows a scatter plot of simulated versus observed groundwater levels and summarizes the statistical analysis of the goodness-of-fit. The data indicate a very good overall fit with observed groundwater levels (r^2=0.98).

Table 4.4

Simulated and observed water levels for the calibrated model.

Well ID	Model Layer	Observed SWL (masl)	Simulated SWL (masl)	Sim-Obs (m)
Intermediate Dam				
X24A/C	2	1027.77	1028.70	0.94
X24D	5	1027.87	1028.69	0.82
X25A/B	2	1027.54	1027.56	0.02
P01-03	2	1028.02	1028.89	0.88
P01-04A	4	1028.99	1027.64	-1.35
P01-04B	5	1028.46	1027.71	-0.75
Cross Valley Dam				
P01-11	3	1016.82	1016.82	0.00
P05-01-01	6	1016.25	1015.81	-0.44
P05-01-02	4	1016.10	1015.54	-0.56
P05-01-03-04	3	1016.06	1015.55	-0.51
P05-01-05-06	2	1015.63	1015.53	-0.10
PW1[1]	2,4	1014.98	1015.54	0.56
MW1[1]	2,4	1015.05	1015.51	0.47
P05-02	2	1014.79	1015.49	0.69
MW2[1]	2,3	1015.09	1015.63	0.54
PW2[1]	2,3	1015.17	1015.66	0.49
P05-03	2	1015.20	1015.81	0.62
P01-02A	2	1017.61	1020.00	2.39
P01-02B	5	1020.04	1020.63	0.59
P03-09-01-02	5	1015.62	1014.83	-0.79
P03-09-03-04	4	1015.26	1014.78	-0.48
P03-09-05-06	3	1015.09	1014.77	-0.32
P03-09-07-09	2	1014.87	1014.76	-0.11
Regional Bores d/s of Cross Valley Dam				
X16A	2	1013.10	1012.45	-0.65
X16B	4	1013.17	1012.44	-0.73
X17A	2	1012.77	1012.85	0.08
X17B	4	1012.86	1012.85	-0.01
X18A	3	1015.19	1013.00	-2.19
X18B	6	1015.54	1013.83	-1.71
P01-01A	3	1012.12	1012.56	0.45
P01-01B	6	1012.04	1012.59	0.56
			RMS (m)	**0.89**
			Mean Error (m)	**0.02**

1. Average simulated heads provided for wells screened over multiple layers

Note that the remaining scatter is random as indicated by a slope of 1.0 for the regression line. The majority of groundwater levels were predicted within the “acceptable” target range of 1m. The poorest match was obtained for P01-02A (in layer 2) which was overpredicted by about 2.4m. Note also that

the mean error was very close to zero (0.02m), indicating that there is no systematic bias in the simulated heads (Table 4.4).

Table 4.5 compares the simulated flow rates with the calibration targets for selected seepage ditches and stream reaches. The simulated seepage rates along the toe of the Cross Valley Dam are in good agreement with the seepage rates observed on September 7, 2005. Note that the model reproduces not only the total toe seepage (at X13) but also the large spatial differences in seepage between the north side (X11) and the south side (X11).

Table 4.5

Target and simulated fluxes for the calibrated model.

Station ID	Calibration Target[1,2] (L/s)	Simulated Fluxes (L/s)
X11	8.3 – 10.2	8.2
X12	0.1 – 0.3	0.3
X13	25 – 31	31
CVS1	41 – 51	38
RCDC	40 - 200[3]	38
Rose Creek	43 -175[3]	12

Notes:
1. Positive values (black font) indicate a net gain in streamflow (groundwater discharge)
2. Negative values (red font) indicate a net loss of streamflow (recharge to groundwater)
3. lower limit based on April 2005 measurement

Figure 4.9 shows the simulated groundwater levels (piezometric head) in model layer 2. Very similar groundwater levels were simulated for the deeper layers in the Rose Creek aquifer (see Appendix G for detailed output of the calibrated model). Table 4.6 summarizes the calibrated hydraulic conductivity (K) values for the different sub-regions and model layers. The results of the steady-state calibration can be summarized as follows:

- The calibrated hydraulic conductivity of the glacio-fluvial sediments ranges from 1.5 $x10^{-4}$ to 5.0$x10^{-4}$ m/s. This range is in good agreement with earlier modeling results (GLL, 2002) and our initial K estimates for the aquifer (2$x10^{-4}$ m/s). The higher K estimate in the sub-region along the toe of the Cross Valley Dam was primarily required to explain the pumping test results (see section 4.4.2 below) but also improved the steady-state head solution and calibration of the toe seepage;

- The seepage rates along the toe of the Cross Valley Dam are sensitive to both the hydraulic conductivity in the valley sediments and the seepage rate from the Polishing Pond. Uniform properties in the PP sediments and the underlying aquifer could not reproduce the significant seepage observed at X11 (on the northern side of the valley). The hydraulic conductivity in the northern "colluvium" (also referred to as terrace material) had to be reduced and a "seepage window" had to be introduced along the northern side of the PP to reproduce the observed seepage at X11. The lower K estimate for the northern "colluvium" ($1x10^{-5}$ m/s) is conceptually consistent with the 2005 drilling results which suggested a lower proportion of well-washed sediments at P05-01 compared to at MW-1 and MW2 (see section 2.2.1);

- The presence of the southern colluvial fan strongly influences the flow field and the seepage rates in the reach downstream of the toe of the Cross Valley Dam. The calibrated model suggests that this unit is less permeable than the glaciofluvial sediments and therefore restricts groundwater flow down-valley in this area. The resulting flow field suggests that leakage from the Rose Creek Diversion is the primary source of groundwater to this area, which is consistent with the very low sulphate concentrations observed at X17 and X16 (see section 2.3.1). The calibrated hydraulic conductivity for this unit ($K=1x10^{-5}$ m/s) is slightly lower than the only hydraulic testing data available for this area ($6x10^{-5}$ m/s at DH81-K5, Klohn Leonoff, 1981) but is well within the range of K values that can be expected for a colluvial/fluvial fan;

- The simulated streamflow losses from the RCDC and streamflow gains in Rose Creek were much closer to those observed in April 2005 than those observed in September 2005. Sensitivity runs indicated that higher seepage losses from the RCDC would greatly increase the seepage rates at CVS1 (not observed). The calibrated model therefore suggests that the April 2005 survey results are more representative of actual seepage losses to groundwater and gains from groundwater discharge than the September 2004 survey results.

- The simulated discharge of groundwater to Rose Creek (10.5 L/s) was significantly lower than the range of streamflow gains (43-175 L/s) measured using stream surveys. However, it should be noted that the model is poorly constrained in this reach (i.e. no drilling information is available for this reach of the aquifer). Possible reasons for this discrepancy include (i) short-circuiting of RCDC leakage to Rose Creek (e.g. along a historic stream channel), (ii) convergence of groundwater flow due to a restriction in the valley cross-section, (iii) significant recharge from the valley sides and/or (iv) higher groundwater flow in the Rose creek aquifer (than simulated). Additional groundwater investigations (including drilling) would be required to improve the model calibration in this reach of the model domain;

- The model simulated upward groundwater flow (from the bedrock/till into the overlying glacio-fluvial sediments) downstream of the Cross Valley Dam (e.g. at P03-09 and P05-01). However, the simulated vertical hydraulic gradients are significantly lower than observed in the field (Table 4.4). Preliminary sensitivity runs (not shown here) indicated that the observed heads in the till and bedrock (and hence vertical upward gradients) could be explained by very small rates of groundwater flow in bedrock/till from the valley sides. This groundwater flow represented only a small component of total groundwater flow in the Rose Creek aquifer (~1 %) and this added complexity was therefore not included in the current version of the model.

Table 4.6

Calibrated hydraulic parameters.

Hydrostratigraphic Unit	Model layer	Model Sub-region	Calibrated Model Parameters		
			K (m/s)	Ss	Sy
Intermediate Tailings	1	IT	2.0E-08	0.0001	-
Intermediate Dam	1	ID	3.0E-07	-	0.1
Polishing Pond Sediments	1	PP	9.0E-10	0.0001	-
Polishing Pond Window	1	PP	3.0E-07	0.0001	-
Cross Valley Dam	1	CVD	1.0E-07	-	0.1
Glacio-fluvial Sediments	2 - 4	GFS-N	5.0E-04	0.001	0.1
	2 - 4	GFS-S	5.0E-04	0.0001	-
	2 - 4	GFS regional-E	1.5E-04	0.001	-
	2 - 4	GFS regional-W	3.0E-04	0.001	-
Colluvium	2 - 4	COL-N	1.0E-05	0.001	-
	2 - 4	COL-S	1.0E-05	0.0001	-
Basal Till	5	TB	3.0E-07	0.000001	0.01
Slope Till	5	TS	3.0E-06	0.000001	0.05
Bedrock	6	BR	2.0E-07	0.000001	0.001

4.4.2 Transient Calibration

Table 4.6 summarizes the calibrated storage parameters for the different sub-regions and model layers. Figures 4.10 and 4.11 show the simulated drawdown and recovery in the pumping well and selected monitoring wells for the 52-hr pumping test in PW-1 and the 24-hr test in PW-2, respectively. The observed time trends of drawdown are shown for comparison. Contour plots of maximum drawdown in the various model layers for the two tests are provided in Appendix G.

The results of the transient calibration can be summarized as follows:

- The shape of the drawdown and recovery curves in monitoring wells in close proximity to the pumping well was very sensitive to the hydraulic conductivity (K) of the local sediments. The hydraulic conductivity in the sediments along the toe of the CVD had to be increased to $5x10^{-4}$ m/s to match the observed drawdown at PW1 and PW2. This K estimates is identical to the "effective" K estimated from the pumping test at PW1 but lower than the "effective" K estimated for the pumping test at PW2. The K of the sediments in the immediate vicinity of PW-2 had to be locally increased even further (to $5x10^{-3}$ m/s) to provide a good match with the test response in PW-2, MW-2 and P05-03. The presence of such a high K zone is consistent with the higher T estimate for the pumping test on the south side (PW2) and air venting in the near-by monitoring wells MW-2 and P05-03 during well development (see section 2.1.2).
- The shape of the drawdown and recovery curves in monitoring wells at greater distance from the pumping well was very sensitive to the assumed storage parameter of the aquifer. The very fast response to pumping in monitoring wells at greater distances (50-200m) suggested generally low specific storage values (10^{-3} to 10^{-4}) characteristic of a confined aquifer. Only the northern portion of the valley sediments (near PW1) responded like an unconfined aquifer (Sy=10%). It should be noted that the aquifer below the CVD and the PP remained confined

throughout the test (i.e. groundwater levels remained above ground surface due to the presence of the PP);

- The analysis of the pumping test at PW1 (north side) was complicated by the fact that the pumping well is screened in two layers separated by lower permeability silts. As a result, the relative rate of pumping from each screen is unknown. Furthermore, the upper screen desaturated almost completely throughout the test. For the purpose of this analysis, we assumed a constant pumping ratio proportional to the screen length (33% in model layer 2 and 67% in model layer 4). The relatively poor match of the calibrated model for PW1 and P05-02 for this test (Figure 4.10) suggests that (i) a higher proportion of groundwater was pumped from the upper screen (at least initially) and/or (ii) the upper screened sediments are more permeable than the lower screened sediments. However, a more detailed analysis of the pumping test at PW1 was beyond the scope of this study;
- The deep piezometer P03-09-01 screened in till showed a much higher drawdown during pumping at PW1 than simulated by the model (Figure 4.10). The higher drawdown would suggest a higher K in this unit. However, the calibrated model simulated the response in the same piezometer very well during pumping at PW2 (Figure 4.11). These results suggest small-scale heterogeneity and/or anisotropy in the basal till (and possibly the underlying bedrock) which could not be addressed within the scope of this study.

4.4.3 Discussion

Table 4.7 shows the simulated water balance for the calibrated groundwater flow model. The primary sources of water into the Rose Creek aquifer (within the reach of the model domain) include:

- Underflow of groundwater into the model domain at upstream boundary (28.5 L/s);
- Net seepage from the Intermediate Pond (9.3 L/s);
- Net seepage from the Polishing Pond (14.3 L/s); and
- Leakage from the Rose Creek Diversion Canal (38.7 L/s).

The simulated seepage rate from the Intermediate Pond is about three times higher than modeled in the 2003 GLL model (3.3 L/s) but significantly lower than the (approximate) seepage rate estimated earlier using changes in the pond level during winter base flow (30 L/s, see Robertson GeoConsultants Inc., 2006). The simulated seepage rate from the Polishing Pond is less than 50% of the seepage modeled in the 2003 GLL model (33.8 L/s) but higher than estimated in the water and load balance study (9.4 L/s, see Robertson GeoConsultants Inc., 2006). Note, however, that the seepage rates estimated for the water and load balance study represented fairly rough estimates based on changes in the pond level during winter base flow.

As noted earlier, the simulated leakage from the Rose Creek Diversion Canal is lower than the October 2004 survey estimate but agrees well with the April 2005 survey (40 L/s). Note that most of the simulated Rose Creek Diversion leakage (~33.5 L/s) occurs downstream of the Cross Valley Dam. More accurate flow measurements along the Rose Creek Diversion Canal would be required to further constrain the model for this important water balance component. Note that the simulated leakage from the Rose Creek Diversion Canal is significantly higher than the leakage simulated in the earlier 2003 GLL model (1.4 L/s).

The calibrated flow model suggests that the total flux of groundwater passing beneath the Cross Valley Dam is about 58 L/s. This flux estimate is near the lower bound of groundwater fluxes initially estimated from the pumping test data using Darcy's Law (45 – 147 L/s, see table 3.4). Assuming all groundwater flowing beneath the Cross Valley Pond requires collection and treatment (some time in the future), the seepage interception system at the toe of the Cross Valley Dam would have to intercept at least ~60 L/s. Note that the simulated groundwater flux at the Cross Valley Dam is about 15% lower than simulated by the 2003 GLL model (66.7 L/s).

Table 4.7

Simulated water balance for Rose Creek aquifer.

FLUXES	Flux In L/s	Flux Out L/s
Upstream Underflow	28.5	-
Intermediate Pond Seepage	9.3	-
Polishing Pond Seepage	14.2	-
RCDC Leakage	38.7	-
Toe Seepage (X13)	-	30.7
GW Discharge to CVS1	-	38.9
GW Discharge to Rose Creek (d/s of X10)	-	10.5
Downstream Underflow	-	10.6
Total	**90.7**	**90.7**

The primary outflows from the Rose Creek aquifer (all downstream of the Cross Valley Dam) include:

- Toe seepage immediately d/s of Cross Valley Dam at X13 (30.7 L/s);
- Discharge of groundwater along the drainage ditch between X13 and confluence with Rose Creek (38.9 L/s);
- Discharge of groundwater to Rose Creek channel (10.5 L/s); and
- Underflow of groundwater out of model domain at downstream boundary (10.6 L/s).

The simulated seepage intercepted by the toe drains (at X13) and the downstream drainage ditch (a combined total of 69.6 L/s) falls within our target range (based on field measurements) of 66 to 82 L/s (Table 4.3). These observed and simulated seepage rates are about two times higher than simulated in the 2003 GLL model (34.4 L/s).

The simulated discharge to Rose Creek (between X14 and model boundary) is significantly lower than earlier estimates of stream flow gains along this reach (~125 L/s in October 2004 and 150 L/s in April 2005, see Robertson GeoConsultants Inc., 2006). These large net gains in stream flow could not be reproduced by the model without also increasing seepage flows to the drainage ditch significantly. Note that stream water quality did not change in Rose Creek along this reach despite these (apparent) large increases in flow. It therefore appears doubtful whether the measured net gains in stream flow indeed represent groundwater inflow. Additional field work would be required

(including flow measurements and groundwater characterization) to better quantify and explain stream flow gains along this reach of Rose Creek.

The underflow of groundwater leaving the downstream model domain (10.6 L/s) is also smaller than predicted by the 2003 GLL model (32.3 L/s) and 2005 water and load balance model (52 – 64 L/s, Robertson GeoConsultants Inc., 2006). The relatively low rates of underflow leaving the model domain at this boundary are a result of the very low hydraulic gradients in this area (inferred from the meandering stream channel). Again, additional groundwater characterization work would be required to better constrain the groundwater flow model in this part of the model domain (no monitoring wells are available in this area).

Figure 4.12 shows the trajectories of particles released in the Rose Creek aquifer (in all model layers) beneath the Cross Valley Dam (using MODPATH) superimposed on the calibrated (steady-state) groundwater flow field. This figure illustrates the fate of the potentially impacted groundwater that is moving beyond the Cross Valley Dam. According to the calibrated model, the considerable leakage from the RCDC "pushes" the underflow beneath the Cross Valley Dam towards the central and northern portion of the aquifer where most of the flow discharges to the drain system. The calibrated model suggests that only a small portion of this potentially impacted groundwater is currently continuing further downgradient along the far northern side of the Rose Creek aquifer.

This simulated flow pattern agrees very well with the observed spatial pattern in groundwater quality downstream of the Cross Valley Dam, in particular the very low sulphate concentrations observed on the southern and central portion of the valley (at X16 and X17) relative to the northern portion of the valley (at X18 and P01-01).

4.5 Model Limitations

The groundwater flow model for the Rose Creek aquifer along the reach of the Cross Valley Dam has been calibrated successfully using both steady-state conditions as well as transient pumping test responses on the north and south-central side of the valley. However, while a good calibration with the calibration targets has been achieved, the calibration is not unique and other combinations of model parameters may equally (or potentially better) explain the observed aquifer responses.

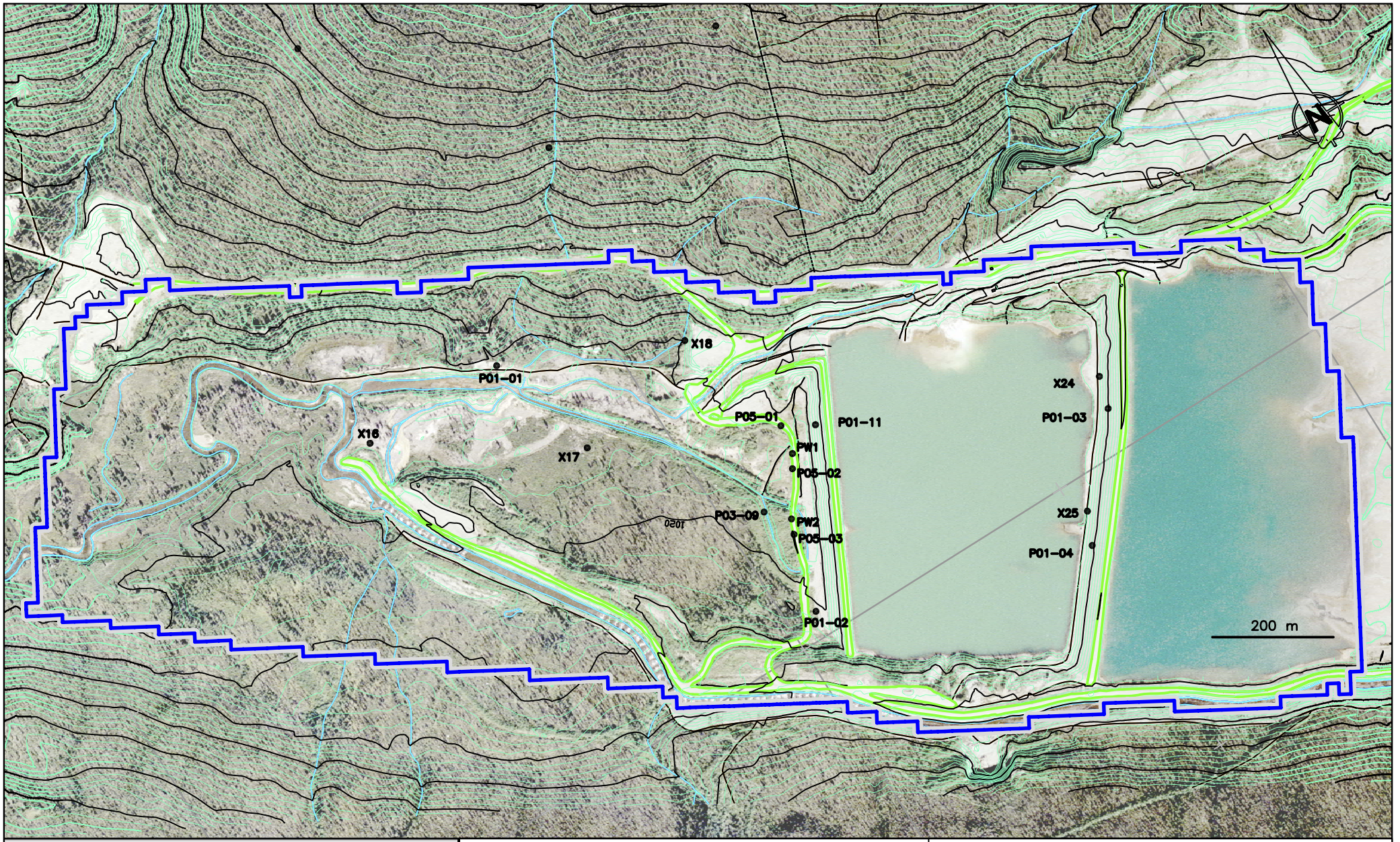
Furthermore, the groundwater flow model still represents a rather simplified representation of actual subsurface conditions. These simplifying assumptions should be considered when interpreting any predictions made with the model. The key simplifying assumptions of the model that may require further scrutiny include:

- The complex geology in the Rose Creek valley can be simplified into a set of five major hydrostratigraphic units (glacio-fluvial sediments, colluvium, slope till, basal till and bedrock) with representative ("effective") hydraulic parameters. In practice, the hydraulic properties of these hydrostratigraphic units can be expected to vary significantly in space and depth. A more detailed representation of the local geology was beyond the scope of this modeling exercise.
- The glacio-fluvial sediments were assumed to be homogeneous and isotropic throughout the depth of the aquifer, i.e. the three model layers representing the sediments were assigned the same K within a given zone. In reality, drilling indicated significant vertical variation in material grading ranging from very coarse gravel layers to fine-grained silt. An explicit

representation of this small-scale heterogeneity and/or the use of vertical anisotropy was beyond the scope of this study;

- The anthropogenic units (Intermediate Dam and Pond as well as CVD and PP) are highly simplified in the existing model. A more detailed representation of the dam structures (including dam core and drainage elements) would potentially improve the model predictions related to toe seepage downstream of the CVD;
- Recharge from the valley sides can be assumed to be negligible. This simplifying assumption is probably adequate for the immediate vicinity of the CVD (where pond seepage is a major component of groundwater flow). However, recharge from the valley sides may influence the piezometric heads on the valley sides (e.g. at X18A/B, P01-02A/B) and in bedrock/till in the valley center (e.g. P01-09-01) and may also represent a significant component of the water balance for the reach downstream of the CVD;
- The groundwater levels near the downstream boundary of the model domain (downstream of X14) were assumed to be very close (within ~0.5m) to the (inferred) water level in Rose Creek. No information is presently available to better constrain the model in this part of the model domain.

We recommend that additional sensitivity analyses be carried out to assess the influence of these assumptions on model calibration and model predictions.



Groundwater Flow Model Boundary

CLIENT:	Deloitte & Touche Inc.
PROJECT:	Rose Creek Groundwater Interception
REPORT:	118004
FILE:	fig4-1 domain boundary.dwg
LOCATION:	Faro Mine Yukon Territory, Canada

ROBERTSON GEOCONSULTANTS INC.
Consulting Geotechnical and Environmental Engineers

Domain of Groundwater Flow Model

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		4.1

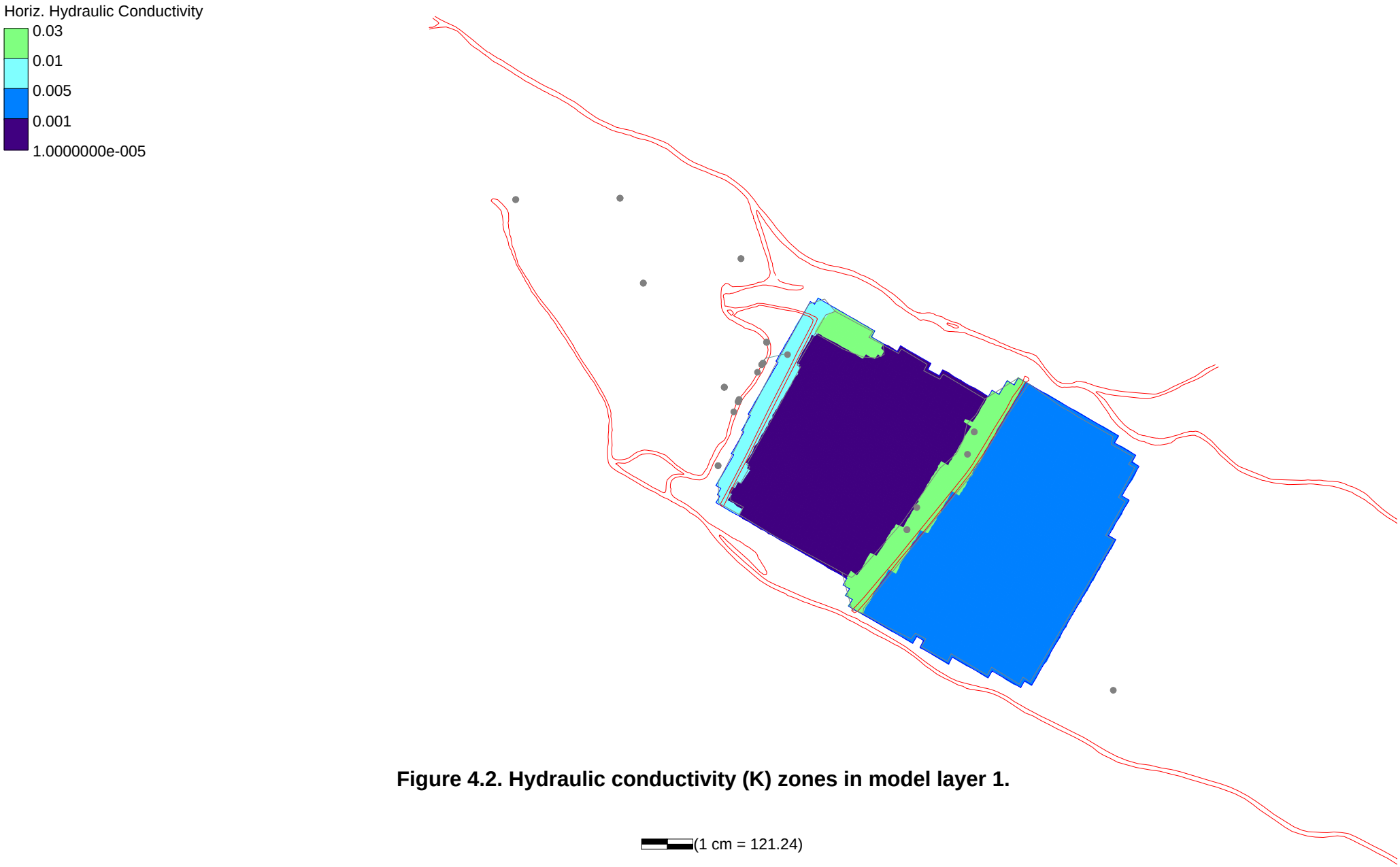


Figure 4.2. Hydraulic conductivity (K) zones in model layer 1.

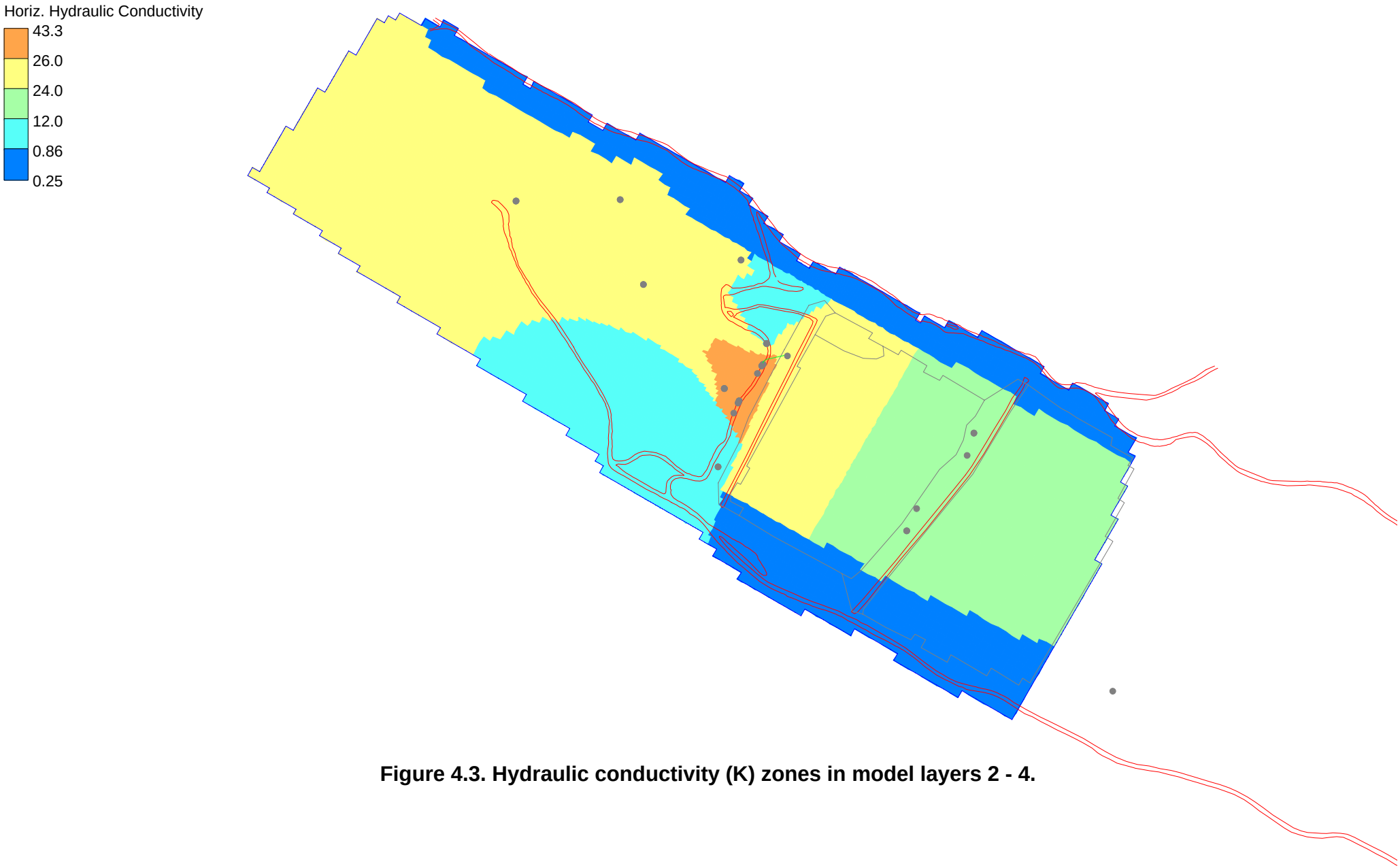


Figure 4.3. Hydraulic conductivity (K) zones in model layers 2 - 4.

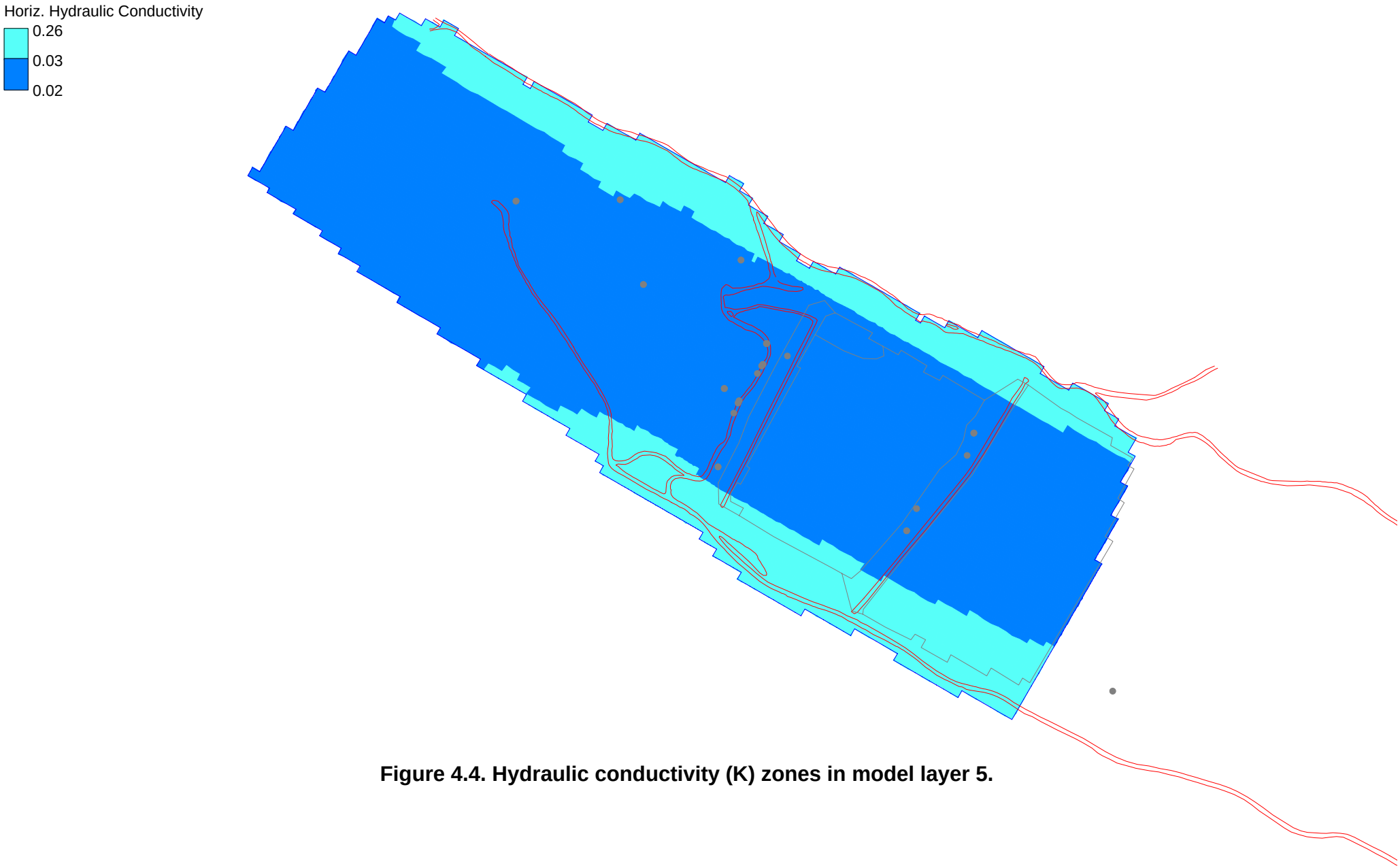


Figure 4.4. Hydraulic conductivity (K) zones in model layer 5.

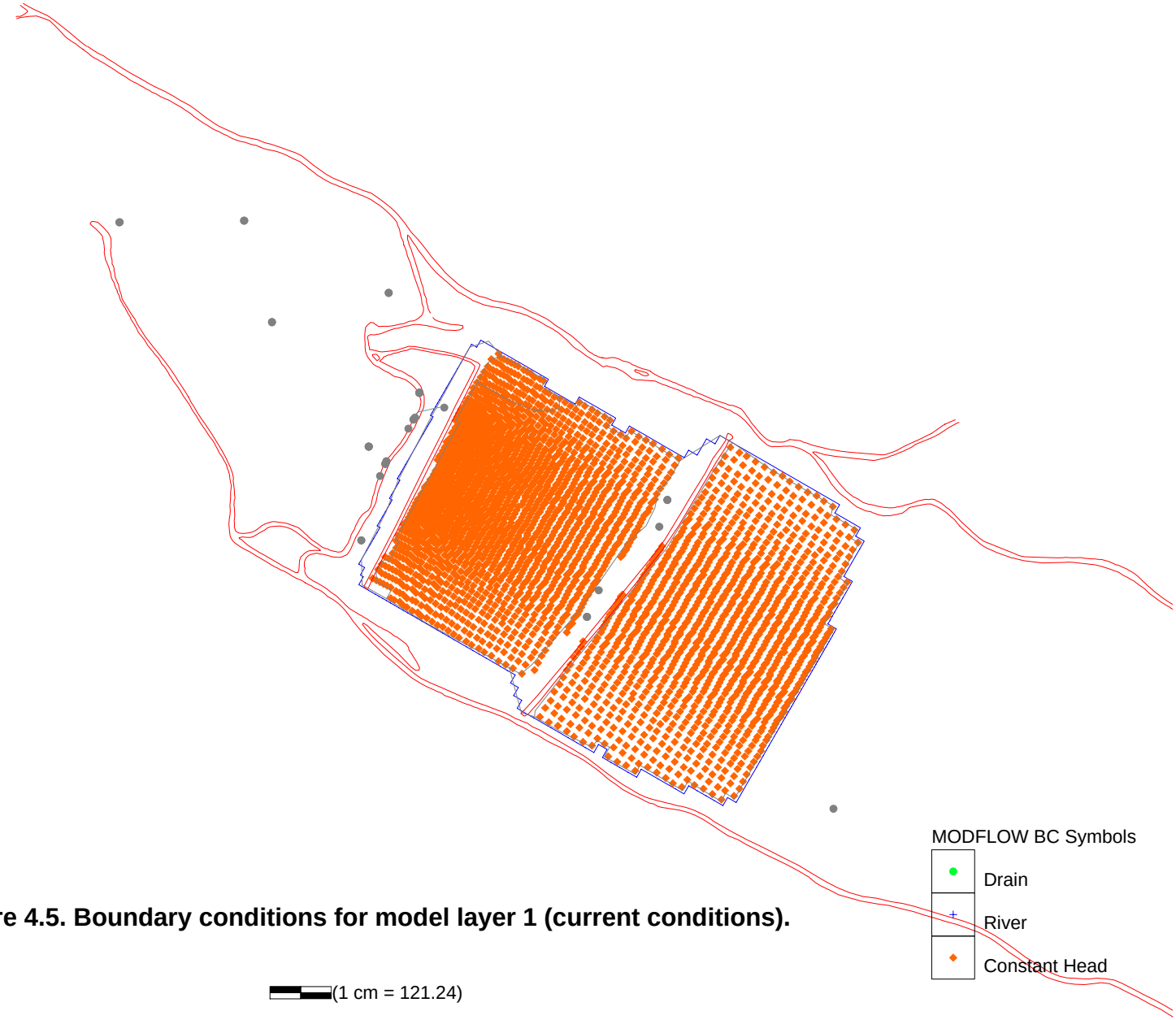


Figure 4.5. Boundary conditions for model layer 1 (current conditions).

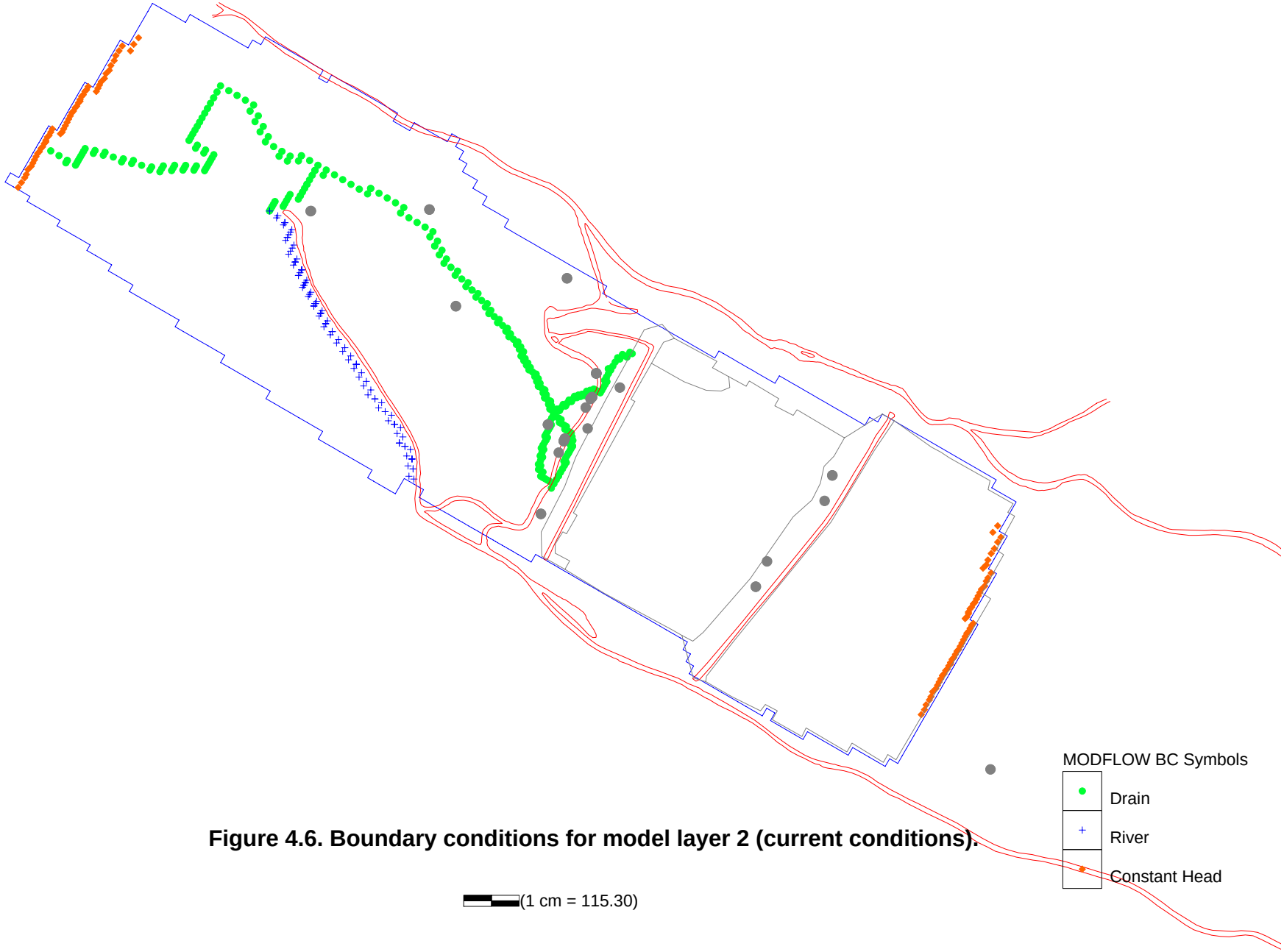


Figure 4.6. Boundary conditions for model layer 2 (current conditions).

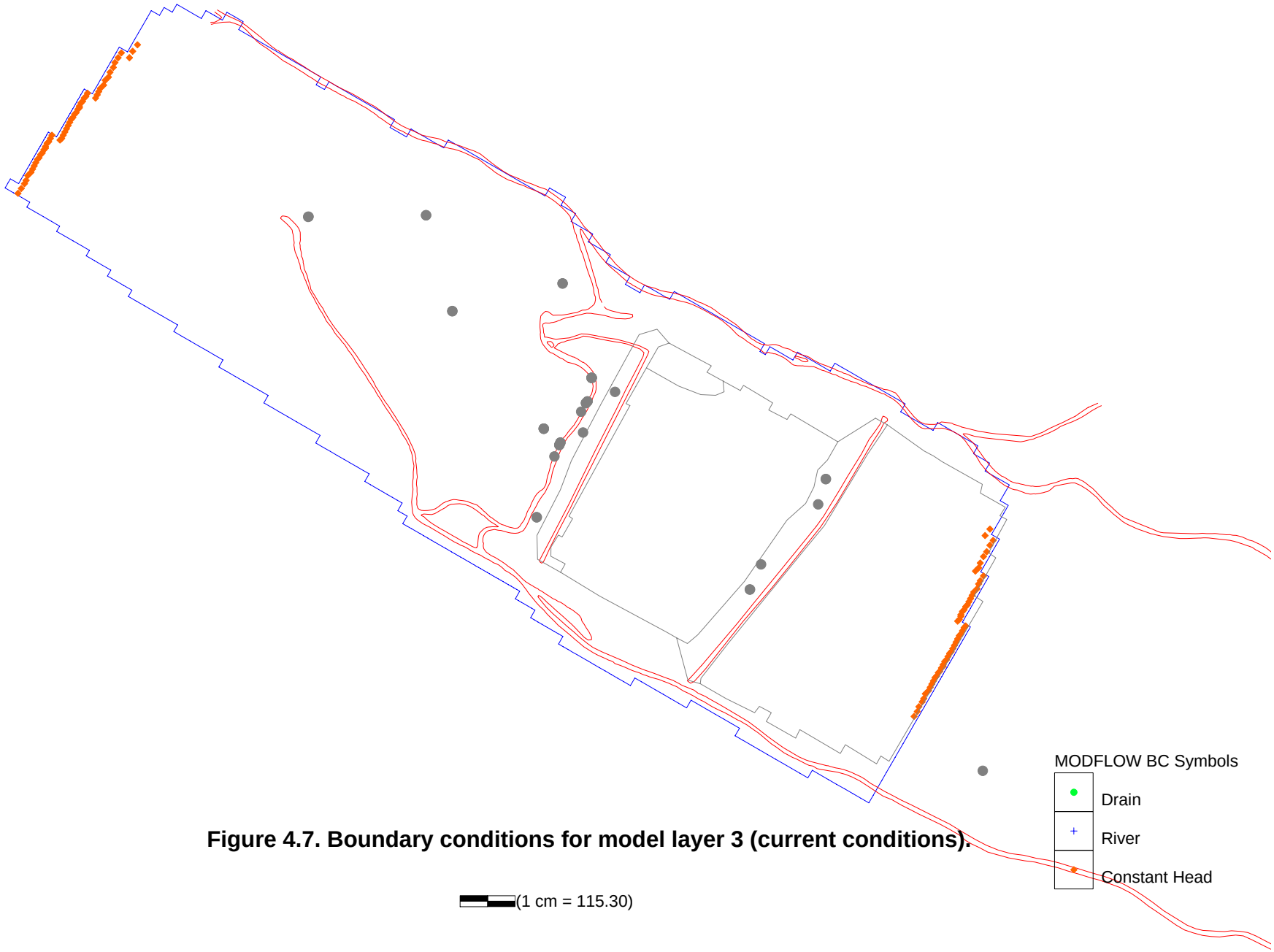


Figure 4.7. Boundary conditions for model layer 3 (current conditions).

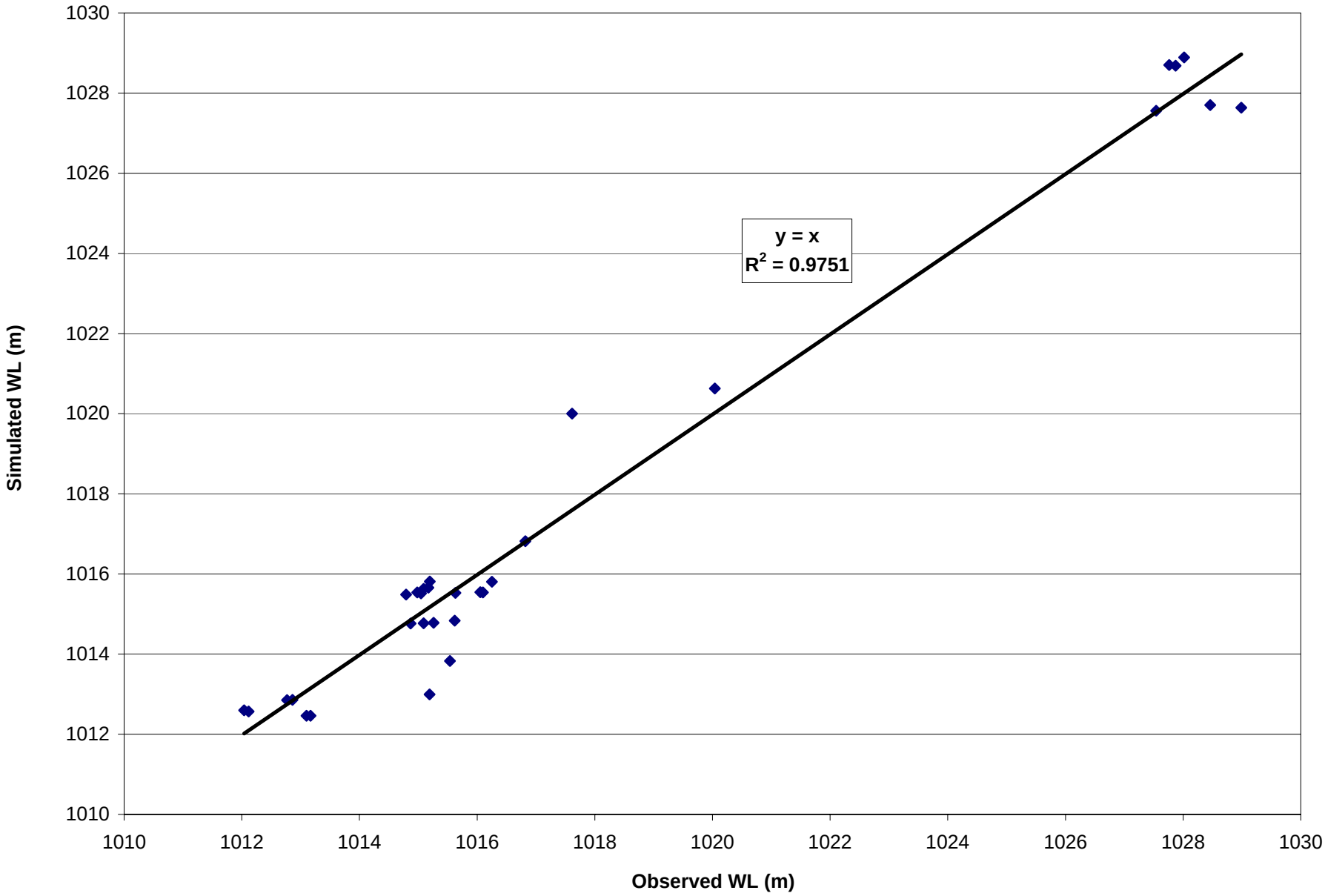


Figure 4.8. Simulated versus observed groundwater levels (steady-state calibration).

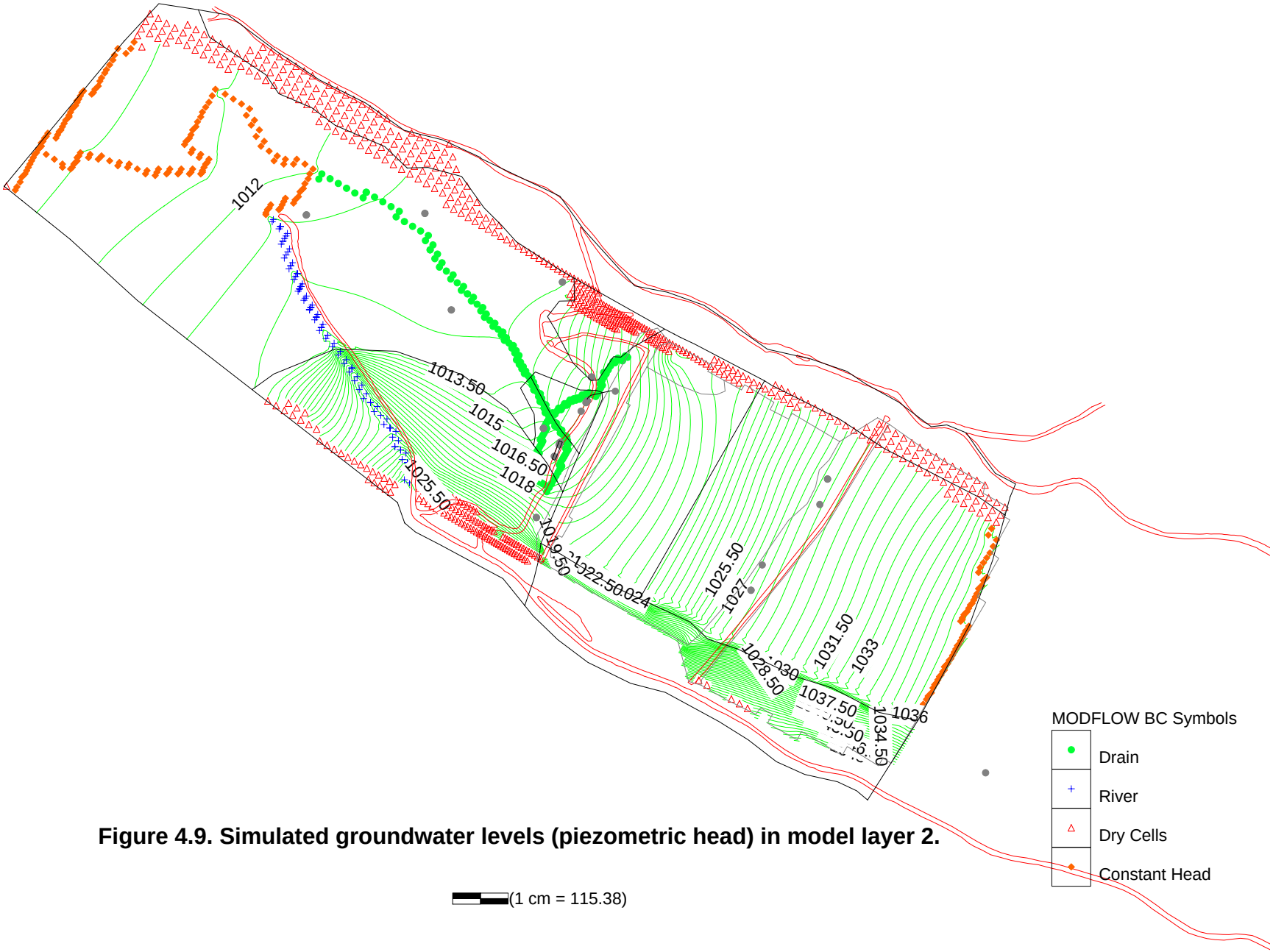


Figure 4.9. Simulated groundwater levels (piezometric head) in model layer 2.

Figure 4.10. Simulated drawdown and recovery versus time for the 52-hr pump test in PW-1.

Material	Layer	K (m/d)	K (m/s)	Sy/Ss
CVD	1	0.00864	1.0E-07	0.1 / -
PP Seds	1	0.000078	9.0E-10	- / 0.0001
PP Window	1	0.026	3.0E-07	- / 0.0001
ID	1	0.026	3.0E-07	0.1 / -
Tailings	1	0.0017	2.0E-08	- / 0.0001
Coll N	2, 3, 4	0.864	1.0E-05	0.001/0.001
Coll S	2, 3, 4	0.864	1.0E-05	0.0001/0.0001
U/S All.	2, 3, 4	12.96	1.5E-04	0.001/0.001
D/S All.	2, 3, 4	25.92	3.0E-04	0.001/0.001
All. (CVD-N	2, 3, 4	43.2	5.0E-04	0.1/0.001
All. (CVD-S	2, 3, 4	43.2	5.0E-04	0.0001/0.0001
All. PW2	2, 3	432	5.0E-03	0.0001/0.0001
Basal Till	5	0.026	3.0E-07	0.01 / 1e-6
Slope Till	2, 3, 4, 5	0.26	3.0E-06	0.05 / 1e-6
Bedrock	6	0.017	2.0E-07	0.001 / 1e-6
Drain Conductance (m/d)			51.84	PW1 (33/67C)
River Conductance (m/d)			5.184	498 GPM

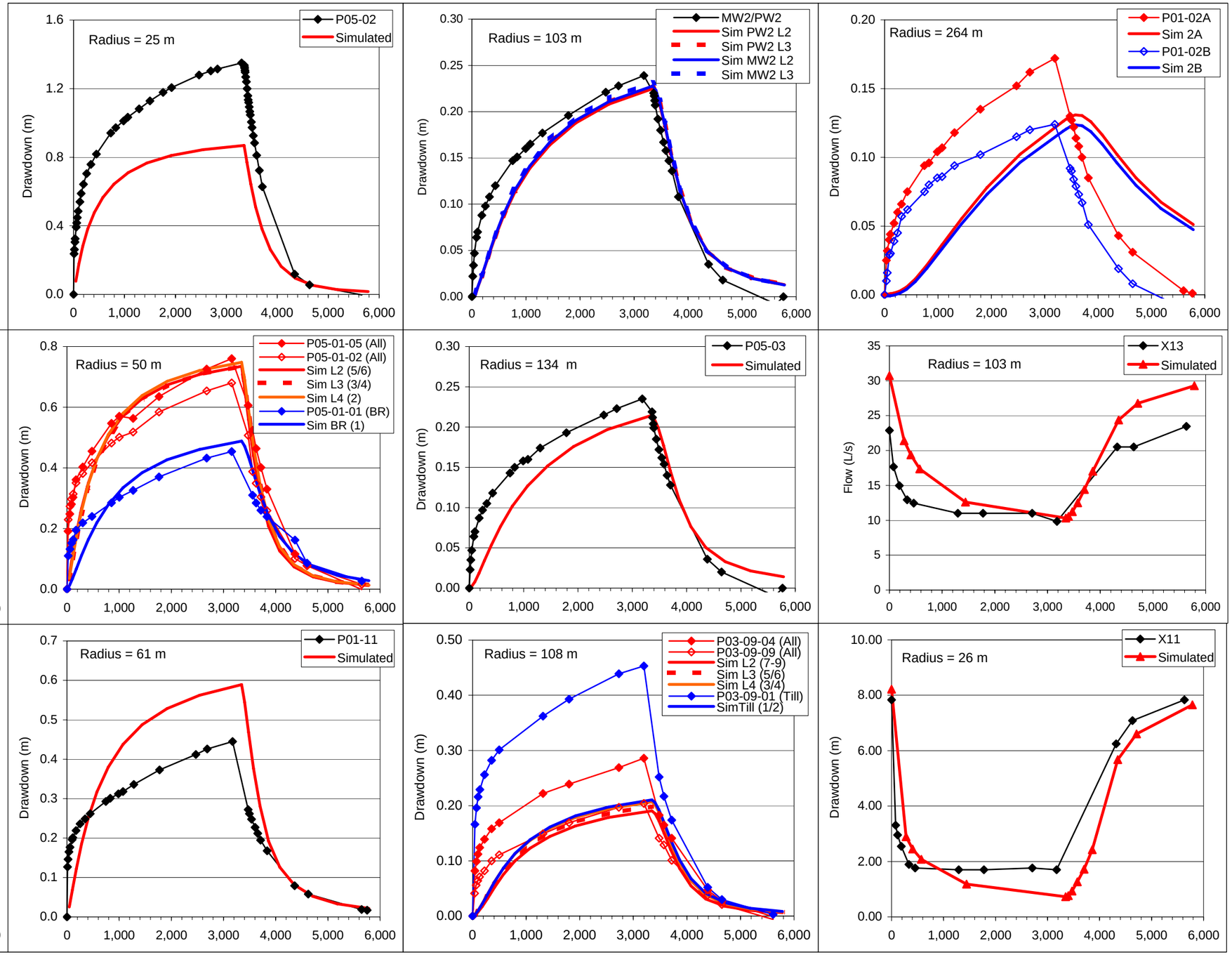
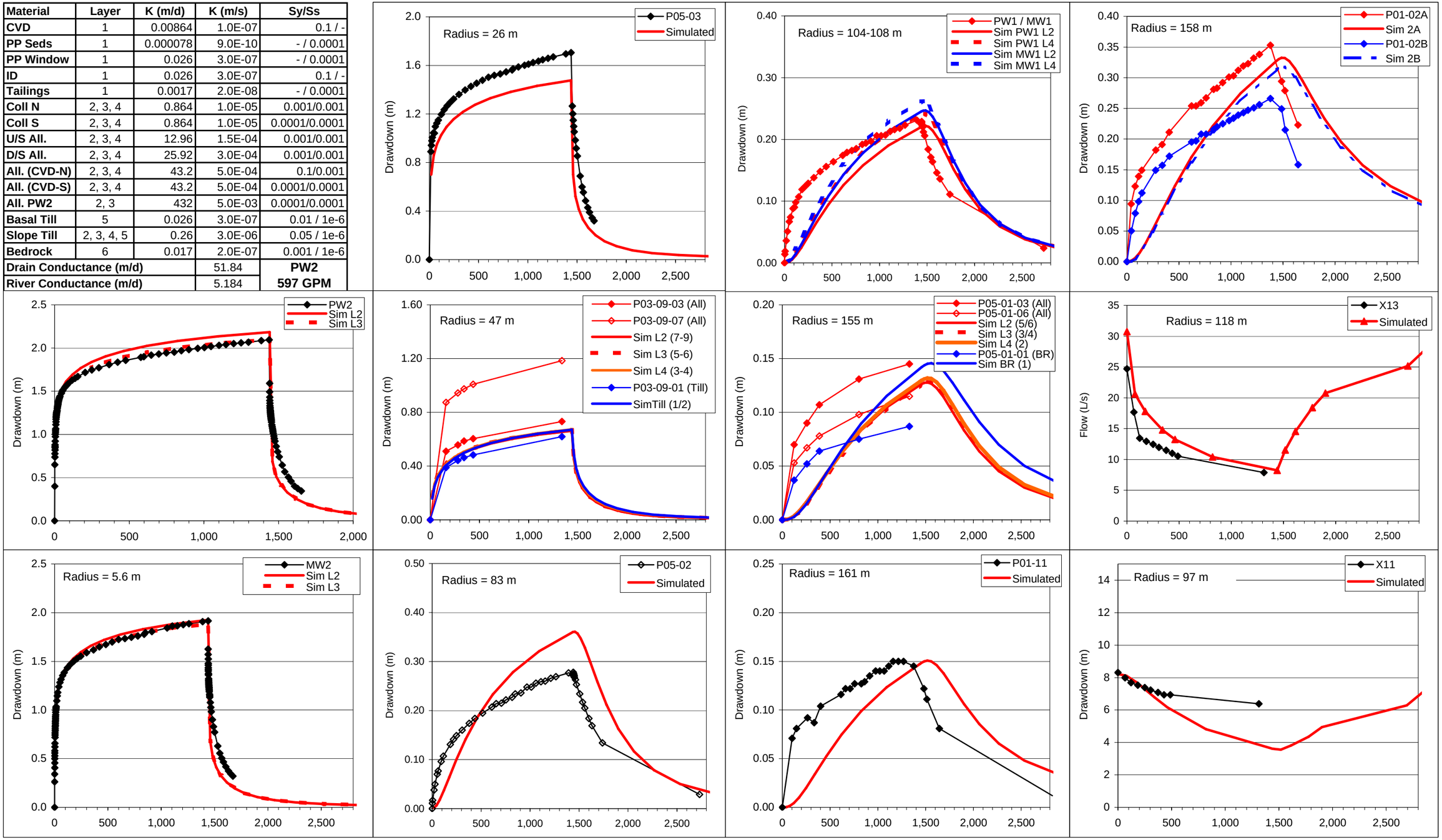


Figure 4.11. Simulated drawdown and recovery versus time for the 24-hr pumping test at PW-2.

Material	Layer	K (m/d)	K (m/s)	Sy/Ss
CVD	1	0.00864	1.0E-07	0.1 / -
PP Seds	1	0.000078	9.0E-10	- / 0.0001
PP Window	1	0.026	3.0E-07	- / 0.0001
ID	1	0.026	3.0E-07	0.1 / -
Tailings	1	0.0017	2.0E-08	- / 0.0001
Coll N	2, 3, 4	0.864	1.0E-05	0.001/0.001
Coll S	2, 3, 4	0.864	1.0E-05	0.0001/0.0001
U/S All.	2, 3, 4	12.96	1.5E-04	0.001/0.001
D/S All.	2, 3, 4	25.92	3.0E-04	0.001/0.001
All. (CVD-N)	2, 3, 4	43.2	5.0E-04	0.1/0.001
All. (CVD-S)	2, 3, 4	43.2	5.0E-04	0.0001/0.0001
All. PW2	2, 3	432	5.0E-03	0.0001/0.0001
Basal Till	5	0.026	3.0E-07	0.01 / 1e-6
Slope Till	2, 3, 4, 5	0.26	3.0E-06	0.05 / 1e-6
Bedrock	6	0.017	2.0E-07	0.001 / 1e-6
Drain Conductance (m/d)			51.84	**PW2**
River Conductance (m/d)			5.184	**597 GPM**



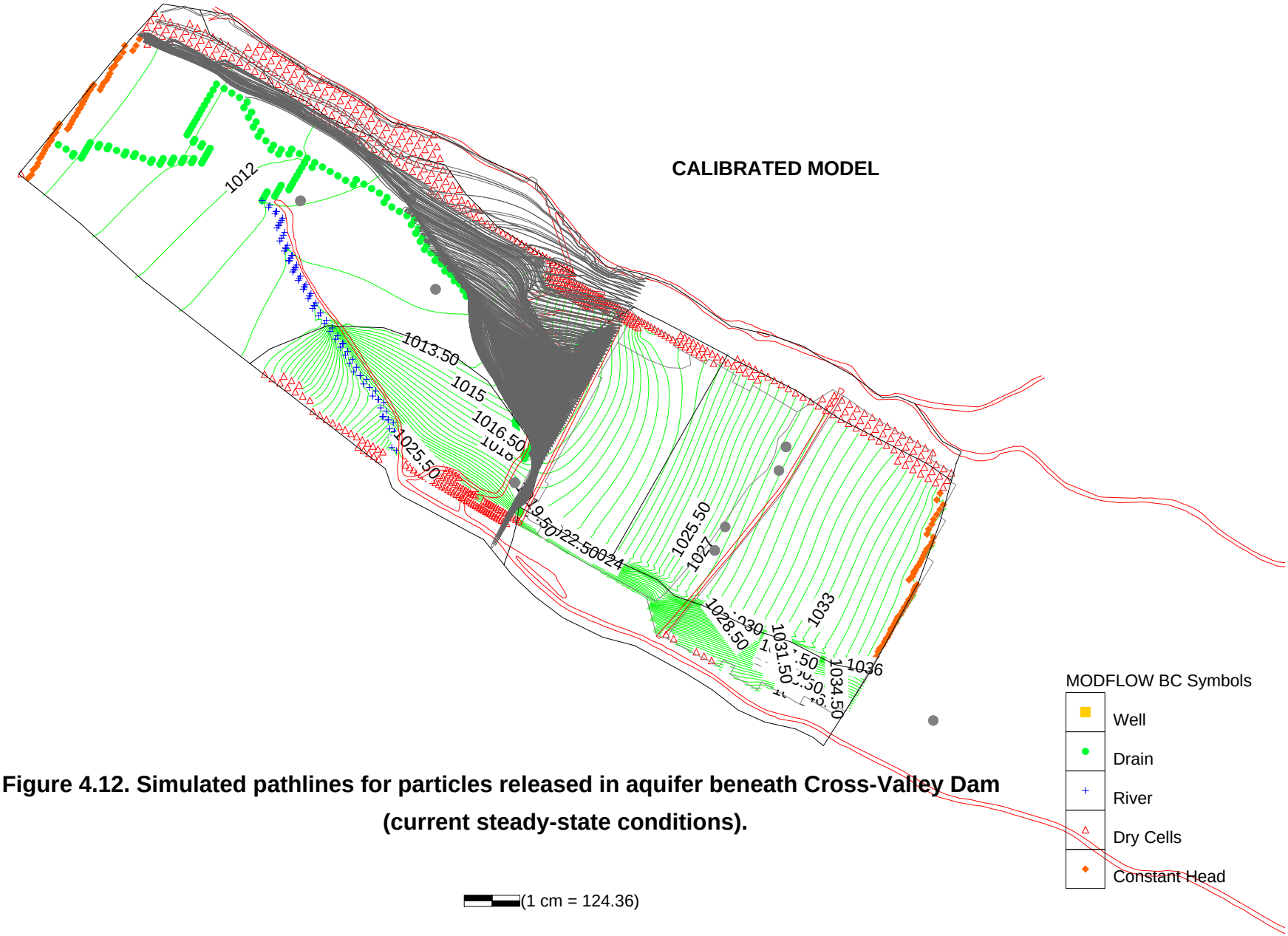


Figure 4.12. Simulated pathlines for particles released in aquifer beneath Cross-Valley Dam (current steady-state conditions).

5 SIMULATION OF GROUNDWATER INTERCEPTION SCENARIOS

5.1 Overview

The calibrated groundwater flow model was used to predict the performance of alternative groundwater interception scenarios at the toe of the Cross Valley Dam. The primary design criteria to be assessed with the model included:

- Number & location of pumping wells,
- Required depth of pumping wells;
- Individual and total pumping rates

The majority of groundwater interception scenarios were simulated for calibrated conditions, i.e. assuming the current configuration of dams, drains and diversions. The following four groundwater interception scenarios were evaluated assuming current conditions:

Scenario 1: Five pumping wells at ~100m spacing ("full penetration");

Scenario 2: Ten pumping wells at ~50m spacing ("full penetration");

Scenario 3: Ten pumping wells at ~50m spacing ("partial penetration");

Scenario 4: Ten pumping wells at ~50m spacing ("full penetration") PLUS cut-off wall across the entire Rose Creek aquifer (at X13).

Figures 5.1 to 5.3 show the assumed locations of the pumping wells and cut-off wall for the different interception scenarios. In all scenarios, the pumping wells were assumed to be located along the access road at the toe of the Cross Valley Dam. Note that the two recently installed pumping wells (PW-1 and PW-2) were included into the fence of pumping wells[4]. Scenarios 1, 2 and 4 assume that all pumping wells (except PW-1 and PW-2) are screened across the entire depth of the permeable sediments. In Scenario 3, all pumping wells were assumed to be partially screened (i.e. only in the upper, middle or lower portion of the permeable sediments). Scenario 4 evaluates the benefit of installing a low-permeability cut-off downgradient of the fence of pumping wells (see Figure 5.3 for location).

The groundwater flow model was also used to provide a preliminary assessment of groundwater interception for two different closure scenarios:

Scenario 5: Rose Creek Diversion Canal fully lined (i.e. no leakage from RCDC)

Scenario 6: Cross Valley Dam and Polishing Pond removed (i.e. no seepage from PP).

For both of those potential future closure scenarios, a fence of ten fully penetrating pumping wells (~50m spacing as shown in Figure 5.2) was assumed to allow a direct comparison with Scenario 2.

[4] Note that the true distance between PW-1 and PW-2 is ~107m. The inclusion of those existing pumping wells resulted in small variations from the "nominal" well distances listed for each scenario. Also note that PW-1 and PW-2 are not fully screened across the entire aquifer and were therefore simulated as partially penetrating wells in all scenarios.

5.2 Numerical Methods

All groundwater interception scenarios were simulated assuming steady-state conditions, i.e. continuous pumping from the pumping wells.

Figures 5.4 to 5.6 show the boundary conditions for the interception scenarios 1 to 4. The boundary conditions were essentially identical to those used for the calibrated groundwater flow model except for the use of the interception measures (pumping wells and cut-off wall). The pumping wells were simulated assuming a constant drawdown rather than a constant pumping rate. This approach was implemented in MODFLOW by assuming a specified (constant) head at the nodes representing the well screen. In scenarios 1, 2 and 4 (representing "fully penetrating" wells) constant heads were used in model layers 2 to 4[5]. In scenario 3 (representing partially penetrating wells), the constant heads were only assigned to one of the three model layers representing the permeable sediments (model layers 2, 3 or 4).

For scenario 4, a cut-off wall was simulated using the Horizontal Flow Barrier (HFB) package of MODFLOW (Figure 5.6). For the purpose of this study, we assumed a nominal thickness of 1m and an effective hydraulic conductivity of $1x10^{-8}$ m/s for the cut-off wall. This hydraulic conductivity is consistent with effective permeabilities reported for this type of installation in the literature (Burke, 1992). The flow barrier was assumed to be installed parallel to the Cross Valley Dam (i.e. perpendicular to the main valley) running through monitoring station X13 (see Figure 5.3 for assumed alignment). On the northern side the flow barrier was assumed to extend to the model domain boundary. On the southern side the flow barrier was assumed to terminate just north of the Rose Creek diversion. The cut-off wall was assumed to be installed to various depths ranging from 30m to 45m. Cut-off walls can be installed to depths up to 50-60 m using the trench remixing and deep wall method (TRD) (Angelo, 2005).

The only other change in boundary conditions from the calibrated flow model was the use of constant heads (rather than drain nodes) for the Rose Creek stream channel (Figures 5.4 -5.6). This change in the type of internal sink/source was required to allow leakage from the Rose Creek into the aquifer in the case of significant drawdown (below the streambed elevation) due to pumping.

Figures 5.7 and 5.8 show the boundary conditions for the interception scenarios 5 and 6, respectively. These two scenarios required further changes to the boundary conditions of the groundwater flow model. In scenario 5, the constant heads and river drains representing leakage from the RCDC were removed. This change in the boundary conditions implies that the RCDC is perfectly sealed and is no longer a source of water to the model. Note, however, that the Rose Creek diversion canal is still assumed to be physically present as it provides water to the historic Rose Creek stream channel (which is modeled using constant head boundaries).

Scenario 6 assumes removal of the Cross Valley Dam and the Polishing Pond. This configuration was implemented by removing the cells (in model layer 1) representing the dam material and the PP sediments (including the constant heads). Initial simulations with this setup indicated significant flooding in the former foot print area of the Polishing Pond. In practice, this toe seepage from the

[5] For the existing pumping wells the constant head nodes were assigned to the model layer closest to the actual screening intervals: at PW-1, constant head nodes were assumed in model layers 2 and 4; at PW-2, constant head nodes were assumed in model layers 2 and 3.

Intermediate Dam would be collected and pumped to a treatment plant for treatment. This collection was simulated in the model by introducing drain nodes along the toe of the Intermediate Dam (Figure 5.8). The elevation of the drain nodes was set to the inferred natural ground elevation (approximately 1016.5 m AMSL in the southern and central area and gradually increasing to 1023.5 m AMSL on the northern side). Additional drain nodes were also used on the southern side of the valley to prevent flooding in this area due to leakage from the RCDC. However, the intercepted flows along this drain arc were much smaller than in the toe drain at the Intermediate Dam.

The use of a new toe drain at the Intermediate Dam initially resulted in a large (about 100%) increase in underflow from the upstream boundary into the model domain. In our opinion, this large increase in underflow is unlikely to occur because the sources for groundwater flow are finite. Instead, groundwater levels at the upstream boundary of the model are likely to drop due to the removal of the Polishing Pond. For the purpose of this study, the constant head at the upstream boundary was subsequently reduced by about 7m to maintain a roughly equal underflow into the model domain. This is a preliminary assumption and additional work would be required to better understand the influence of the removal of the Polishing Pond on groundwater flow in the Rose Creek aquifer. Note, however, that the assumed constant head along the upstream boundary primarily influences the groundwater flow discharging to the toe drain at the Intermediate Dam, not the groundwater flow intercepted at the (former) Cross Valley Dam.

For interception scenarios 1 and 2, the drawdown in the pumping wells was systematically varied to assess the relationship of drawdown to groundwater collected and groundwater by-passing the collection system. Note that the rate of groundwater by-pass is a better indicator of system performance than total flow collected since the total amount of (potentially contaminated) groundwater to be collected varies between the different scenarios. The amount of groundwater by-pass for a given scenario was assumed to be equal to the simulated groundwater flow leaving a cross-section across the entire aquifer (including all model layers) at X13. This is a conservative (high) estimate of by-pass since some of this groundwater may indeed still be captured (by reversing flow and re-entering the section at a different location).

For the purpose of this study, groundwater by-pass is expressed in both absolute terms (in L/s) as well as relative terms (%). The relative by-pass was calculated by comparing the rate of by-pass to the simulated rate of groundwater flux beneath the Cross Valley Dam (in all model layers). This approach implies that all groundwater flowing in the Rose Creek aquifer beneath the Cross Valley Dam is considered sufficiently contaminated to require collection and treatment. Note that this is a conservative approach as certain portions of the Rose Creek aquifer (for example the southern side of the aquifer which is currently not significantly impacted) may not require collection and treatment for a long time, if at all. Nevertheless, this conservative assumption was judged to be prudent considering the potential for long-term impacts from all parts of the Rose Creek tailings facility located immediately upstream.

For each scenario and sensitivity run, the capture zone and by-pass for the interception system was visually illustrated using particle tracking. For this purpose, particles were released across all model layers beneath the Cross Valley Dam using MODPATH[96] (Version 3.0). It should be noted that the number of particles released in each cell are independent of the groundwater flux through a given cell (we used 6 particles per cell in this application). As a result, the particle tracking plots only provide a qualitative tool for assessing the degree of capture (or by-pass) for a given system. System

performance was therefore entirely judged by comparing the absolute (and relative) by-pass flows calculated from the flow budget (see above).

5.3 Results and Discussion

Table 5.1 summarizes all results of the various interception scenarios simulated. The first two columns identify the scenario and model run. The next five columns summarize the assumed system components (# of pumping wells and assumed drawdown, depth of cut-off wall). The remaining five columns summarize the simulated performance of the interception scenario, including the water collected by the system (as toe seepage or pumped groundwater) and the amount of groundwater by-pass (in L/s and % of groundwater flow under the Cross Valley Dam). Summary output sheets for each individual model run, showing individual pumping rates and drawdown for each pumping well as well as plots of simulated heads and drawdown, are provided in Appendix H.

Figures 5.9 to 5.18 show simulated groundwater levels (piezometric surface in layer 2) for selected scenarios. The simulated path lines of particles released beneath the Cross Valley Dam are superimposed on each figure to illustrate capture and by-pass.

In the following we briefly summarize the model findings for each scenario.

5.3.1 Scenario 1: Five Pumping Wells (Full Penetration)

Scenario 1 assumes pumping from the two existing pumping wells (PW-1 and PW-2) located in the central “high K” zone and three additional pumping wells located in the northern and southern “low K” zones (Figure 5.1).

The rate of groundwater pumped from the interception system is primarily a function of the drawdown in the two pumping wells located in the high K zone. For example, a 2m drawdown at PW-1 and PW-2 (Scenario 1c) extracts 45.8 L/s but allows 16 L/s by-pass. In contrast, a 5m drawdown in both pumping wells (Scenario 1f) is predicted to extract 72.9 L/s of groundwater resulting in a residual by-pass of only about 0.3 L/s (Table 5.1).

Figures 5.9 and 5.10 provide a visual comparison of these two scenarios. In scenario 1c, the assumed drawdown of 2m is not adequate to overcome the regional hydraulic gradient and groundwater is by-passing between the pumping wells in all model layers (Figure 5.9). In Scenario 1f, the assumed drawdown of 5m lowers the groundwater levels downgradient of the Cross Valley Dam sufficiently to reverse the regional hydraulic gradient and “pull” groundwater from the downstream side towards the groundwater collection system (Figure 5.10). The only by-pass occurs along the far northern side (in till and bedrock). Note again, however, that the number of particles (i.e. path lines) by-passing the collection system is not representative of the flow rate.

The assumed drawdown in the three pumping wells in the low K zones is predicted to have relatively little influence on system performance. For example, increasing the drawdown from 5m to 10m (with 5m drawdown in the high K wells) increases groundwater capture by only 0.6 L/s and reduces groundwater by-pass by only 0.07 L/s (Table 5.1). The much lower pumping yields (and capture efficiencies) of these pumping wells are a result of the much lower (inferred) hydraulic conductivity in those areas compared to the central valley (about 50 times lower K).

Table 5.1

Summary of simulated performance of groundwater interception scenarios.

Scenario	MODFLOW File	Description					Water Collection (L/s)			System Performance	
		# Wells		Drawdown (m)		Cutoff Wall	Toe Seepage[1]	GW Pumped	Total Water Collected	Bypass Flow (L/s)	% Bypass
		High K	Low K	High K	Low K						
Scenario 1: 5 Wells Pumping at ~ 100 m spacing (Polishing Pond Included)											
1a	RCC33_2_1	2	3	1	1	n/a	9.3	27.9	37.2	23.6	39.8%
1b	RCC33_2_3	2	3	1	5	n/a	8	30.0	38.0	23.2	39.9%
1c	RCC33_2_4	2	3	2	5	n/a	1.3	45.8	47.1	16.0	26.2%
1d	RCC33_2_5	2	3	3	5	n/a	0.2	57.7	57.9	7.4	11.7%
1e	RCC33_2_6	2	3	4	5	n/a	0	67.4	67.4	1.0	1.6%
1f	RCC33_2_2	2	3	5	5	n/a	0	72.9	72.9	0.32	0.48%
1g	RCC33_2_7	2	3	5	10	n/a	0	73.5	73.5	0.2	0.35%
Scenario 2: 10 Wells Pumping at ~ 50 m spacing (Polishing Pond Included)											
2a	RCC33_2_8	4	6	2.5	2.5	n/a	0.2	57.7	57.9	7.2	11.3%
2b	RCC33_2_12	4	6	1	5	n/a	4.9	36.2	41.1	20.9	34.9%
2c	RCC33_2_13	4	6	2	5	n/a	0.6	52.2	52.8	11.6	18.6%
2d	RCC33_2_14	4	6	3	5	n/a	0	65.3	65.3	1.7	2.6%
2e	RCC33_2_15	4	6	4	5	n/a	0	72.0	72.0	0.28	0.42%
2f	RCC33_2_10	4	6	5	5	n/a	0	77.7	77.7	0.26	0.39%
2g	RCC33_2_11	4	6	5	10	n/a	0	78.5	78.5	0.19	0.28%
Scenario 3: 10 Wells Pumping at ~ 50 m spacing (Partial Penetration, Polishing Pond Included)											
3a	RCC33_2_17	4 - L2	6 - L2	5	5	n/a	0	63.9	63.9	3.77	5.88%
3b	RCC33_2_18	4 - L3	6 - L3	5	5	n/a	0	76.5	76.5	0.28	0.42%
3c	RCC33_2_19	4 - L4	6 - L4	5	5	n/a	0	74.1	74.1	0.32	0.47%
Scenario 4: 10 Wells Pumping at ~ 50 m spacing (Cutoff Wall at X13, Polishing Pond Included)											
4a	RCC36_2_1	4	6	2	5	L2	0.7	52.4	53.1	11.32	18.2%
4b	RCC36_1_1	4	6	2	5	L2-L5	2.3	61.2	63.5	0.31	0.49%
Scenario 5: 10 Wells Pumping at ~ 50 m spacing (RCDC Fully Lined, Polishing Pond Included)											
5a	RCC35_1_1	4	6	2	5	n/a	0.4	47.4	47.8	11.2	18.8%
5b	RCC35_1_2	4	6	5	5	n/a	0	67.2	67.2	0.3	0.48%
Scenario 6: 10 Wells Pumping at ~ 50 m spacing (Polishing Pond Removed)											
6a	RCC34_3_1	4	6	1	3	n/a	27.7	15.8	43.5	2.92	17.5%
6b	RCC34_3_2	4	6	3	3	n/a	22.5	30.9	53.4	0.16	0.73%

1. In closure scenarios 1-5 toe seepage is collected at X13; in closure scenario 6, toe seepage is collected at the toe of the Intermediate Dam.

For the purpose of this discussion we assumed that the groundwater interception system would require a capture efficiency of 99% (i.e. a by-pass of less than 1%) to be “acceptable”. Based on this criterion, a uniform drawdown of at least 5m in all five wells would be required for successful groundwater interception using 5 pumping wells (Scenario 1f).

5.3.2 Scenario 2: Ten Pumping Wells (Full Penetration)

Scenario 2 assumes pumping from the four pumping wells in the high K zone (including the two existing pumping wells PW-1 and PW-2) plus three additional pumping wells each in the northern and southern “low K” zones (Figure 5.5).

Figures 5.11 and 5.12 show the simulated groundwater levels and path lines for scenarios 2c and 2f (2m and 5m drawdown in the high K zone, respectively). The simulated flow fields using 10 wells are very similar to those discussed earlier for 5 pumping wells (c. Figures 5.9 and 5.10). Again, an assumed drawdown of 2m is not adequate to overcome the regional hydraulic gradient and groundwater is by-passing between the pumping wells in all model layers (Scenario 2c). In contrast, an assumed drawdown of 5m lowers the groundwater levels downgradient of the Cross Valley Dam sufficiently to reverse the regional hydraulic gradient and “pull” groundwater from the downstream side towards the groundwater collection system (Scenario 2f). Again, the only by-pass occurs along the far northern side (in till and bedrock).

Despite these similarities in the simulated groundwater flow field, the use of 10 pumping wells is predicted to provide an incremental improvement in system performance relative to the use of 5 pumping wells. For example, assuming 2m drawdown in the high K zone allows only 11.6 L/s (or 18.6%) by-pass using 10 wells (Scenario 2c) compared to 16.0 L/s (or 26.2%) by-pass using 5 wells (Scenario 1c, Table 5.1).

Using our criterion of <1% by-pass, a uniform drawdown of at least 4m in all wells would be required for successful groundwater interception using 10 pumping wells (Scenario 2e). In other words, the use of 10 pumping wells (instead of 5) would reduce the required drawdown by about 1m.

5.3.3 Scenario 3: Ten Pumping Wells w/ Partial Penetration

Scenario 3 assumes the same number and locations of pumping wells as for Scenario 2. However, this scenario evaluates the influence of the depth of screening of the pumping wells. The first sensitivity run (scenario 3a) assumed screening in model layer 2 only (to a depth of ~1010 m AMSL). Figure 5.13 shows the simulated groundwater flow field and path lines for this scenario. The modeling results suggest that the use of shallow pumping wells (only screened in the upper third of the Rose Creek aquifer) significantly reduces system performance. In this case, groundwater by-pass increases from 0.26 L/s (0.4%) to about 3.8 L/s (5.9%).

Model simulations suggest that partial screening in the middle or lower third of the Rose Creek aquifer sediments would only marginally reduce system performance (Table 5.1). It should be noted, however, that the groundwater flow model assumes no vertical anisotropy (Kh=Kv). While no vertical anisotropy was required to calibrate the model, drilling information suggests that some degree of vertical anisotropy might be present. If the silt layers encountered during drilling (e.g. at PW-1) are laterally extensive than partial penetration can be expected to reduce system performance significantly more than predicted with the current model.

In our opinion, further work will be required to better understand the influence of partial penetration on system performance (see section 5.5).

5.3.4 Scenario 4: Ten Pumping Wells plus Cut-off Wall

Scenario 4 assumes the same number and locations of pumping wells as for Scenario 2. However, a cut-off wall is added to the collection system about 100m downstream of the fence of wells (Figure 5.3). Scenario 4a assumes a partial cut-off wall in model layer 2 only (to a depth of ~1010 m AMSL). Scenario 4b assumes a deep cut-off wall keyed into bedrock (a nominal depth of 30-40 m below ground surface).

The use of a shallow cut-off wall (Scenario 4a) has virtually no benefit for system performance. Assuming a 2m drawdown in the high K zone (5m in the low K zones) groundwater by-pass remains at 18.2% compared to 18.6% without a cut-off wall (Table 5.1). In other words, a partially penetrating barrier does not improve capture efficiency as the groundwater is simply flowing underneath the barrier. However, as stated earlier, the groundwater flow model does not explicitly include the presence of low-permeability silt layers. If the silt layers encountered during drilling (e.g. at PW-1) are laterally extensive than a partial cut-off wall (keyed into those silt layers) could have a greater effect on capture efficiency (at least locally) than predicted by the model. Again, further work would be required to better understand the influence of a partially penetrating cut-off wall on system performance (see section 5.4).

Figure 5.14 shows the simulated groundwater flow field and path lines for the scenario of a fully penetrating cut-off wall (Scenario 4b). The model predicts that a fully penetrating barrier significantly improves system performance. The primary benefit of installation of a cut-off wall is that a significantly lower drawdown is required in the pumping wells to achieve the same degree of capture. For example, an assumed drawdown in the high K wells of only 2m (Scenario 4b) collects a total of 63.5 L/s and allows a by-pass of only 0.3 L/s (or 0.5%). In the comparative scenario 2c without a cut-off wall, the by-pass flow would be about 11.6 L/s (or 18.6%). Note also, that the fully penetrating cut-off wall reduces the total amount of groundwater required for successful capture by about 12% (from 72 L/s (Scenario 2e) to 63.5 L/s, Table 5.1). The reduced pumping requirement is a direct result of hydraulic isolation of the downstream portion of the aquifer from the interception system.

5.3.5 Scenario 5: Ten Pumping Wells – RCDC Fully Lined

Scenario 5 assumes that the Rose Creek Diversion Canal (RCDC) is fully lined, such that no further leakage occurs along the reach of interest. The pumping configuration for this scenario is identical to that described in Scenario 2 (10 fully penetrating pumping wells).

Figure 5.15 shows the simulated groundwater levels and path lines for Scenario 5 prior to any groundwater interception (pumping). The lack of leakage from the RCDC changes the groundwater flow field in the area downgradient of the Cross Valley Dam significantly (c. Figures 5.15 and 4.12). In the absence of any leakage, groundwater flows parallel to the main axis of the Rose Creek valley (in a northwesterly direction). Note that in this scenario, potentially contaminated groundwater from the south side of the Rose Creek aquifer would not be diverted to the north side (and intercepted by the drain system) but instead continues downstream impacting the entire width of the aquifer (Figure 5.15).

Table 5.1 summarizes the modeling results for groundwater interception with the RCDC removed. For this scenario, two different drawdown scenarios were simulated: (i) assuming 2m drawdown in the high K zone and 5m in low K zone, and (ii) assuming 5m drawdown in both high K and low K zones. Figure 5.16 shows the simulated groundwater flow field and path lines for the scenario with 5m drawdown in all pumping wells (Scenario 5b). In general, the lining of the RCDC reduces the amount of groundwater collected (for a given drawdown) but does not change the system performance (i.e. total or relative by-pass flow). For example, the total volume of groundwater collected in the 5m drawdown case (Scenario 5b) is 67.2 L/s compared to 77.7 L/s for Scenario 2f (a reduction of almost 10 L/s), yet the system performance in both scenarios is nearly identical (~0.3 L/s by-pass flow).

5.3.6 Scenario 6: Ten Pumping Wells – CVD and Polishing Pond Removed

Scenario 6 assumes that the Cross Valley Dam and the Polishing Pond are removed. The pumping configuration for this scenario is identical to that described in Scenario 2 (10 fully penetrating pumping wells).

Figure 5.17 shows the simulated groundwater levels and path lines for Scenarios 6 prior to any groundwater interception (pumping). As described in the methods section, removal of the CVD and dewatering of the Polishing would result in significant flooding in the former foot print area of the Polishing Pond. In practice, this toe seepage from the Intermediate Dam would be collected and pumped to a treatment plant for treatment. The model predicts that the seepage at the toe of the Intermediate Dam would be about 31 L/s. Note that this simulation assumed continued presence of the Intermediate Pond (seepage rate ~ 11 L/s). Depending on the closure scenario implemented for the Intermediate Pond and Intermediate tailings the rate of toe seepage could be substantially lower.

The removal of the Polishing Pond results in a significant reduction in the local hydraulic gradient, in particular in the vicinity of the CVD (c. Figures 5.17 and 4.12). Groundwater levels at the toe of the CVD declined by about 1.4m in the centre of the valley and by as much as 2.5m along the sides of the valley. The reduced hydraulic gradient results in a commensurate decrease in the rate of groundwater flow moving in the aquifer beneath the (now removed) CVD. The model predicts a steady-state groundwater flow at this location of about 13.9 L/s. This represents a reduction in groundwater flow to be collected at this location (by pumping wells) by a factor of four. Note, however, that an additional 31 L/s is predicted to discharge into the new drain system to be installed at the toe of the Intermediate Dam. Hence, the overall reduction in groundwater flow (requiring collection and treatment) are predicted to be reduced by only about 25% as a result of removal of the CVD and Polishing Pond.

Table 5.1 summarizes the modeling results for groundwater interception with the CVD and Polishing Pond removed. For this scenario, two different drawdown scenarios were simulated: (i) assuming 1m drawdown in the high K zone and 2m in low K zone, and (ii) assuming 3m drawdown in both high K and low K zones. Figure 5.18 shows the simulated groundwater flow field and path lines for the scenario with 3m drawdown in all pumping wells (Scenario 6b). It should be noted that the drawdown is calculated relative to the respective steady-state solution. Since groundwater levels along the toe of the CVD are reduced already by 1.4-2.5m (due to removal of the Polishing Pond, this additional drawdown should be added to the specified drawdown listed in Table 5.1 for comparison of absolute (geodetic) pumping levels. As a result, Scenario 6a has a similar absolute drawdown in the ten

pumping wells as Scenario 2d, and Scenario 6b is most closely compares with Scenario 2f (see Appendix H for actual geodetic heads assumed at each pumping well).

As expected, the removal of the Polishing Pond reduces the pumping requirements in the groundwater interception system significantly. For example, the volume of groundwater required for successful operation of the groundwater interception system (Scenario 6b) is only 30.9 L/s compared to about 72 L/s with the Polishing Pond present (Scenarios 2e). However, an additional 22.5 L/s will have to be collected as toe seepage for a combined total of about 53.4 L/s. Hence, the total reduction in contaminated water to be treated is about 18.6 L/s or about 25%.

5.4 Implications for Design of Groundwater Interception System

The simulation of groundwater interception scenarios with the current groundwater flow model (Version 2006.1) suggest that collection of potentially contaminated groundwater at the toe of the Cross Valley Dam is technically feasible with a high to very high capture efficiency.

5.4.1 Comparison of Groundwater Interception Scenarios

The three primary criteria for selecting a preferred design of a groundwater interception system include:

- Total amount of by-pass (L/s);
- Total pumping rate (L/s);
- Drawdown at pumping wells (m).

The total amount of by-pass is the primary criterion for system performance. For the purpose of this study an interception option was considered “fatally flawed” if the total amount of by-pass was greater than 0.5 L/s (representing about 1% or more of the total flow beneath the CVD). Hence, only those scenarios meeting this criterion are discussed further in this section.

In general, the other two performance criteria, i.e. the total amount of groundwater pumped (and treated) and the drawdown in the pumping wells should be minimized while still providing the required capture efficiency. The pumping rate should be minimized to reduce initial capital costs (well diameter, pump size etc) and the long-term costs of pumping and treatment. The drawdown in the pumping wells should be minimized as much as possible to improve well efficiencies and reduce the potential for desaturation of the well screen[6].

Table 5.2 summarizes the predicted pumping rates (individual and total) for selected scenarios which met our criterion of <0.5 L/s (or <1%) by-pass. The predicted total pumping rate for the case of 5 pumping wells is essentially equivalent to the scenario of 10 pumping wells. However, the individual pumping rates, in particular for the “high K” wells, are significantly higher for scenario 1 than for scenario 2 (almost double). There is some concern, that the required individual pumping rates for

[6] It should be noted that the drawdown in the pumping wells will be somewhat higher than simulated here due to (i) scaling effects (the minimum cell size used for a pumping well (5m by 5m) is still significantly larger than the effective well radius and (ii) potential well losses during pumping. Field experience and scoping calculations suggest that the actual drawdown in the pumping wells may be greater by a factor of 1.3-1.5 in the high K zones and 1.5-2.0 in the lower K zones.

scenario 1 (5 wells) are not achievable for long-term pumping. For example, the required pumping rate at PW-1 would have to be about 26.6 L/s (422 USGPM). Pumping tests carried out PW-1 suggested that this well can provide short-term yields as high as 31.5 L/s (500 USGPM). However, this pumping rate was not sustainable and progressive dewatering of the upper well screen occurred, resulting in greater drawdown and ultimately pump failure (believed to be due to caving of formation material in the desaturated upper screen) (see section 3.1). Based on this experience, it is questionable whether PW-1 could be pumped continuously at the required pumping rate of 26.6 L/s.

Table 5.2

Predicted pumping rates for selected groundwater interception scenarios.

Pumping Well	Layers Pumped	Predicted Pumping Rates							
		Scenario 1f 5 Pumping Wells		Scenario 2e 10 Pumping Wells		Scenario 4b 10 Pumping Wells plus Cut-Off Wall		Scenario 6b 10 Pumping Wells (w/o Polishing Pond)	
		L/s	USGPM	L/s	USGPM	L/s	USGPM	L/s	USGPM
PWA	2, 3, 4	1.25	20	0.27	4	0.42	7	0.05	1
PWB	2, 3, 4	0.80	13	1.27	20	1.91	30	0.28	5
PWC	2, 3, 4	0.65	10	2.27	36	3.72	59	0.55	9
PW1	2, 4	26.6	422	15.59	247	12.55	199	6.81	108
PWD	2, 3, 4	n/a	n/a	21.31	338	16.88	268	9.68	153
PW2	2, 3	43.6	691	21.69	344	17.79	282	9.30	147
PWE	2, 3, 4	n/a	n/a	7.71	122	5.23	83	3.57	57
PWF	2, 3, 4	n/a	n/a	0.72	11	1.19	19	0.16	3
PWG	2, 3, 4	n/a	n/a	0.60	10	0.86	14	0.23	4
PWH	2, 3, 4	n/a	n/a	0.53	8	0.69	11	0.28	4
TOTAL Capture		**72.9**	**1155**	**72.0**	**1141**	**61.2**	**971**	**30.9**	**490**

Clearly, individual pumping rates are significantly lower when a greater number of pumping wells is utilized. For example, in Scenario 2 (10 wells), PW-1 would only have to be pumped at a rate of about 15.6 L/s (250 USGPM). The 4-day step test in PW-1 carried out as part of the 2005 pumping test program suggests that this pumping rate is sustainable in this well (see section 3).

Another benefit of using a greater number of pumping wells is the reduction in individual drawdown required at each pumping well. Model simulations suggest that a system with 5 pumping wells would require a drawdown of ~5 m whereas 10 pumping wells would reduce this requirement to ~4m drawdown. This difference may not be significant if there is sufficient available drawdown in a given pumping wells (e.g. at PW-2). However, this is not the case across the entire width of the aquifer. At PW-1, for example, the highly permeable, coarse sediments are separated by a thick layer of low permeability silts. The available drawdown in the upper aquifer is only about 5.2m. In other words, a drawdown of 5 m would dewater almost the entire upper aquifer. Note that significant silt ingress was observed during the 56-hr pumping test at PW-1 (causing damage to the pump) after the upper well screen had been completely desaturated.

One way to circumvent the issue of dewatering a well screen would be to screen only the deeper portion of the aquifer. This way, the well screen would remain saturated at all times and silting (due to caving) and potential well fouling (due to aeration of the well screen) would not be a factor. Preliminary model predictions with the current model suggest that partial penetration (at depth) would

not negatively impact system performance (see Scenarios 3b and 3c). However, additional work would be required to confirm this preliminary assessment (see section 5.5 below).

In summary, the use of 10 pumping wells is preferred over the use of 5 pumping wells because it reduces the individual pumping rates and requires less drawdown at the individual pumping wells. Nevertheless, both scenarios may require partially penetrating pumping wells (screened in the lower portion of the aquifer) at least at some locations, to prevent desaturation of the well screen.

Figure 5.19 shows the simulated hydraulic heads for the scenario with 10 wells (Scenario 2e) along a cross-section along the centre of the valley (halfway between pumping wells PW1 and PW-D). This figure illustrates the asymmetrical groundwater flow field with steep gradients on the upstream side and very low hydraulic gradients on the downstream side. Note that the “hinge line”, i.e. the line up to which hydraulic gradients are reversed due to pumping, extends as much as ~450m downstream of the fence of wells in the permeable sediments and still ~ 300m in the deeper bedrock. In other words, the model predicts that the assumed drawdown (4m in the high K zone) is adequate to reverse gradients in all layers of the model

Scenario 4b represents the most effective groundwater interception system for current conditions because the use of a (fully penetrating) cut-off wall minimizes both the pumping rates and the required drawdown. The total (and individual) pumping rates are reduced by about 15-20% (Table 5.2) because the cut-off wall essentially eliminates the inflow of (uncontaminated) groundwater from further downstream. In addition, the required drawdown in the high K pumping wells is only about 2m. In other words, there would be little, if any, issue with well performance related to excessive drawdown and/or desaturation of the well screen.

The primary disadvantage of the installation of a cut-off wall is cost which would be significant at the required length (~500 m) and depths (up to 45m).

The available information suggests that a groundwater interception system with 10 pumping wells (e.g. Scenario 2e) is adequate for successful capture of the contaminated groundwater. However, the current analysis does not account for all the heterogeneity observed in the field which could reduce system performance (see section 5.4.3). In our opinion, the installation of a cut-off should therefore be included in the design of a groundwater interception system as a contingency measure.

5.4.2 Assessment of Alternative Closure Scenarios

The model suggests that leakage from the RCDC currently dilutes groundwater in the southern portion of the Rose Creek aquifer and forces contaminated groundwater to the northern side of the Rose Creek aquifer downstream of the Cross Valley Dam (Figure 5.15). These modeling results are consistent with available groundwater quality data (see section 2.2.3). If this hypothesis is correct then groundwater interception in the southern portion of the aquifer may not be required, at least as long as the contaminant concentrations in the groundwater are sufficiently diluted by leakage from the RCDC.

While leakage from the RCDC is beneficial in a sense that it dilutes the contaminated groundwater in the centre of the Rose Creek valley, this source of water also increases the total volume of groundwater to be pumped. The model predicted that lining of the RCDC (Scenario 5a/b) would reduce the required total groundwater capture rates by about 10 L/s (or 14%) (Table 5.1). There would be no other significant benefits to the performance of the groundwater interception system.

The potential savings realized by reduced pumping rates (cost of pumping and treatment) would have to be compared to the costs incurred to fully line the existing RCDC.

The removal of the Polishing Pond (Scenario 6b) would provide significant advantages for the groundwater collection system, including:

(i) a large reduction (about 43%) in groundwater to be extracted via pumping wells (Table 5.2);

(ii) a moderate reduction (about 25%) in total water to be treated (toe seepage and groundwater combined) (Table 5.1).

The significant reduction in pumping rates would result in potential savings during system construction (smaller pumping well diameter and pump sizes) and during long-term operation (power & maintenance costs). Despite these large reductions in required pumping rates, the model predicts very little benefit in terms of the required drawdown (in absolute terms). In other words, removal of the Polishing Pond does not eliminate the potential issues with limited available drawdown in the upper portion of the aquifer (see above).

The primary disadvantage of removing the Polishing Pond would be the requirement for earth works in the foot print area of the pond (to remove sediments and place fill material in borrow pit areas below the future groundwater table) and for construction/maintenance of a drain collection system at the toe of the Intermediate Dam. The potential benefits of reduced pumping rates due to pond removal would have to be weighted against the costs of collecting contaminated seepage at the toe of the Intermediate Dam.

Note that additional pumping wells could be placed at the toe of the Intermediate Dam to reduce (or eliminate) any toe seepage should this be desired (e.g. to eliminate glaciation problems in a toe drain). A simulation of this scenario was beyond the scope of this study.

5.4.3 Model Limitations and Implications for Design

As discussed in section 4.5, the calibrated groundwater flow model is a simplified representation of highly complex subsurface conditions observed in the Rose Creek valley. These model limitations result in uncertainty of model predictions which should be kept in mind when interpreting the predicted performance of a groundwater interception system for the Rose Creek aquifer.

The key uncertainties in the current version of the model are as follows:

- Uncertainty in hydrostratigraphy;
- Heterogeneity & vertical anisotropy;
- Recharge and Underflow;
- Seepage from Polishing Pond/Intermediate Pond

In the following we briefly highlight these uncertainties and discuss implications for the design of a groundwater interception system.

5.4.3.1 Uncertainty in Hydrostratigraphy

The geology and resulting hydraulic properties along the sides of the model domain are not very well defined. The model predicts much lower pumping requirements for the lower K zones ("colluvium" or "slope till") inferred for these peripheral areas than for the high K sediments in the center of the valley. While the spatial extent of these zones is based on model calibration, this process is not unique and there is significant remaining uncertainty as to the spatial extent and hydraulic properties of those "low K" zones. Note that some drill logs (e.g. at P05-01) suggest significant interfingering of low K and high K material making it difficult to predict the "effective" hydraulic properties of these sediments from drilling alone. Additional field work (drilling and hydraulic testing) will be required to better define the hydrogeological conditions along the sides of the Rose Creek valley near the CVD (see section 5.5 for details). The results of this field work could significantly influence the design of a groundwater interception system (# of wells, screening interval, pumping rates). This field work should therefore be completed prior to final design of a groundwater interception system.

Note also that the model did not take into account the potential presence of a (likely thin) layer of more permeable weathered and/or fractured bedrock above the intact bedrock. Little is known about the potential presence of such a layer and its hydraulic properties. The calibrated model matched the observed drawdown in shallow bedrock fairly well (Figures 4.10 and 4.11). However, monitoring data in bedrock for the two pumping tests were very limited (only P05-01 is screened in bedrock) at the CVD and the response in P05-01 may not be representative of all bedrock in this area. We recommend that the current model be updated to assess the importance of a weathered bedrock layer on groundwater capture efficiency (see section 5.5). This model should then be used to study the influence of a weathered/fractured bedrock layer on the performance of a groundwater interception system (by way of sensitivity analysis). If it can be shown that by-pass may occur in such a (potentially permeable) zone then the pumping wells may have to be keyed into the upper bedrock (say 5 – 10m) to intercept any preferred groundwater flow at or beneath the sediment-bedrock interface.

5.4.3.2 Heterogeneity and Vertical Anisotropy

The 2005 drilling results demonstrated that the glaciofluvial sediments in the Rose Creek valley exhibit significant heterogeneity, both within a given borehole and between neighbouring boreholes. An explicit representation of this heterogeneity was beyond the scope of this study and may not be feasible considering the degree of heterogeneity and the limited number of drill holes. However, this small-scale heterogeneity may result in anisotropy at the model scale.

At present, it is unclear whether the observed heterogeneity is random (and can be ignored at the larger scale) or whether this heterogeneity does indeed result in large-scale anisotropy. The depositional environment (glacio-fluvial sediments) would suggest that the observed low permeability silt lenses/layers are of limited lateral extent. However, the presence of these low K sediments may produce some degree of vertical anisotropy (Kh > Kv). While no vertical anisotropy was required to calibrate the model, the pumping tests were also not specifically designed to assess vertical heterogeneity. For example, PW-1 was screened in permeable sediments above and below a thick package of mixed sediments (including low K silt layers). In other words the test stressed both the upper and the lower portion of the aquifer providing limited insight on the degree of vertical anisotropy, potentially caused by the lower K sediments in the middle portion of the aquifer.

The primary influence of vertical anisotropy on the performance of a groundwater interception system would be to reduce capture efficiency (increase the potential for by-pass) provided partially penetrating pumping wells and/or cut-off walls would be used. In most of our scenarios we assumed fully penetrating pumping wells. In practice, however, fully penetrating pumping wells may not be always achievable considering the small-scale heterogeneity (see for example installation of PW-1). Furthermore, field experience and modeling results suggest that pumping wells screened only in the deeper portion of the aquifer may perform better as they would avoid problems related to desaturation of the uppermost well screen(s). In other words, partially penetrating wells may be required for the interception system whether by design or necessity.

The current model suggested that pumping wells screened in the deeper portion of the aquifer would perform almost as well as those fully penetrating the Rose Creek sediments (Table 5.1). However, this model prediction does not take into account the presence of confining layers of lower K sediments (silt) and the resulting potential for vertical anisotropy. Considering the likelihood of having to use partially penetrating pumping wells, the influence of small-scale heterogeneity and vertical anisotropy on system performance will have to be investigated further prior to final design of a groundwater interception system. Recommendations for further field testing and modeling work to address this issue are provided in section 5.5.

5.4.3.3 Recharge and Underflow

The model assumed no recharge to the groundwater system in the Rose Creek Valley from precipitation and/or groundwater inflow from the valley sides. In practice some recharge from both sources will occur during year-round operation of a groundwater interception system. This simplifying assumption would tend to underestimate the actual pumping rates required. However, the magnitude of recharge from precipitation and side hill drainage is likely small compared to the total groundwater flow in the Rose Creek Valley.

Another uncertainty in the groundwater flow model is the amount of underflow of groundwater entering the model domain from the upstream boundary. The amount of underflow was estimated using a constant head boundary. The hydraulic head in this area was not known and therefore had to be estimated. The amount of underflow for current conditions seems reasonable as it provides a reasonable water balance for the system. In contrast, the amount of underflow for future conditions, in particular after removal of the Polishing Pond is less certain. For the purpose of this study, we assumed that the amount of underflow entering the upstream model domain would remain constant after removal of the Polishing Pond. In other words, the amount of groundwater inflow was assumed to be independent of the head conditions in the downstream portion of the aquifer. However, the rate of underflow may in fact increase with removal of the Polishing Pond if there is a large source of water available. This latter scenario would result in larger flows to the toe drain of the Intermediate Dam (to be collected and treated) than predicted here. However, it would not significantly change the predicted pumping rates from the pumping wells at the toe of the CVD.

5.4.3.4 Seepage from Polishing Pond/Intermediate Pond

As discussed in section 4.5, a detailed representation of the dam structures (Cross Valley Dam and Intermediate Dam) was beyond the scope of this study and there remains some uncertainty about the predicted seepage rates from the Polishing Pond and the Intermediate Pond. This uncertainty is

perhaps most critical for the assessment of Scenario 6, i.e. removal of the Polishing Pond. This scenario represents a significant change in the overall flow regime at the toe of the Cross Valley Dam, for which the model was calibrated. As a result the model predictions for Scenario 6 (CVD and PP removed) have a greater uncertainty.

In order to obtain greater confidence in model predictions for Scenario 6 (CVD and PP removed) additional calibration work would be required. This additional calibration work would be justified if removal of the CVD and the Polishing Pond was to be selected as the preferred closure alternative. Recommendations for additional work to better constrain the seepage from the Polishing Pond and the Intermediate Pond are provided in section 5.5.

5.5 Further Work

The following recommendations are provided to improve the current groundwater flow model for improved predictions such as might be needed in final design of a groundwater interception system at the toe of the Cross Valley Dam (in order of priority):

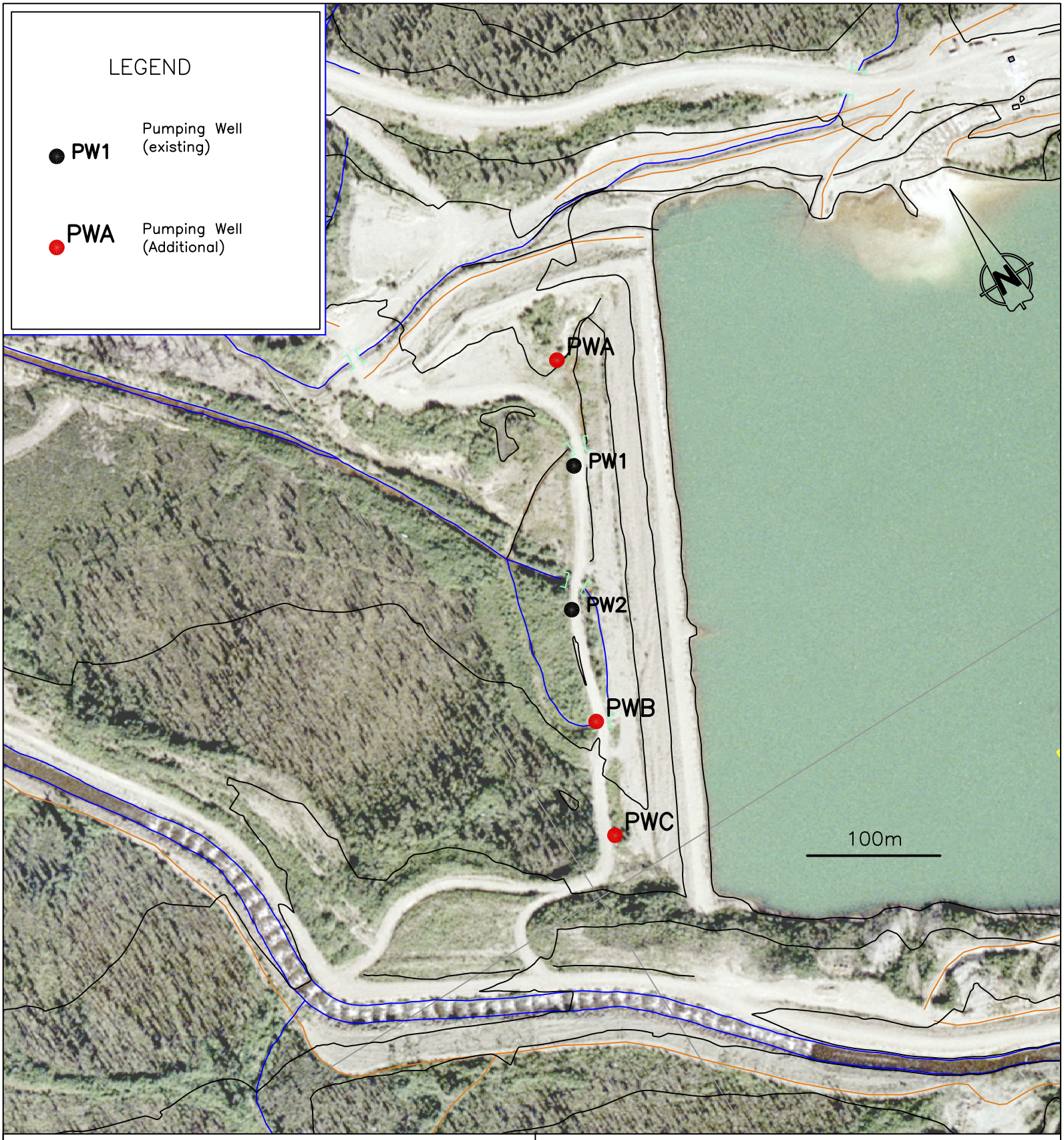
- Additional hydraulic testing and model calibration should be carried out to assess the influence of small-scale heterogeneity and vertical anisotropy on performance of a groundwater interception system. We recommend that a 7-day pumping test be conducted in the lower screen of PW-1 (the upper screen should be isolated using a packer system). The results of this pumping test should be used to re-calibrate the groundwater flow model. This calibration work would determine whether the vertical heterogeneity in the glacio-fluvial sediments can be simulated implicitly (assuming vertical anisotropy) or must be included explicitly to predict the performance of partially penetrating pumping wells on system performance. The results of this recommended work could significantly influence the design of a groundwater interception system (in particular the feasibility of using partially penetrating pumping wells). This recommended work should therefore be completed prior to final design of a groundwater interception system.
- Additional field work (drilling and hydraulic testing) should be carried out to better define the hydrogeological conditions along the sides of the Rose Creek valley near the CVD. We recommend that one pumping well and two monitoring wells each be installed on the south and north side of the proposed alignment of the fence of wells. One of the two monitoring wells on each side should be completed in the weathered and/or fractured bedrock. A 24-hr pumping test should then be carried out at both new pumping wells to determine the hydraulic properties (transmissivity, storage) in the local sediments and underlying weathered/fractured bedrock and the hydraulic connection to the central portion of the valley. The results of these two tests should be used to re-calibrate the groundwater flow model and, if required, update the design of the groundwater interception system. The results of this work could significantly influence the design of a groundwater interception system (# of wells, screening interval, pumping rates). This work should therefore be completed prior to final design of a groundwater interception system.
- Additional calibration work should be carried out to better constrain the seepage rate from the Polishing Pond and the Intermediate Pond. While this work is important, but not essential, for the current dam configuration (Scenarios 1-5), this work is considered a priority if removal of

the CVD and the Polishing Pond (Scenario 6) was the preferred option for closure of the Rose Creek Tailings Facility. The following activities are proposed for this work:

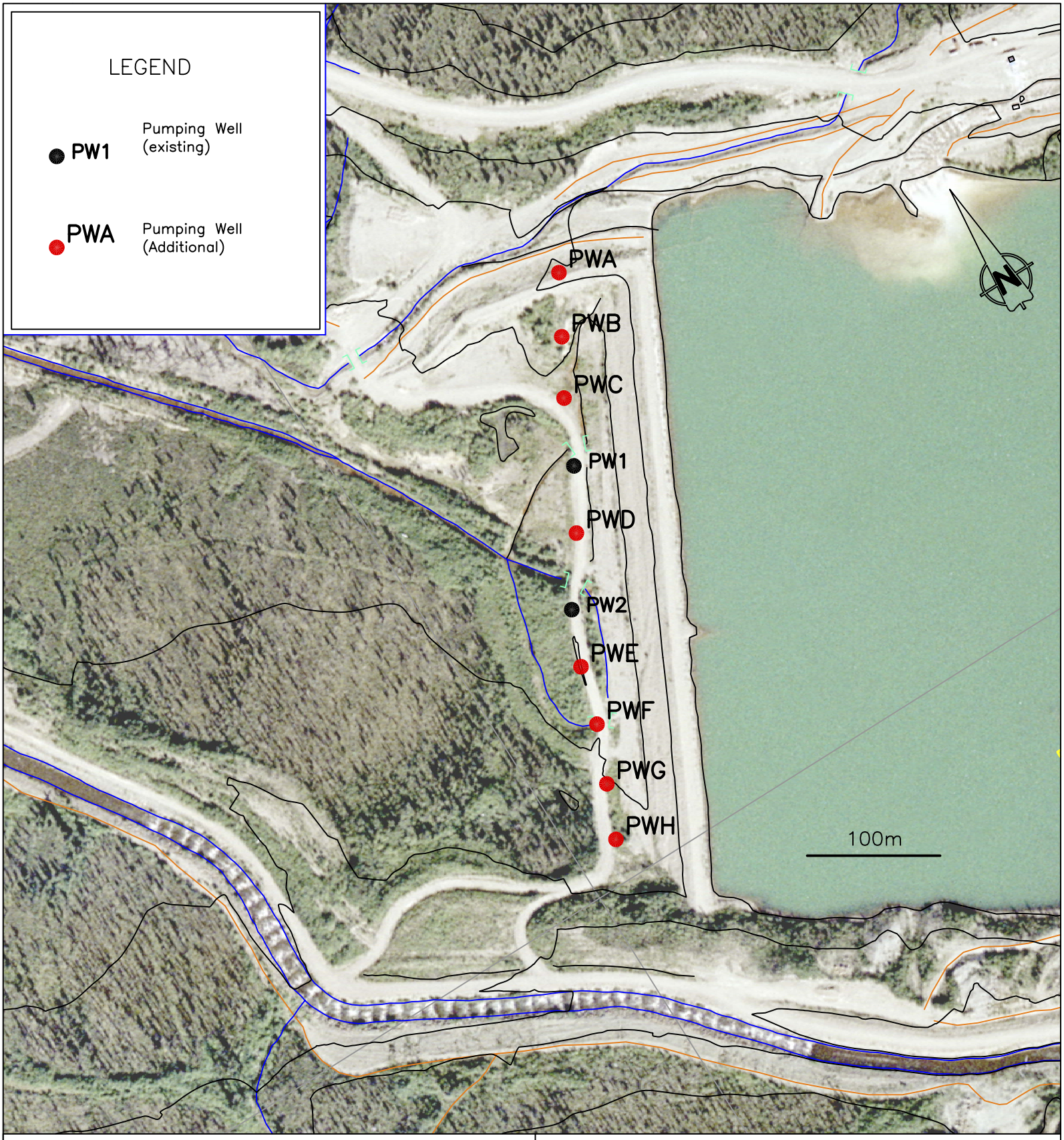
- Complete a water and load balance for the Polishing Pond and the Intermediate Pond for the inactive winter period (taking into account changes in the pond level and any inflows/outflows); for this purpose, depth-discrete sampling of water quality should be carried out in the Polishing Pond (if not already available) to determine the importance of vertical stratification,
- Monitor pond levels, groundwater levels and seepage flows at the toe of the CVD (monthly readings) for one winter period;
- Monitor pond levels, groundwater levels and seepage flows at the toe of the CVD (daily readings) during a period of active drawdown in the Polishing Pond (e.g. during active pumping/drawdown prior to winter shut-down);
- Survey the elevations of the toe drains and review as-built drawings of the CVD; incorporate these updated elevations and any important dam design elements and re-calibrate the groundwater flow model using the transient monitoring results observed for (i) the winter base flow period and (ii) the short-term pumping period.

It should be noted that, depending on the schedule of closure, some of the proposed monitoring work may not be required. For example, if there is adequate time between removal of the CVD/Polishing Pond and final design of a groundwater interception system, the response of the groundwater system to the draining of the PP could be monitored and these observations could be used to re-calibrate the groundwater flow model and finalize the design of a groundwater interception system.

- Sensitivity analyses should be carried out to study the influence of a layer of (potentially permeable) weathered and/or fractured bedrock on system performance. For this purpose, an additional layer of weathered/fractured bedrock (assuming a uniform thickness) should be introduced in the model. Selected groundwater interception scenarios would then be simulated assuming a plausible range of transmissivities for this weathered/fractured bedrock layer. If required, the initial design would be modified (e.g. pumping wells extending into shallow bedrock) to achieve the required capture efficiency. This modeling work should be carried out after (or in conjunction with) the assessment of vertical anisotropy and characterization/pumping tests on the north and south side (see above) as these field investigations provide additional information on the hydraulic properties of the shallow bedrock.
- Sensitivity analyses should be carried out to assess the influence of recharge from the valley sides and underflow from upstream on system performance, in particular the total pumping rates required to achieve the required capture efficiency.

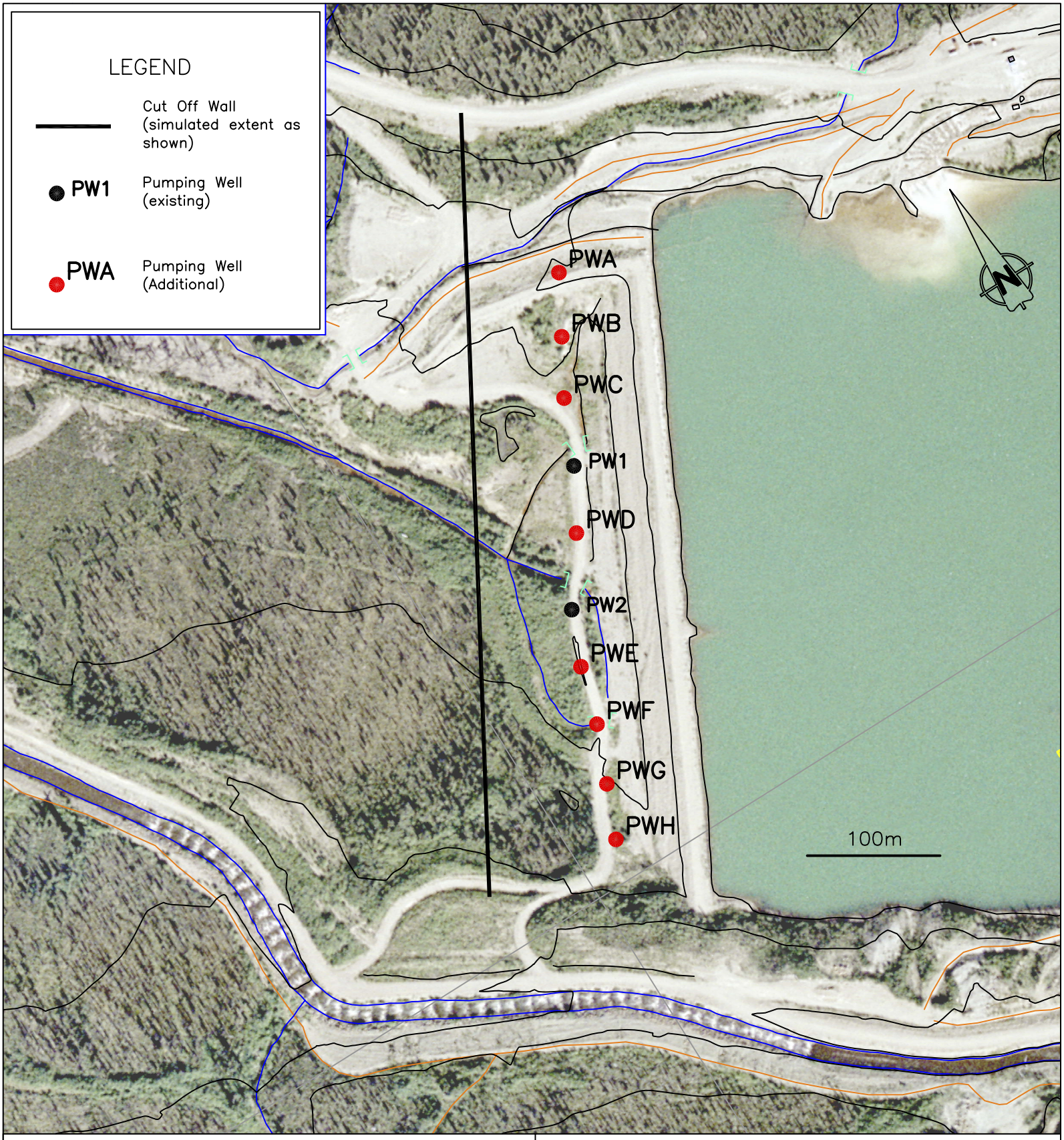


CLIENT: Deloitte & Touche Inc.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers
PROJECT: Rose Creek Groundwater Interception	Layout of Groundwater Interception Scenario 1
REPORT: 118004	
FILE: fig5-1 pump model.dwg	
LOCATION: Faro Mine, Yukon Territory, Canada	DATE: July, 2006 / DRAWN BY: MP / FIGURE: 5.1



CLIENT: Deloitte & Touche Inc.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers
PROJECT: Rose Creek Groundwater Interception	Layout of Groundwater Interception Scenarios 2 and 3
REPORT: 118004	
FILE: fig5-1 pump model.dwg	
LOCATION: Faro Mine, Yukon Territory, Canada	

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		5.2



CLIENT: Deloitte & Touche Inc.	ROBERTSON GEOCONSULTANTS INC. Consulting Geotechnical and Environmental Engineers
PROJECT: Rose Creek Groundwater Interception	Layout of Groundwater Interception Scenario 4
REPORT: 118004	
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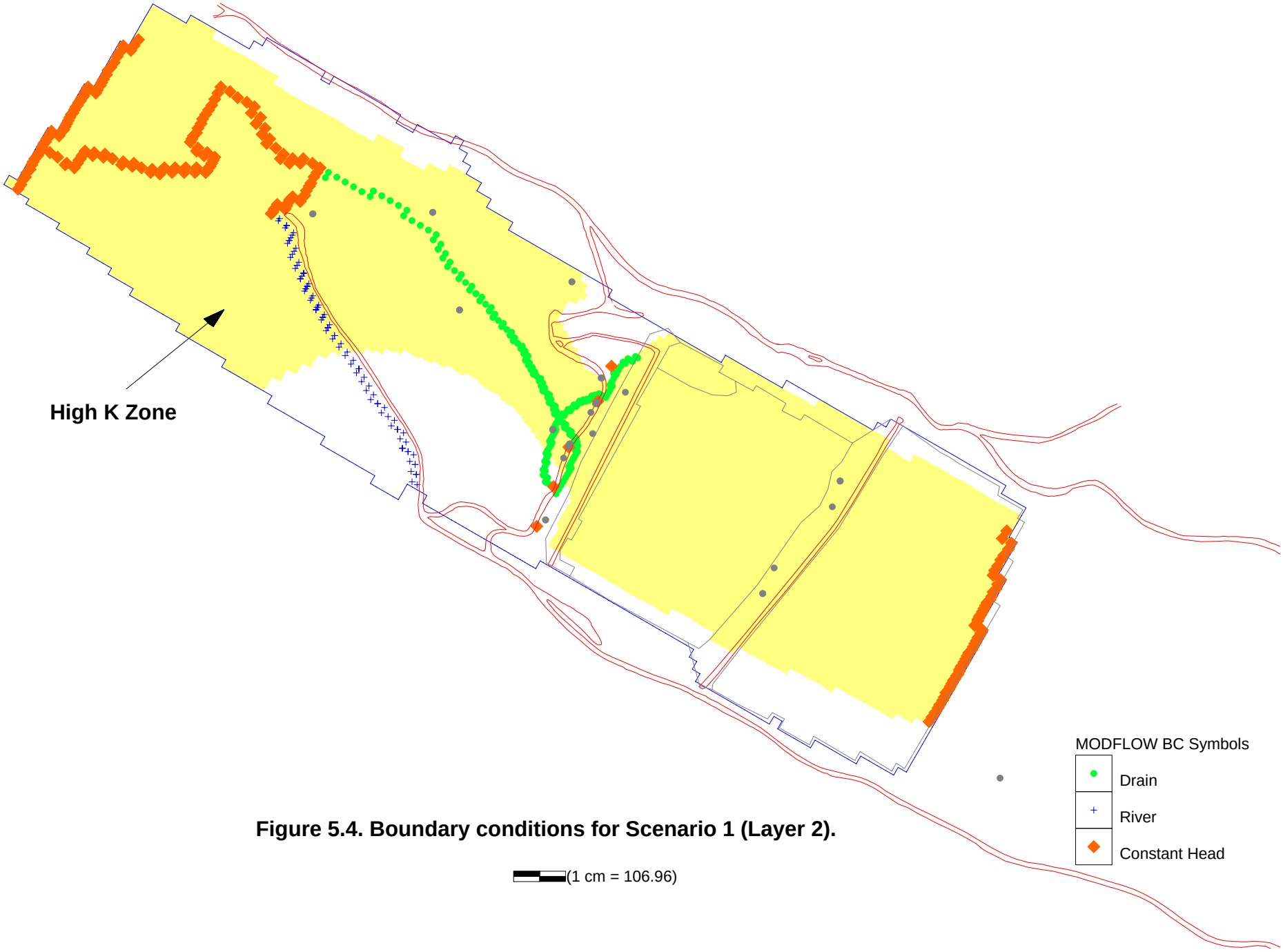


Figure 5.4. Boundary conditions for Scenario 1 (Layer 2).

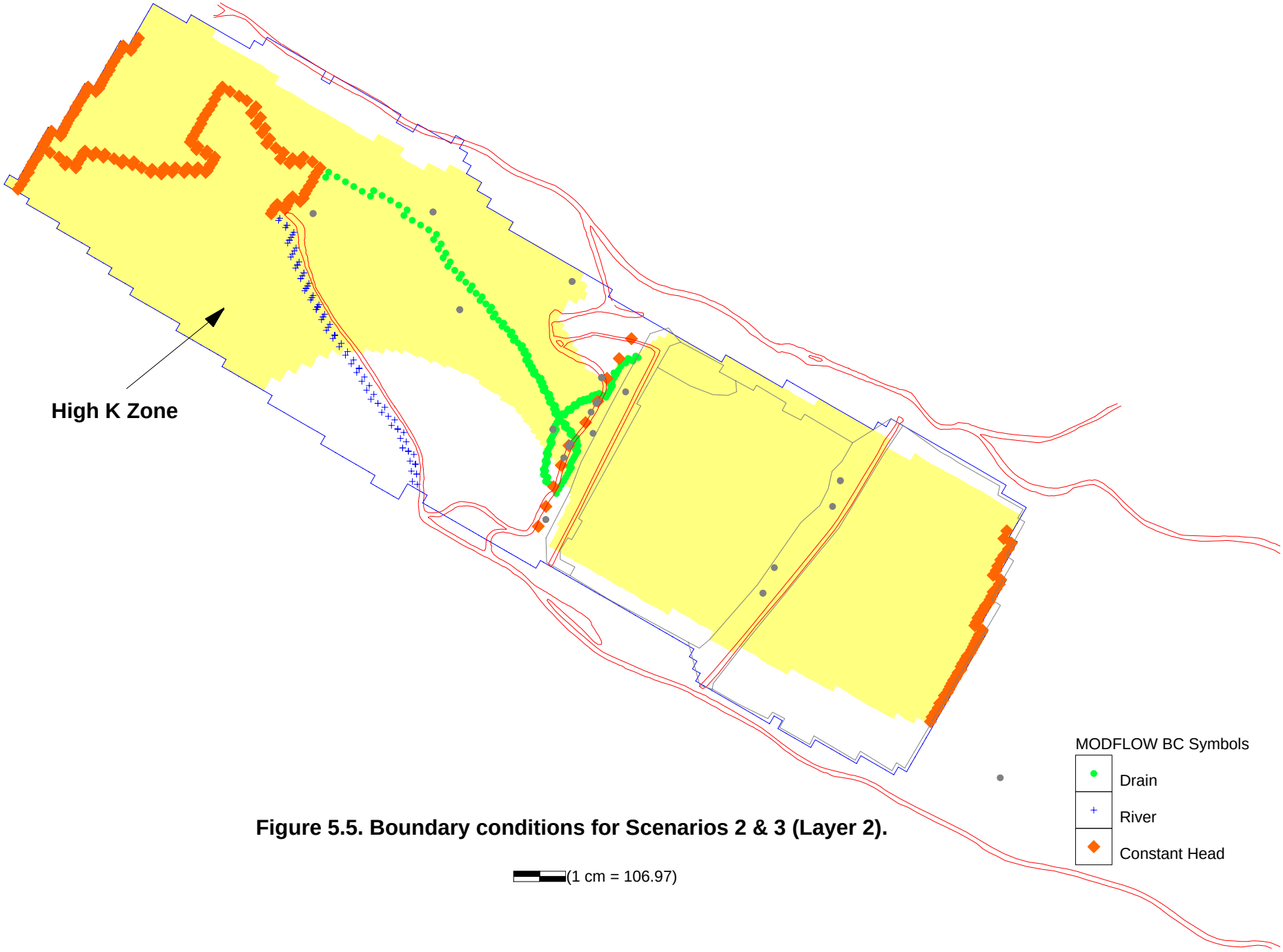


Figure 5.5. Boundary conditions for Scenarios 2 & 3 (Layer 2).

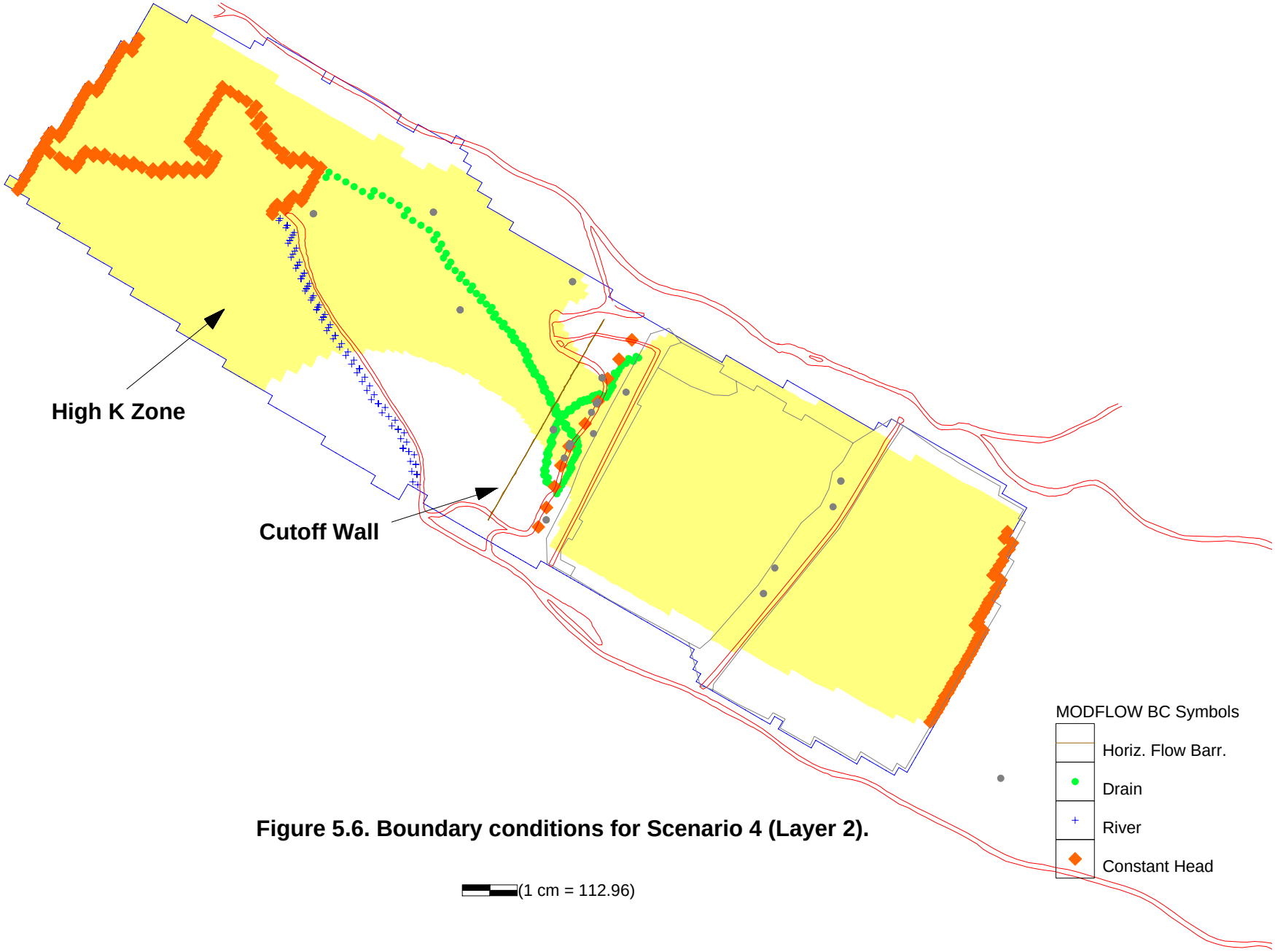


Figure 5.6. Boundary conditions for Scenario 4 (Layer 2).

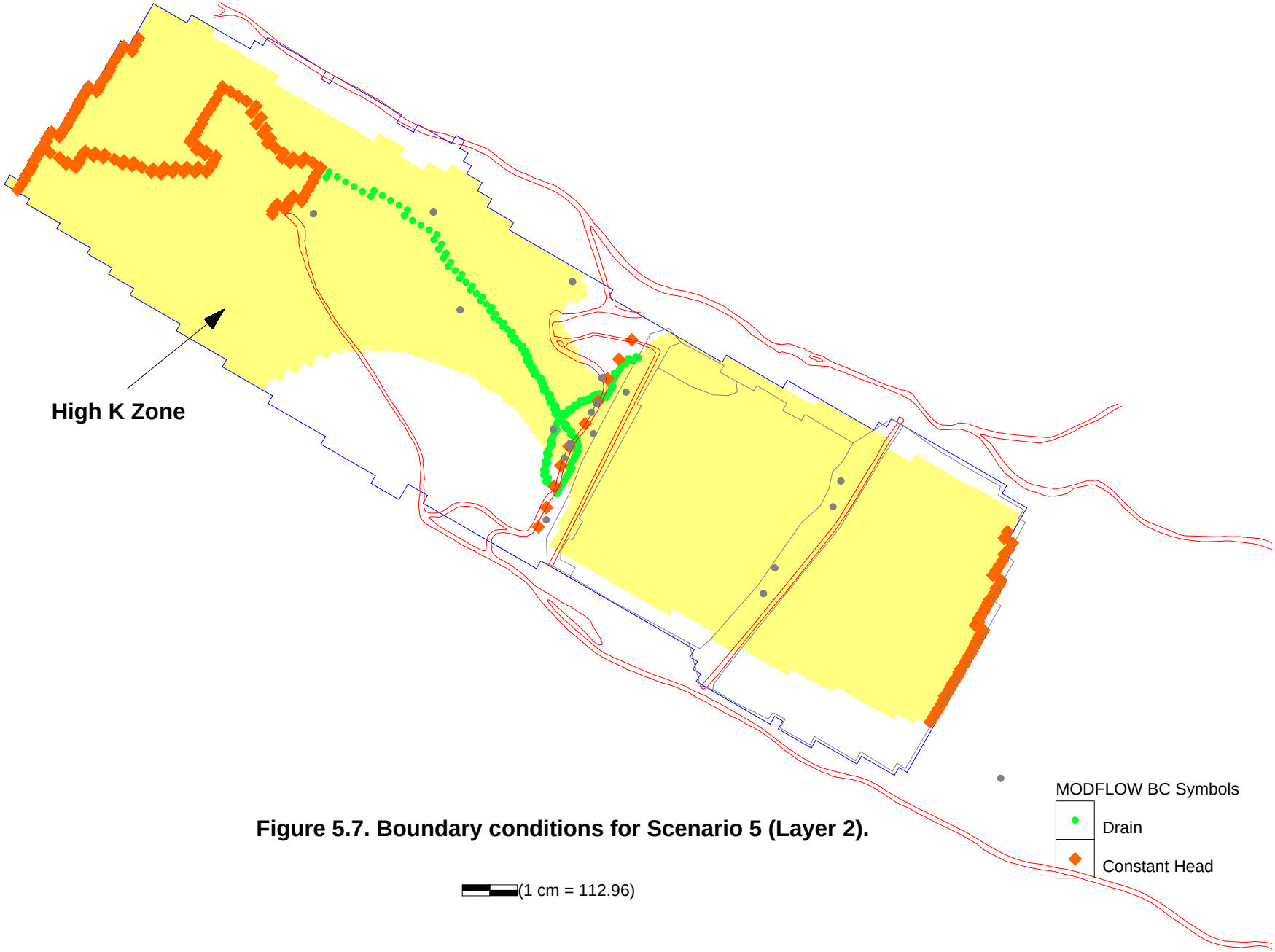


Figure 5.7. Boundary conditions for Scenario 5 (Layer 2).

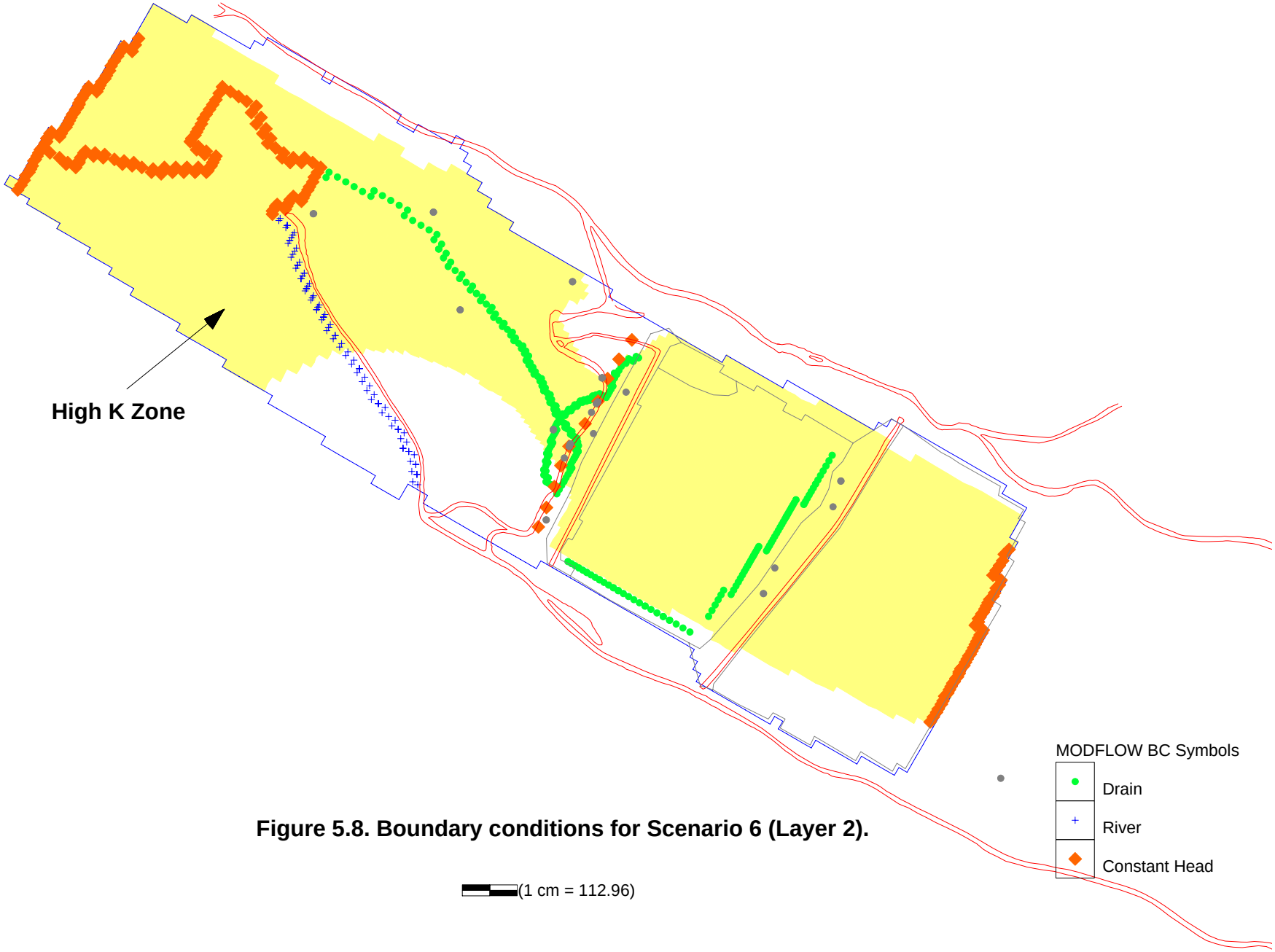


Figure 5.8. Boundary conditions for Scenario 6 (Layer 2).

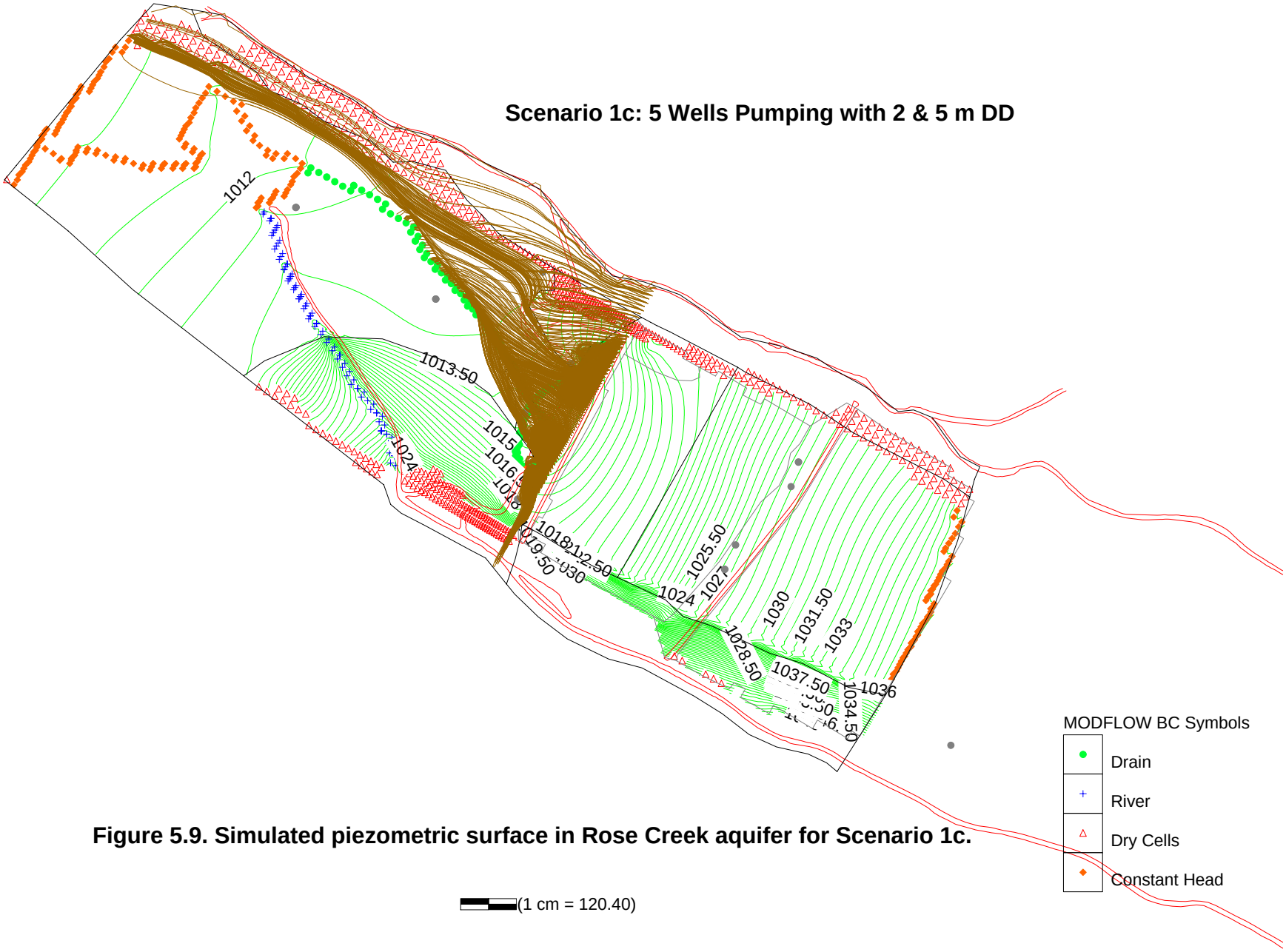


Figure 5.9. Simulated piezometric surface in Rose Creek aquifer for Scenario 1c.

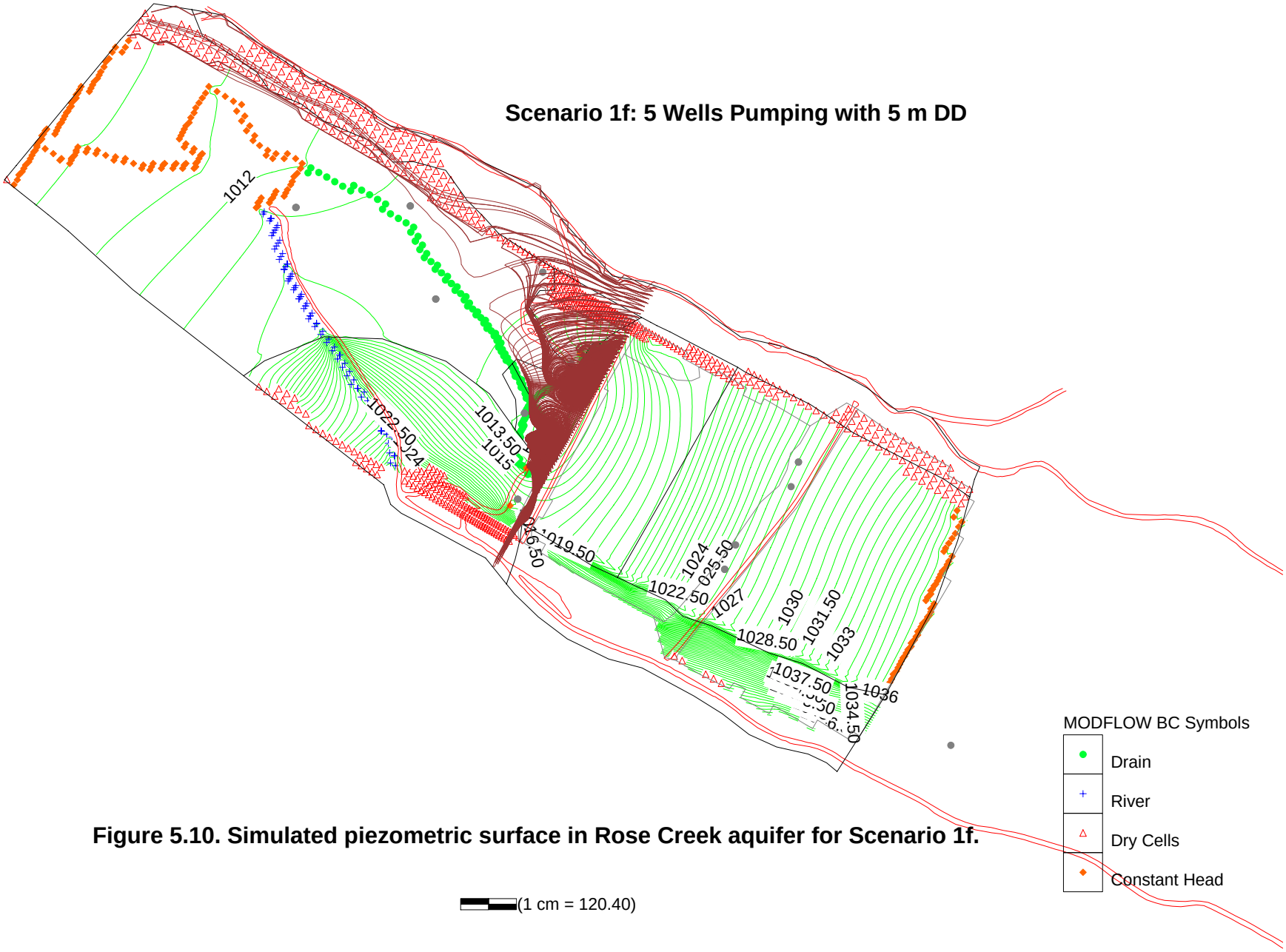


Figure 5.10. Simulated piezometric surface in Rose Creek aquifer for Scenario 1f.

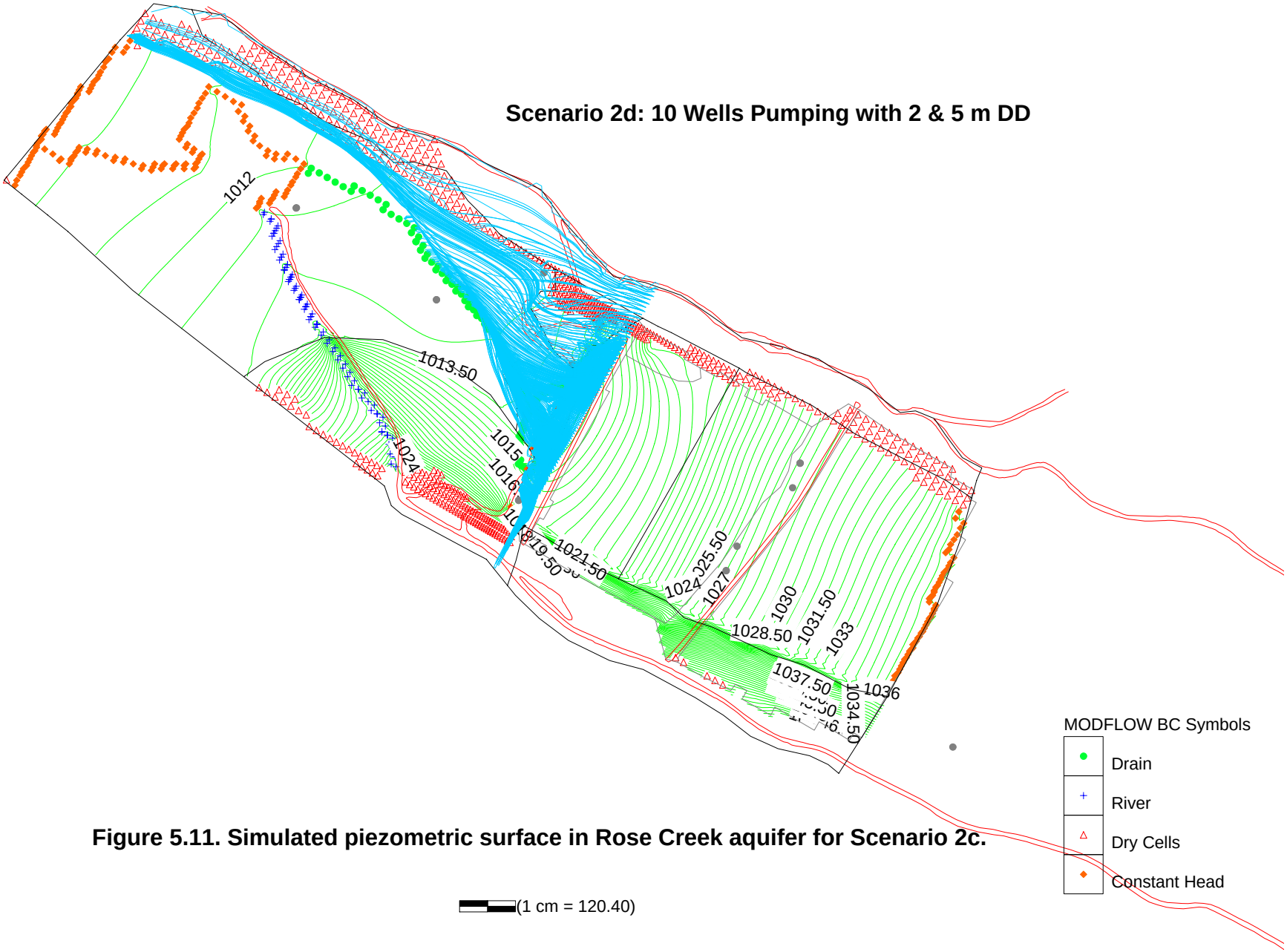


Figure 5.11. Simulated piezometric surface in Rose Creek aquifer for Scenario 2c.

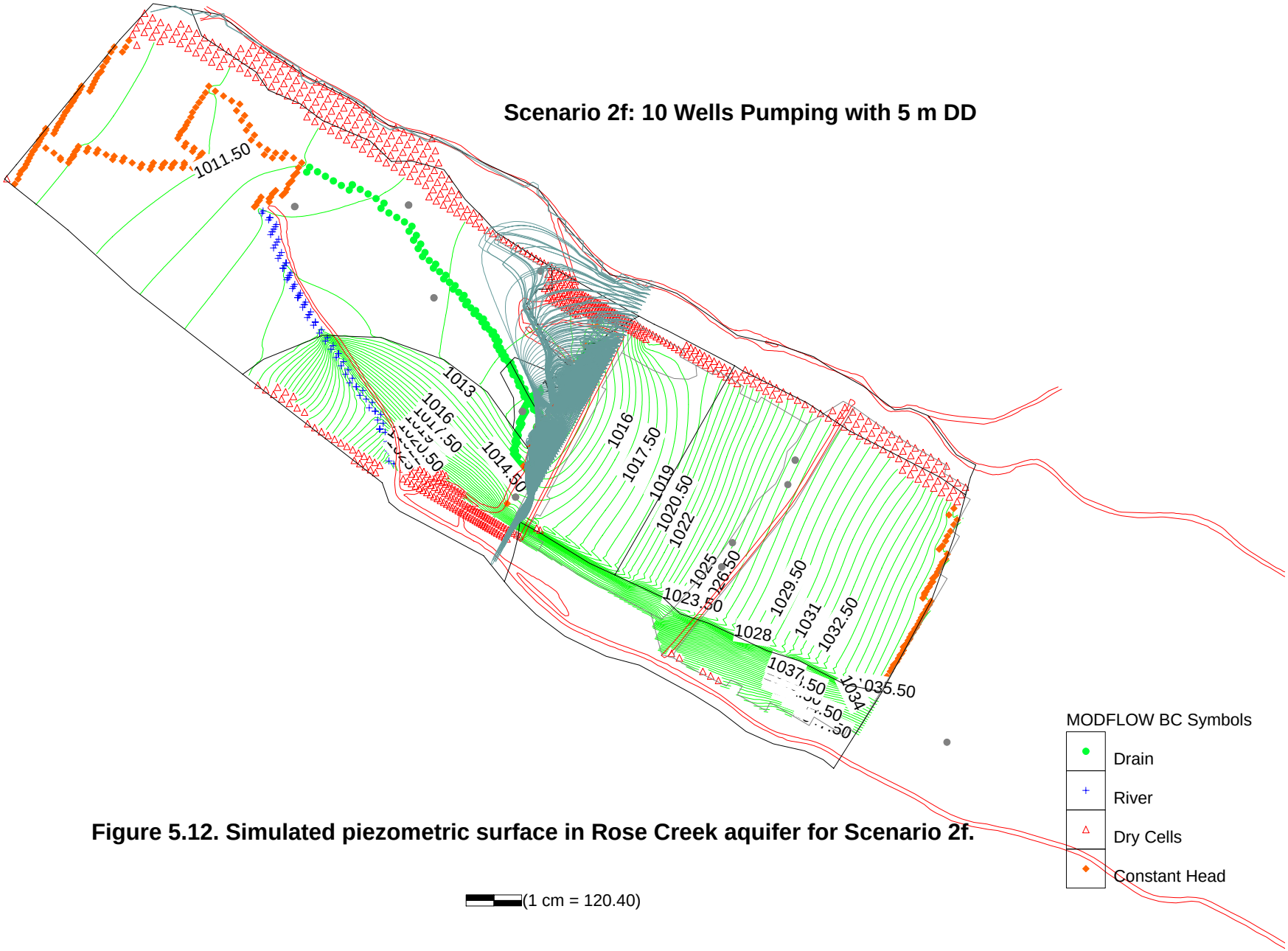


Figure 5.12. Simulated piezometric surface in Rose Creek aquifer for Scenario 2f.

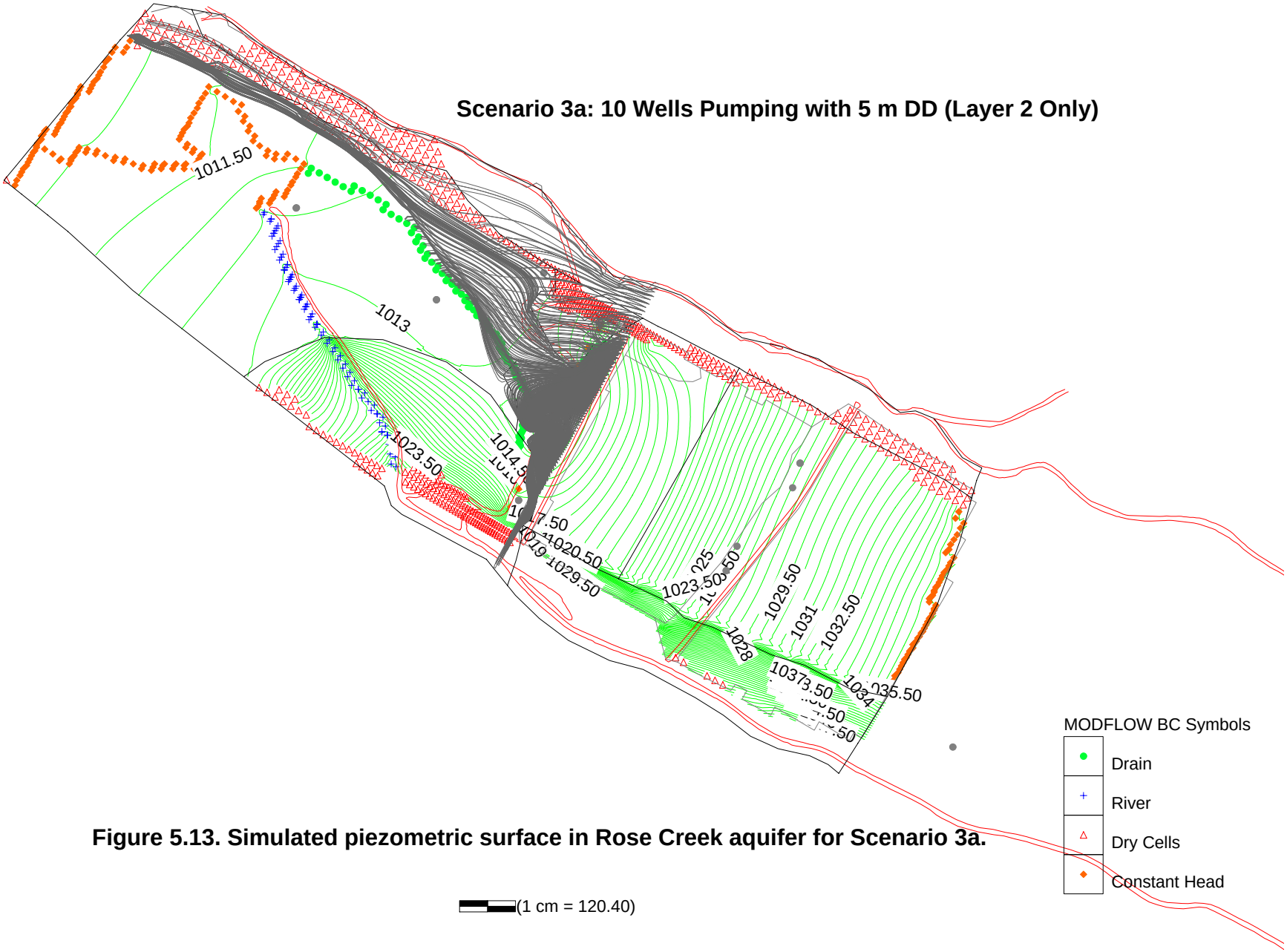


Figure 5.13. Simulated piezometric surface in Rose Creek aquifer for Scenario 3a.

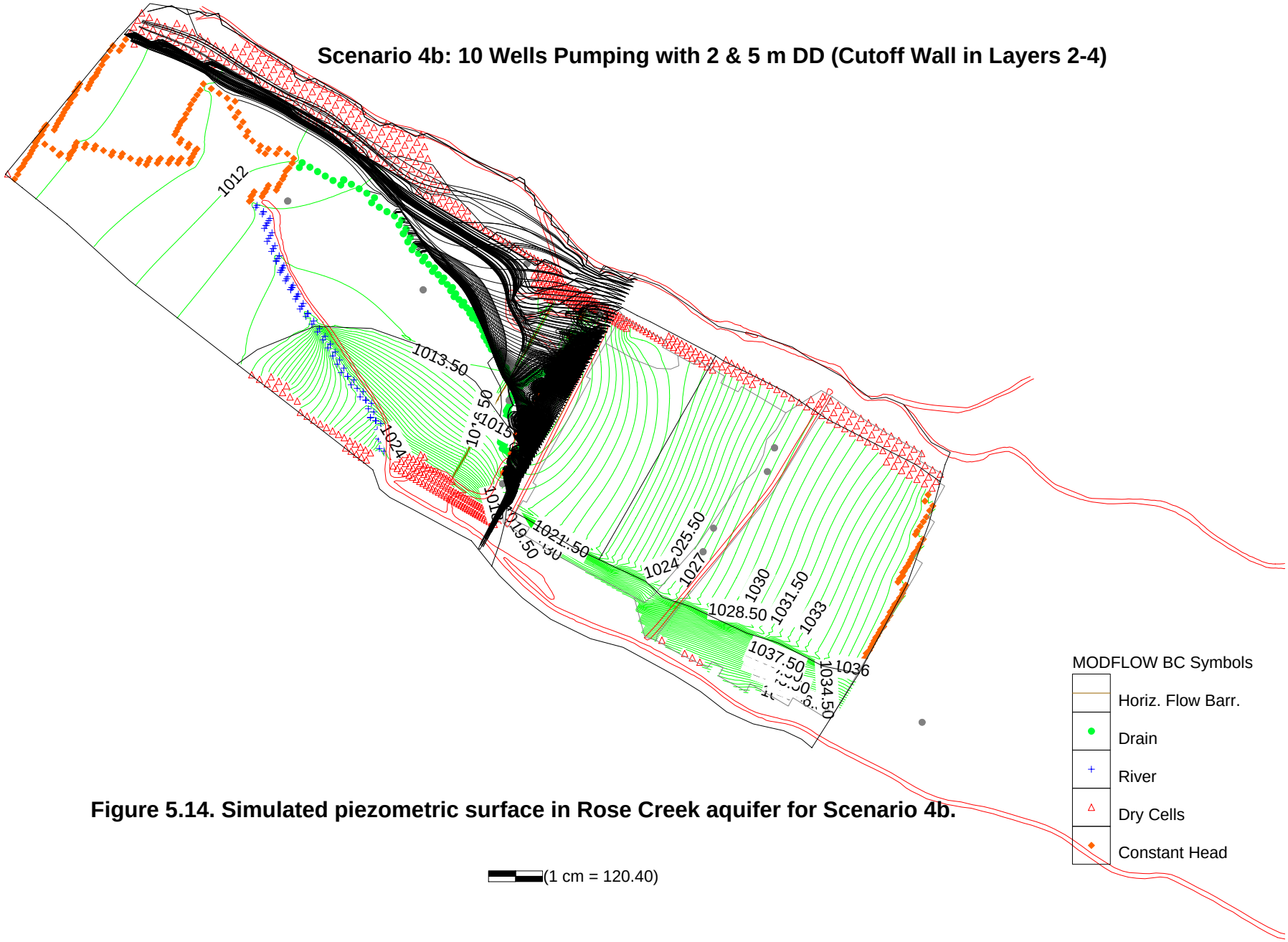


Figure 5.14. Simulated piezometric surface in Rose Creek aquifer for Scenario 4b.

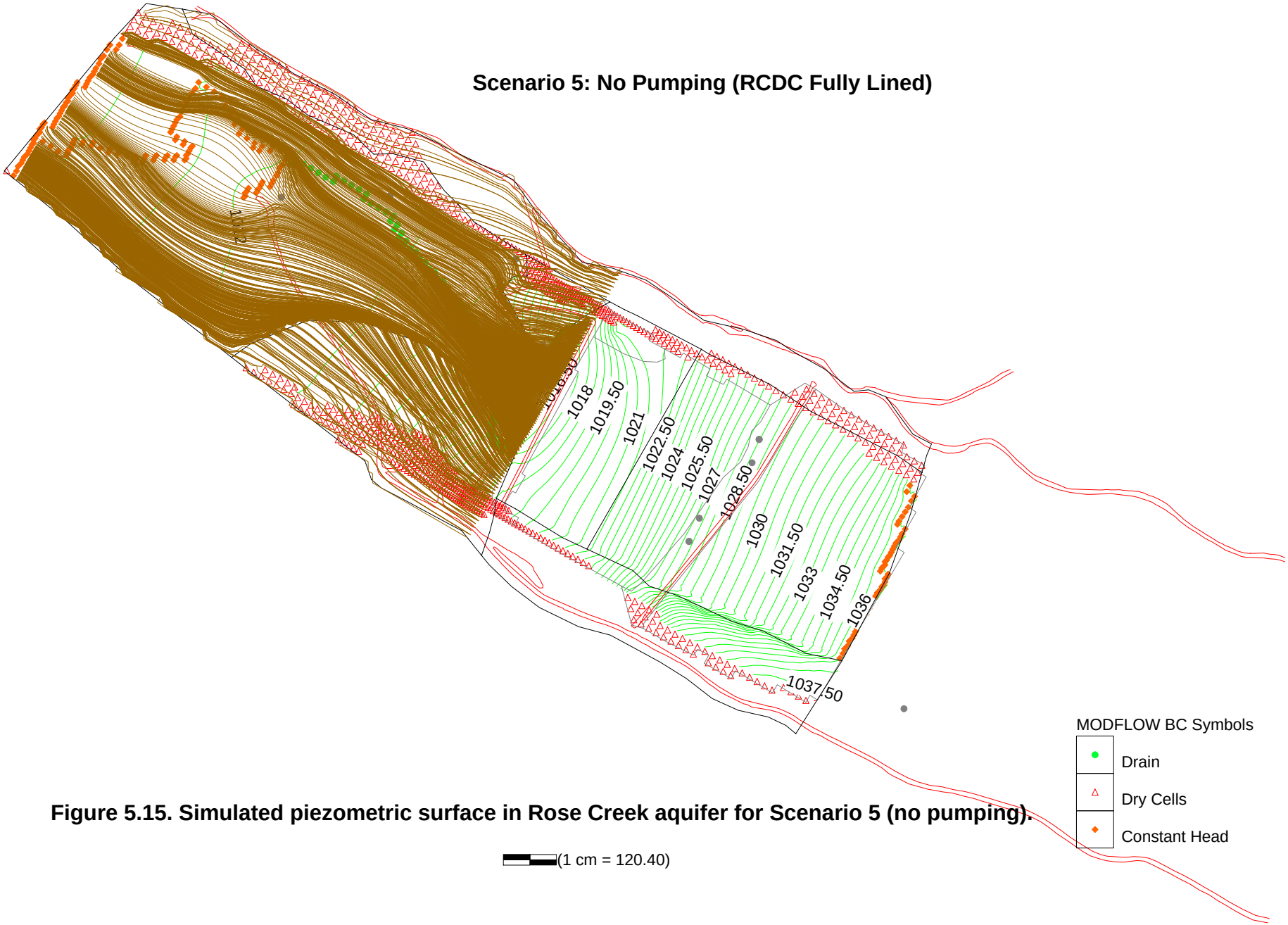


Figure 5.15. Simulated piezometric surface in Rose Creek aquifer for Scenario 5 (no pumping).

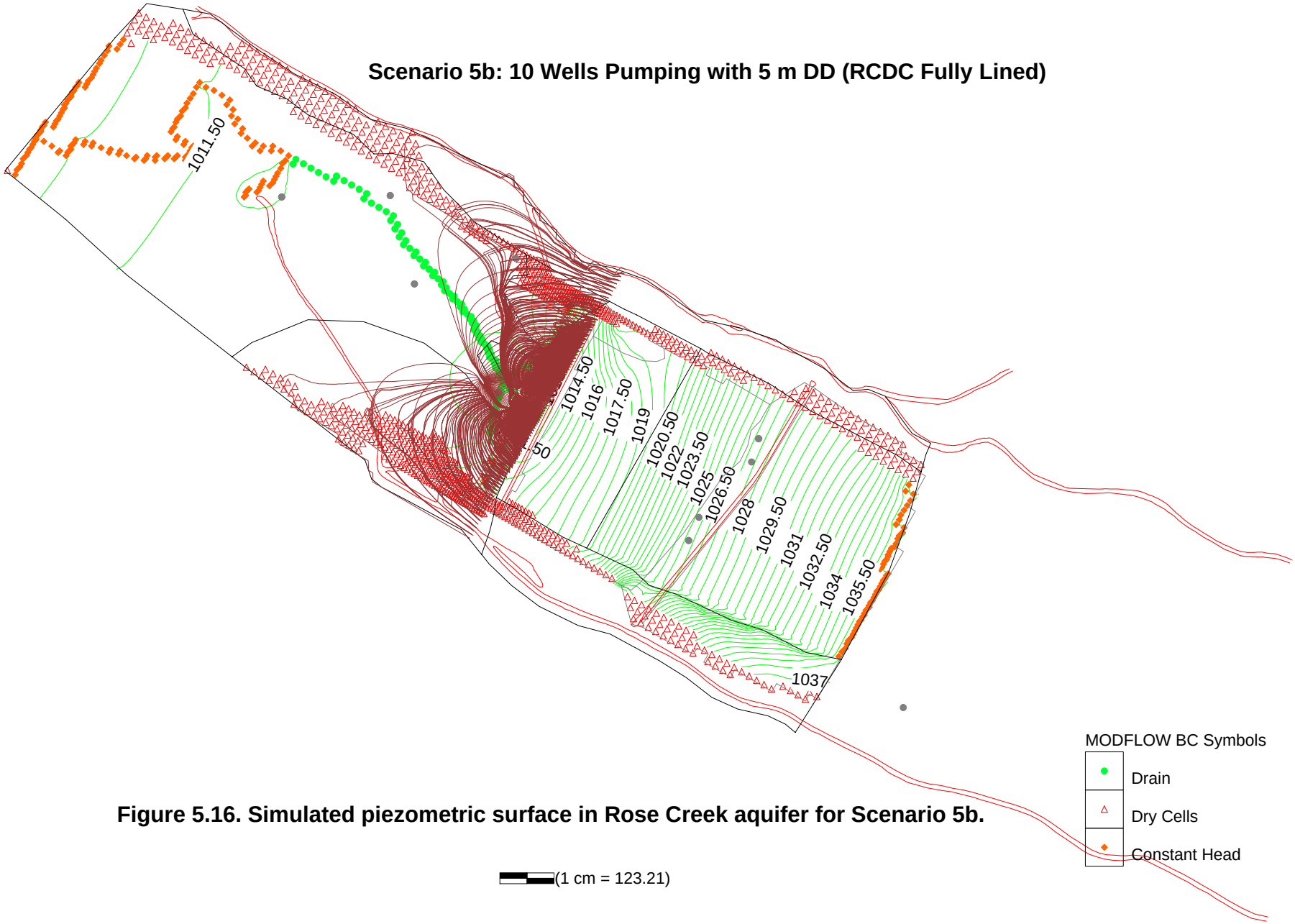


Figure 5.16. Simulated piezometric surface in Rose Creek aquifer for Scenario 5b.

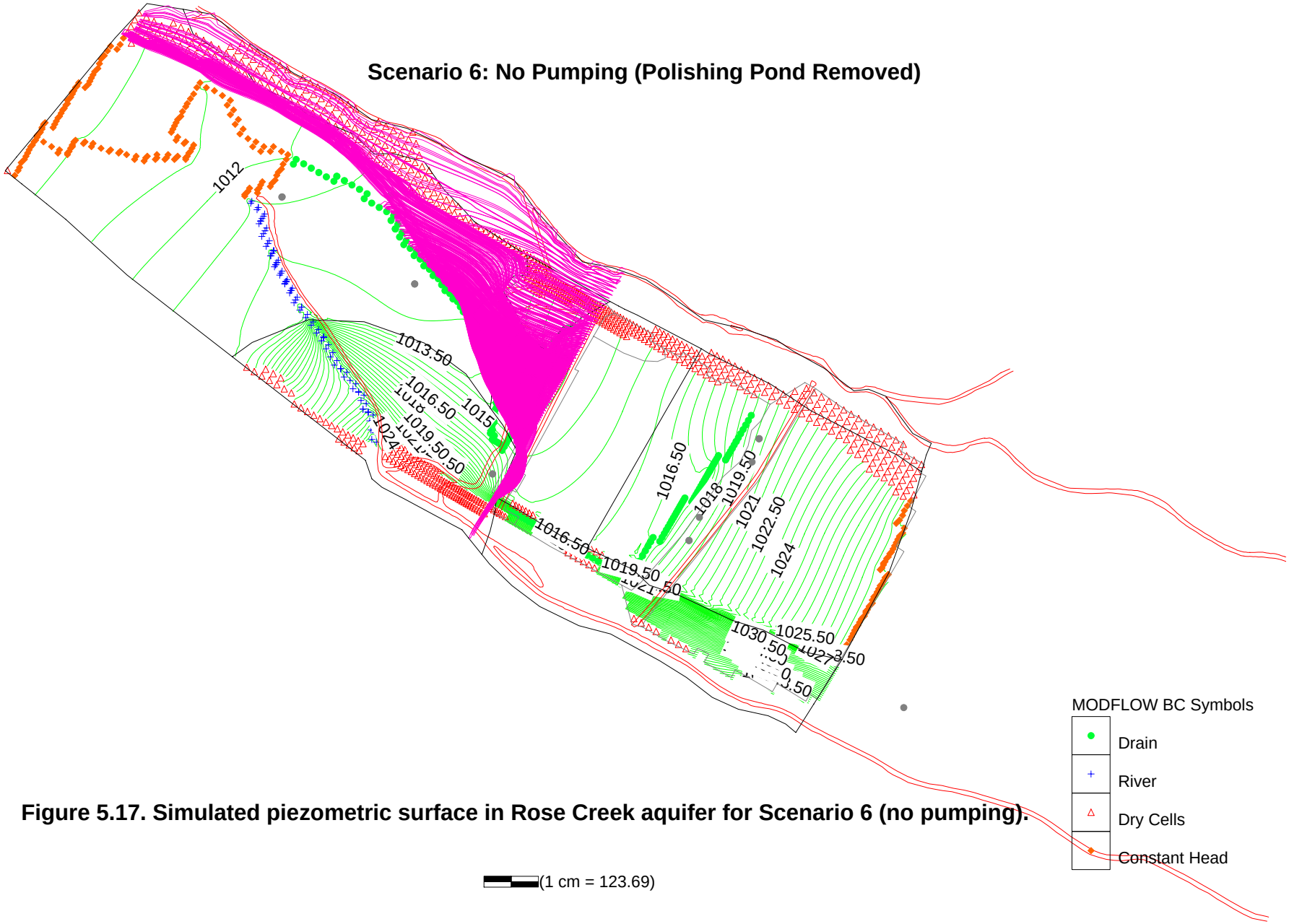


Figure 5.17. Simulated piezometric surface in Rose Creek aquifer for Scenario 6 (no pumping).

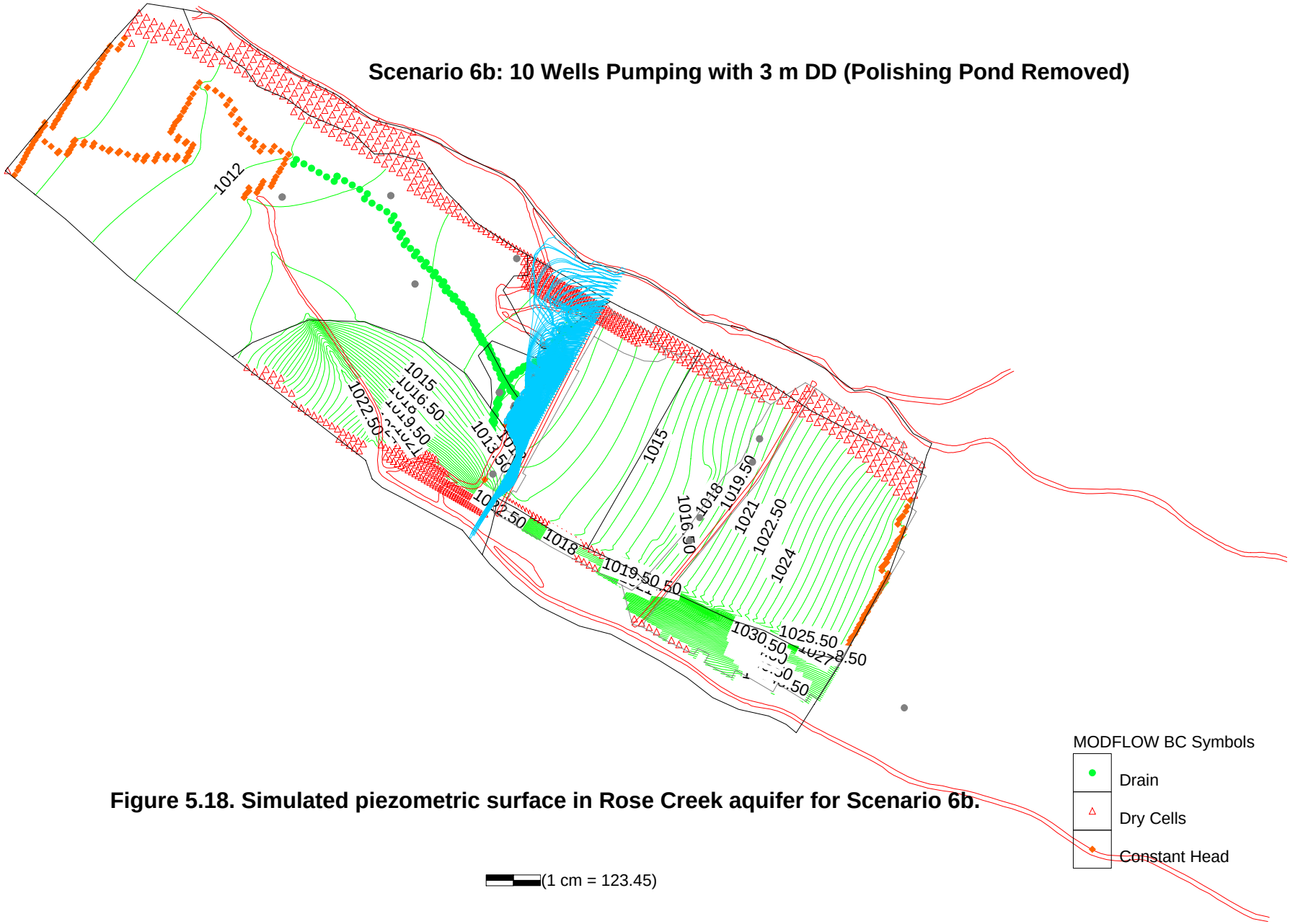


Figure 5.18. Simulated piezometric surface in Rose Creek aquifer for Scenario 6b.

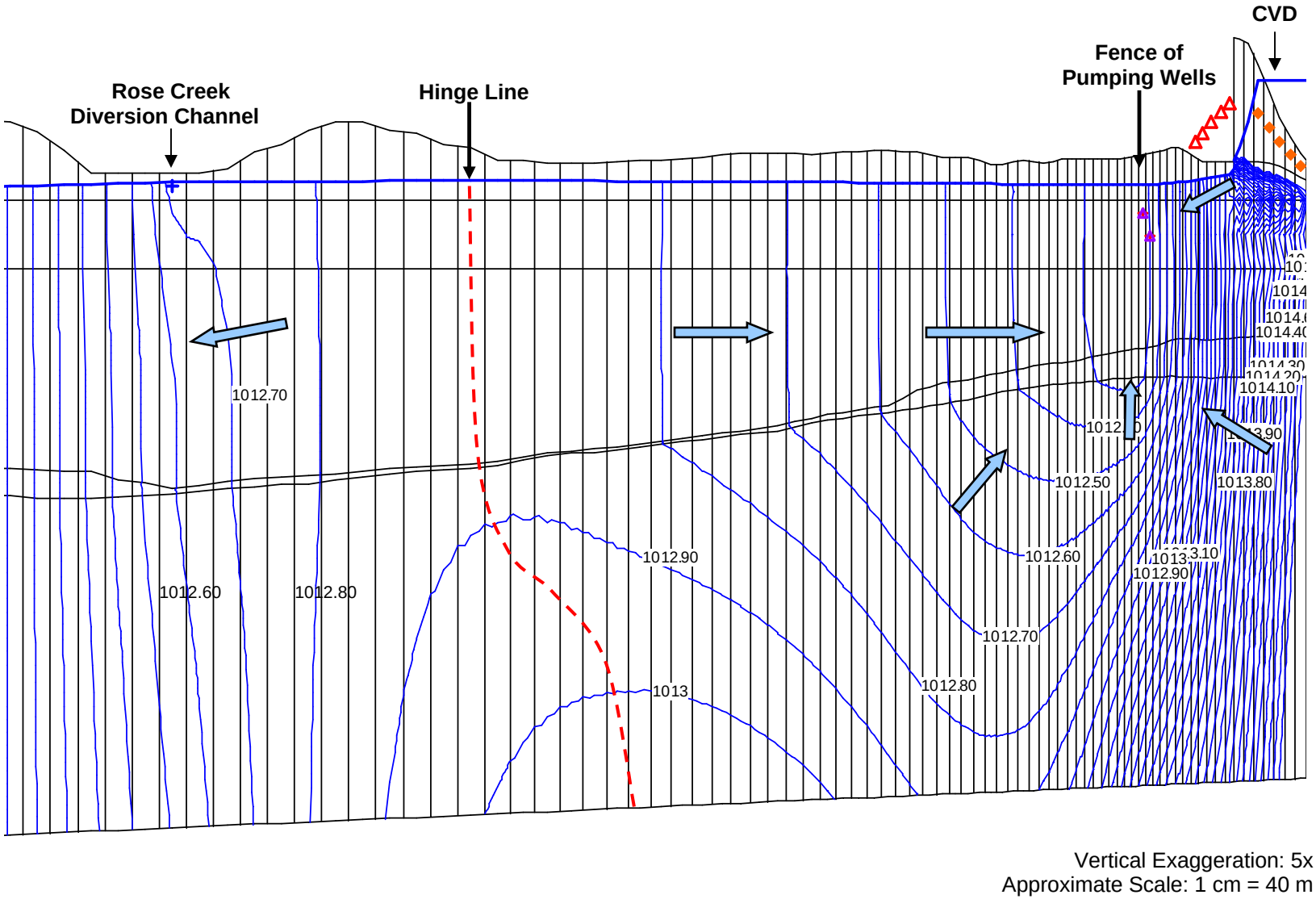


Figure 5.19. Simulated groundwater flow field in centre of Rose Creek aquifer for Scenario 2e (33_2_15).

6 RECOMMENDED DESIGN OF GROUNDWATER INTERCEPTION SYSTEM

Seepage from the Rose Creek tailings facility has been shown to carry highly elevated metal concentrations and this loading is predicted to significantly impact groundwater quality in the Rose Creek aquifer in the long-term (Robertson GeoConsultants Inc., 2006). The requirements for future groundwater interception depend on the closure scenario selected for the Rose Creek tailings facility, the timing of closure implementation and the attenuation capacity of the system. The loading predictions suggest that groundwater interception will likely be required for the long-term (> 750 years) if tailings are remediated *in-situ* and may still be required for an interim period if all tailings are removed from the Rose Creek valley (see Robertson GeoConsultants Inc., 2006 for details).

For the purpose of this study it was assumed that the groundwater interception system would be installed at the toe of the Cross Valley Dam. This location is located downstream of all potential contaminant sources (tailings dams and ponds) and offers easy access for testing, installation and operation. This would be particularly true for the initial period of rehabilitation (until 2020), i.e. as long as the existing dam configuration will likely remain in place. For the purpose of this pre-feasibility design, it was further assumed that the Cross Valley Dam and the Polishing Pond remain in place.

An adaptive management plan is proposed for the design and implementation of the groundwater interception system (GIS) that would ensure that the required capture efficiency is met with a high degree of confidence. The initial design will focus on the glacio-fluvial sediments in the Rose Creek valley which represents the primary pathway of future metal loading in the Rose Creek aquifer. An extensive monitoring network would be implemented to assess the performance of the initial interception system. If required, the initial design would then be upgraded using contingency measures that are clearly defined in the adaptive management plan.

6.1 Available Technologies

Numerous types and configurations of groundwater interception systems are potentially available for the Rose Creek aquifer. These technologies include:

- Pumping wells (with or without permeable trenches);
- Cut-off walls;
- Shallow sumps and trenches;
- Permeable reactive barriers (PRB); and
- Stream isolation.

A brief description of these technologies is provided in a report on seepage interception at the Faro Mine entitled "***2005 Seepage Investigation at the S-cluster area below the Faro Waste Rock Dump***"(SRK, 2006). The reader is referred to this companion report for a general discussion on these technologies.

6.2 Recommended Approach

The recommended approach of seepage interception in the Rose Creek aquifer is based on a combination of field experience (sections 2 and 3) and numerical modeling results (section 5). In

short, the results of numerical modeling assuming homogeneous, isotropic aquifer properties indicate that a fence of pumping wells screened across the entire thickness of glacio-fluvial sediments should be adequate to provide very high capture efficiency. However, field experience suggests that the aquifer is heterogeneous and that it may not be feasible to screen (and pump) all portions of this aquifer. Further work will be required to confirm that partially penetrating extraction wells can in fact achieve the very high capture efficiencies predicted by the model. If not, a deep cut-off wall would be required to prevent any off-site migration.

We therefore recommend a phased approach, potentially including several design components, for groundwater interception at the Cross Valley Dam. During the initial construction phase a fence of extraction wells would be installed at the toe of the Cross Valley Dam. This initial design would be supplemented with a surface collection system for any residual toe seepage. The installation of a deep cut-off wall would be included in this initial design as a contingency measure. Additional extraction wells located upstream or downstream of the recommended alignment of the fence of extraction wells would be included as further contingency measures.

Contingency remedial phases would be implemented in a timely manner according to a well-defined adaptive management plan integrated with an extensive monitoring network.

6.2.1 Initial GIS

The pumping tests have shown that pumping of the alluvial aquifer in this area is a viable option to intercept any potentially impacted groundwater. However, the numerical modeling results suggest that a large number of extraction wells (10 or more) will be required to reduce the individual pumping rates in the individual extraction wells to such levels that are sustainable in the field.

For the purpose of this pre-feasibility design, we have assumed the installation of ten (10) extraction wells at a horizontal spacing of 55 metes aligned parallel to the toe of the Cross Valley Dam (see figure 6.1 for proposed locations). For costing purposes, we have assumed that four (4) extraction wells will be installed in high permeability (K) sediments and six (6) extraction wells will be completed in low permeability (K) sediments (see section 6.3). Note that the two newly completed extraction wells PW-1 and PW-2 (both screened in high K sediments) are included in the proposed fence of extraction wells. The proposed extraction wells would be screened, to the extent possible, across the entire depth of the Rose Creek valley sediments and keyed into the weathered/fractured bedrock (if shown to be permeable). Screening of silt layers of significant thickness (say >0.3m) should be avoided to minimize well development time and the potential for fines intake during long-term pumping. Furthermore, the top of the uppermost screen should be set at least 5-7m below the static groundwater table to prevent desaturation of the well screen during pumping.

Water from all pumping wells would be directed to a pipeline leading to a water treatment plant.

The groundwater modeling predicted that all toe seepage will cease once the GIS is fully operating. Nevertheless, provisions have been made in this initial design to install a shallow concrete sump downstream of monitoring station X13. In the short-tem, this sump would intercept any residual toe seepage that may still occur during initial optimization of the pumping system. In the long-tem, this sump would prevent the discharge of any toe seepage during a temporary failure of the primary groundwater extraction system (e.g. pump failure; power outage). This sump would be equipped with a pump that feeds into the pipeline to the water treatment plant

During this initial phase of construction of the groundwater interception system, no cut-off wall would be installed. However, a detailed monitoring program would be put in place to monitor the performance of the GIS (see below). The performance of the GIS would be tested immediately after construction to ensure that the fence of extraction wells (as constructed) is capable of reversing groundwater gradients throughout the aquifer (and underlying bedrock) and can achieve the required capture efficiency. If this initial testing suggests that pumping can not achieve the required capture efficiency (e.g. due to significant heterogeneity) then a cut-off wall (extending down to bedrock, if required) would be installed immediately downgradient of the fence of pumping wells (see Figure 6.1 for proposed alignment).

Note that the required vertical and lateral extent of such a cutoff wall cannot be determined from the available information. For the purpose of preliminary costing, we have assumed the installation of a fully penetrating ("complete") cut-off wall spanning the entire width of the Rose Creek valley as a worst-case contingency (see section 6.3). However, in our opinion, there is only a relatively small probability that such a "complete" cut-off wall would be required. More likely, "partial" cut-off wall(s) may only be required in certain portions of the aquifer where small-scale heterogeneity is significant and contaminant loads are relatively high (e.g. along the northern side of the Rose Creek aquifer).

6.2.2 Initial Monitoring System

Figure 6.1 shows the layout of the initial monitoring network for the GIS at the Cross Valley Dam. Monitoring wells would be installed in two rows: (i) along the alignment of the fence of pumping wells and (ii) downgradient of the GIS alignment. The upstream monitoring wells would be located halfway between two adjacent extraction wells and along the perimeter of the GIS (Figure 6.1). The downstream monitoring wells would be located approximately 100m downstream of the extraction wells. Modeling results suggest that at this distance the groundwater levels should be notably higher than at the fence of extraction wells (by about 0.1 to 0.2m) but still well within the capture zone of the GIS (see Figure 5.19). About half of the proposed monitoring wells would be drilled at least 10 m into weathered/fractured bedrock and completed as multi-level piezometers with 4-5 ports spaced evenly throughout the depth of the aquifer (including one port in weathered/fractured bedrock). The remaining monitoring wells would be drilled to a depth of about 10-15m below the water table and completed as conventional (single 2-inch) monitoring wells. The existing monitoring wells in the area (in particular the multi-level piezometers P03-09 and P05-01 as well as the downgradient monitoring wells X16A/B, X17A/B, X18A/B and P01-01A/B) would complement the proposed monitoring network (Figure 6.1).

The monitoring network would include one existing monitoring station for toe seepage (at X13) as well as two existing surface water monitoring stations X10 and X14 at the end of the RCDC and Rose Creek, respectively. In addition, a new surface water monitoring station ("XRC") would be established further downgradient in Rose Creek (see Figure 6.1). This new monitoring station is proposed because previous stream flow surveys have indicated significant gains in stream flow downstream of X14, possibly indicating significant groundwater discharge along this reach (Robertson GeoConsultants Inc., 2006). A new weir would be installed at stations X10 and (proposed) XRC to allow more accurate flow measurements to be carried out.

The combined data from the monitoring system would be assessed for three components:

1. Groundwater gradients;
2. Groundwater concentrations; and
3. Creek load.

Groundwater levels in all monitoring wells along the alignment of the GIS would be compared to groundwater levels immediately downgradient to assess the hydraulic performance of the capture system, namely, that the induced gradient is towards the GIS. Water levels in the conventional monitoring wells would be monitored continuously using data loggers to provide detailed information on system performance, at least during initial stages of GIS activation. Water levels in the multi-level piezometers would be monitored manually and less frequently (daily during the initial system start-up and weekly or possibly monthly thereafter).

Monitoring of groundwater concentrations and creek load would be conducted on a quarterly basis, including base flow conditions in the winter and high flow conditions during the spring freshet. System performance would be assessed by comparing groundwater concentrations with pre system installation levels, and monitoring any changes in contaminant loading in Rose Creek. Water quality parameters would likely include the metals of concern (e.g. zinc) plus “indicator” parameters such as sulphate. These indicator parameters are also significantly elevated in the tailings seepage but are not attenuated to the same extent as the metals of concern (e.g. zinc). Hence, monitoring of sulphate (or similar indicator) downgradient of the GIS would allow for an early detection of any potential seepage by-pass.

Intercepted groundwater would also be monitored. Flow meters would be installed on pumping wells and, combined with samples of pumped water, would be used to determine total load captured.

The pumping system would be equipped with remote monitoring capabilities and a battery operated alarm system to facilitate monitoring of the pumping system and to allow a rapid response in the event of system failure (e.g. pump failure, power failure).

6.2.3 Adaptive Management Program

Performance of the GIS would be assessed regularly during operation using groundwater and surface water data from the monitoring system. If groundwater or surface water parameters were observed to have reached or surpassed monitoring triggers (parameter levels indicating constant or rising contaminant concentrations or inappropriate hydraulic gradients), remedial actions would be implemented.

The monitoring system can be sub-divided into two main areas, each of which may have surface water and groundwater components:

1. Area adjacent to GIS; and
2. Area downstream of GIS;

In both areas, the first action upon reaching a monitoring trigger would be an investigation into the cause and determination of appropriate remedial steps. If trigger values would still be reached after these initial remedial action steps, then additional investigations or remedial actions would be taken as described below.

Area Adjacent to GIS

In this area, the monitoring system would consist primarily of multi-level groundwater monitoring wells, located both in line with the fence of pumping wells and down gradient of the GIS itself. The most likely cause for trigger would be contaminant bypass, either between adjacent extraction wells (in the permeable sediments or underlying till/bedrock) or around the edges of the GIS. Triggers of this type could be caused by malfunctioning of the pumping system, pump inefficiency caused by heterogeneity in the overburden materials, or inability to capture flow in the bedrock system. Groundwater monitoring would include two components:

1. Hydraulic gradients; and
2. Groundwater concentrations.

Hydraulic gradients would be monitored primarily between monitoring wells located along the fence of extraction wells and monitoring wells located downgradient of the GIS. The indicator (trigger) for inadequate hydraulic performance would be the absence of a positive gradient towards the GIS (i.e. reversal of hydraulic gradient downstream of the GIS). Lack of adequate drawdown in water levels at these monitoring locations would suggest pump inefficiency or improper spacing. Remedial actions would include an investigation of the pumping system followed by repair or installation of additional pumping wells, as necessary. If no cut-off wall had been installed during the initial construction phase, then installation of such a cut-off wall (partial or complete) could also represent a remedial option.

Monitoring at the downgradient fence of monitoring wells would also include groundwater quality. If the GIS was operating as intended, groundwater quality in these downgradient monitoring wells should improve significantly over time. The indicator (trigger) for an investigation would be a lack of any significant reduction in contaminant concentrations over the short-term (2-3 years) and concentration levels above typical background values for the local groundwater in the area over the mid-term (> 5 to 10 years). If the initial investigation suggested edge bypass, the GIS system (fence of wells and optional cut-off wall) would be extended towards the valley sides. If the investigation suggested that underflow through the deeper bedrock system was occurring, options for grouting and/or additional pumping in the bedrock would be investigated.

The proposed surface water monitoring in this reach includes monitoring for toe seepage and associated contaminant load collected in the surface water collection system (at X13). This flow would be monitored by monitoring pumping hours and flow rates for the toe seepage intercepted and pumped to the water treatment plant. The observation of any contaminated toe seepage occurring during normal pumping operations would represent a trigger for further investigations. First, the nature of the toe seepage (amount and water quality) would be investigated. If it could be shown that contaminated toe seepage occurred despite routine operation of the extraction wells than changes to the operation of the GIS (i.e. increased drawdown) should be evaluated. If increased pumping in the Rose Creek aquifer could also not eliminate contaminated toe seepage (e.g. due to perched seepage through the Cross Valley Dam) than the collection system for toe seepage would have to be upgraded to allow long-term operation.

Area downgradient of GIS

Both surface water and groundwater would be monitored in the downstream reach between X13 and proposed station XRC and compared to baseline trends. The groundwater trigger for investigation and remedial action would be a lack of reduction in contaminant concentrations below baseline concentrations in local groundwater. However, greater time horizons (2-5 years) will be required for a reduction in contaminant levels in the downstream monitoring wells (X16A/B, X17A/B, X18A/B and P01-01A/B) to take effect because of the greater distance from the GIS.

Surface water monitoring in this reach includes stream flow and water quality allowing an estimation of contaminant loading in Rose Creek (at stations X10 and/or XRC). The surface water trigger would be a lack of reduction in the net contaminant loading to Rose Creek along this reach to an acceptable level[7].

A detailed investigation into the likely causes for any lack in load reduction (or potential increase in contaminant loading), including a comparison with upgradient system performance monitoring, would be required before additional remedial action would be taken.

If the investigation indicated that contamination is by-passing the GIS (which should be corroborated with the monitoring in immediate vicinity of the GIS), then the GIS would be improved (see above). If the investigation indicated continued loading along this reach to Rose Creek despite proper hydraulic performance of the GIS then other potential sources of contamination (e.g. leakage from RCDC or historic tailings in this area) would have to be investigated. Potential remedial options to control those additional sources would include (i) installation of additional extraction wells downgradient of the GIS, or (ii) removal of any additional contaminant sources (e.g. historic tailings).

As a final contingency, if initial remedial actions at the toe of the Cross Valley Dam and/or the downstream reach could not provide adequate capture to protect the downstream aquatic environment, Rose Creek could be isolated along a reach further downgradient (between RCDC and XRC) to prevent the discharge of impacted groundwater into the stream along this reach. This would provide additional room for the installation of additional groundwater interception components (extraction wells, cut-off walls etc.) further downgradient.

6.3 Preliminary Costing

A preliminary cost estimate was completed to supply and install the recommended groundwater interception system (capital costs only). A costing of the annual operating and monitoring costs for the groundwater interception system (GIS) was beyond the scope of this study

6.3.1 Primary Groundwater Interception System

The primary groundwater interception system includes the fence of pumping wells plus the required monitoring network. The cost estimate for this primary GIS includes the following major items:

- Drilling and installation of pumping wells:

[7] The parameter of greatest concern is likely zinc. However, a specific contaminant load threshold required for closure along this reach of Rose Creek has not yet been finalized

  - 2 pumping wells drilled with 10-inch bit and outer casing and completed with 8-inch ID SS wire-wrap screens and steel casing (assume total depth of 30m);
  - 6 pumping wells drilled with 8-inch bit and outer casing and completed with 6-inch ID SS wire-wrap screens and steel casing (assume total depth of 30m);
  - Well development of each well (1 day each) using surging and air lifting;
- Drilling and installation of monitoring wells including
  - 13 multilevel piezometers drilled with 4 x 6 SONIC system (assume average depth of 35m) and completed with multilevel bundle (assume 6 sampling ports);
  - 8 single monitoring wells drilled with 4 x 6 SONIC system (assume average depth of 15m) and completed using 2-inch PVC screen and casing;
  - 8 downhole pressure transducers with downhole data loggers plus 1 barometer for barometric correction (e.g. Solinst Leveloggers plus Barologger or equivalent)
- Supply and installation of pumping system including:
  - 4 submersible pumps (7“ Ø) with pump capacity of 475 USGPM (e.g. Grundfos 475S400-4 or equivalent);
  - 6 submersible pumps (5“ Ø) with pump capacity of 100 USGPM (e.g. Grundfos 85S7-6 or equivalent);
  - Flow controller (Variable Frequency Drive), flow meters and sampling ports;
  - Pressure transducers and data loggers to log and remotely monitor pumping rates, pumping levels in well and pump pressure;
  - Dial-out alarm system;
  - Protective housing at each well head (for controls and electrical).
- Supply and installation of surface water collection system including:
  - 1 concrete sump at X13;
  - 1 sump pump w/ controls;
- Supply and installation of surface water monitoring stations including:
  - 2 streamflow gauging stations (at X10 and proposed XRC);
  - Pressure transducers, data loggers & heated hut

The cost for drilling and installation of the pumping wells and monitoring wells was based on the actual cost incurred for very similar work during the 2005 drilling program (section 2). A 10% mark-up was assumed for all unit rates and materials to account for inflation.

The cost for the supply and installation of the pumping system was based on a quote provided by Precision Services & Pumps (see Appendix I).

The cost for supply and installation of the surface water collection system and installation of surface water monitoring stations were estimated based on the experience of the author.

Note that power supply to the CVD and hook-up of the pumping system is not included in this costing. The cost of a pipeline to the water treatment plant (including hook-up of well heads to pipeline) is also not included in this preliminary costing.

Table 6.1 summarizes the preliminary cost estimate for the various components of the recommended initial design of the groundwater interception system (in 2006 Can $). A detailed costing showing assumed unit rates and itemized costs is provided in Table I1 (Appendix I). The total direct costs are estimated at Can $1.5 million. The indirect costs include project management (5% of direct costs), field engineering and QA/QC (15% of direct costs) plus field supervision (7% of direct costs). A contingency of 20% of direct costs was assumed to account for problems/delays during drilling and installation work.

Table 6.1

Preliminary cost estimate for recommended initial design of groundwater interception system (GIS)

Table 6.1: Preliminary Cost Estimate for Recommended Initial Design of GIS

Direct Costs	**Cost**
Item	Can $
1.0 Drilling & Installation of Pumping Wells (Air Rotary)	**$237,940**
2.0 Drilling & Installation of Monitoring Wells (SONIC)	**$780,645**
3.0 Supply & Installation of Pumping System (see Appendix I1 for details)	**$505,400**
4.0 Supply and installation of surface water collection system	**$10,000**
5.0 Supply and installation of surface water monitoring stations	**$10,000**
TOTAL Direct Costs	**$1,543,985**
Indirect Costs	
Item	
Project Management (5% of direct costs)	**$77,199**
Field Engineering & QA/QC (15% of direct costs)	**$231,598**
Field Supervision (7% of direct costs)	**$108,079**
TOTAL Indirect Costs	**$416,876**
Contingency	
Item	
Contingency (20% of direct costs)	**$308,797**
Total Capital Costs	
TOTAL (before taxes)	**$2,269,658**

The estimated total cost (including indirect costs and contingency) for the installation of the recommended initial design of the groundwater interception system is Can $2.3 million plus applicable taxes.

It should be emphasized that the cost of further site characterization work that may be required prior to final design and installation of the GIS is not included in this cost estimate.

6.3.2 Performance Contingency – Cut-off Wall

As discussed in section 6, a cut-off wall may be required if the recommended initial design of the groundwater interception system (i.e. a fence of pumping wells) does not provide adequate drawdown and capture efficiency. At the present time, there is insufficient data available to assess whether such a cut-off wall would be required, and if, what the exact dimensions (lengths, depth) of this wall would be.

For the purpose of this report, a preliminary cost estimate was obtained for the "worst-case" scenario, i.e. installation of a complete cut-off wall spanning the entire width of the Rose Creek valley (Figure 6.1). The following assumptions were made for this costing:

- Installation of a 1m wide soil/bentonite cut-off wall with an effective hydraulic conductivity of less than $1*10^{-8}$ m/s;
- Conventional slurry wall construction using both an extended backhoe reach and a clamshell;
- The total length of the cut-off wall would be approximately 500m with a maximum depth of about 35m; based on figure 2.2, the cross-sectional area to be sealed with the cut-off wall was estimated at 17,000 m^2;

Our preliminary cost estimate for the installation of a cut-off wall was based on unit rates provided by Hayward Baker (Mark Koelling, HBI Seattle, pers. comm.). This company is a large construction company with extensive experience in this type of earth works.

Table 6.2 summarizes the preliminary cost estimate for this contingency measure (in 2006 Can $). The total direct costs are estimated at Can $7.1 million. The indirect costs include project management (5% of direct costs), field engineering and QA/QC (15% of direct costs) plus field supervision (7% of direct costs). A contingency of 30% of direct costs was assumed to account for problems/delays during installation work. A higher contingency was assumed for this work because of the potential presence of boulders which would slow down installation work.

The estimated total cost (including indirect costs and contingency) for the installation of a complete cut-off wall (down to bedrock) is Can $11.1 million plus applicable taxes.

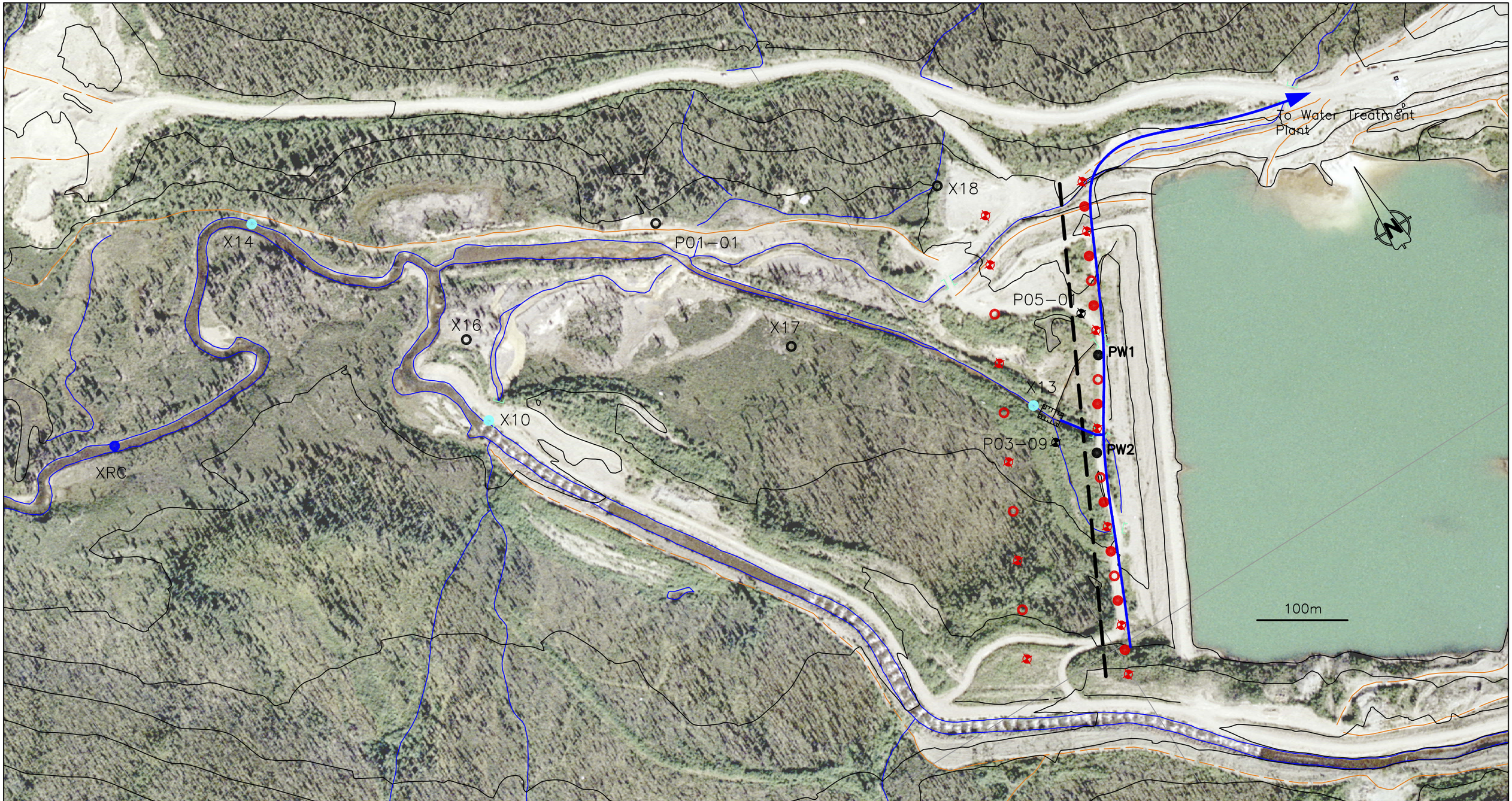
Note that additional site characterization work (e.g. Becker and CPT measurements) would likely be required prior to installation of the cut-off wall. The cost for such pre-installation testing work is not included in this cost estimate.

Table 6.2

Preliminary cost estimate for performance contingency (cut-off wall) for GIS

Direct Costs					**Cost**
Item					Can $
1.0	**Site Preparation**	1	l.s	$50,000	$50,000
2.0	**Mob/Demob**	1	l.s	$250,000	$250,000
3.0	**Installation of Cutoff Walls**	17,000	m2	$400	$6,800,000
	TOTAL Direct Costs				**$7,100,000**
Indirect Costs					
Item					
Project Management (5% of direct costs)					**$355,000**
Field Engineering & QA/QC (15% of direct costs)					**$1,065,000**
Field Supervision (7% of direct costs)					**$497,000**
	TOTAL Indirect Costs				**$1,917,000**
Contingency					
Item					
Contingency (30% of direct costs)					**$2,130,000**
Total Capital Costs					
	TOTAL (before taxes)				**$11,147,000**

It should be emphasized again that these costs assume that the cut-off wall would be constructed across the entire depth and thickness of the Rose Creek aquifer. In our opinion, there is only a relatively small probability that such a complete cut-off wall will be required. More likely, cut-off walls may only be required in certain portions of the aquifer where small-scale heterogeneity is significant and contaminant loads are relatively high (e.g. along the northern side of the Rose Creek aquifer). The cost of a “partial” cut-off wall can easily be derived from the unit rates provided in Table 6.2.



LEGEND

Existing SIS Components

- X14 — Surface Water Monitoring Station
- PW1 — Pumping Well
- X18 — Monitoring Well (single)
- P05–01 — Monitoring Well (multilevel)

Proposed SIS Components

- SW Monitoring Station
- Pumping Well
- Monitoring Well (single)
- Monitoring Well (multilevel)
- Sump
- Deep cutoff wall (as required)
- Pipeline to water treatment plant

CLIENT:	Deloitte & Touche Inc.
PROJECT:	Rose Creek Groundwater Interception
REPORT:	118004
FILE:	fig5–1 pump model.dwg
LOCATION:	Faro Mine, Yukon Territory, Canada

ROBERTSON GEOCONSULTANTS INC.
Consulting Geotechnical and Environmental Engineers

Proposed Design of Initial Groundwater Interception System

DATE:	DRAWN BY:		FIGURE:
July, 2006	MP		6.1

7 CONCLUSIONS AND RECOMMENDATIONS

Seepage from the Rose Creek tailings facility has been shown to carry highly elevated metal concentrations and this loading is predicted to significantly impact groundwater quality in the Rose Creek aquifer in the long-term (Robertson GeoConsultants Inc., 2006). It was concluded that a groundwater interception system would likely be required to protect the water quality in Rose Creek.

As part of the 2005/06 closure studies, a comprehensive groundwater study was carried out under Task 22H(ii) entitled "Complete Groundwater Collection Design". The primary objective of this study was to design a groundwater interception system (GIS) for the Rose Creek Tailings Storage Facility. This study had three major components:

- A field program including installation of 2 pumping wells at the toe of the Cross Valley Dam and pumping tests of the Rose Creek aquifer;
- A groundwater modeling program including development and calibration of a groundwater flow model for the lower Rose Creek aquifer and simulation of alternative groundwater interception scenarios; and
- Design of a groundwater interception system at the toe of the Cross Valley Dam (including preliminary costing).

The following sections summarize the major findings of this study and provide recommendations for future work.

7.1 Conclusions

7.1.1 Field Program

Two large diameter (8") pumping wells were installed in the northern and south-central portion of the Rose Creek aquifer at the toe of the Cross Valley Dam (PW1 and PW2, respectively). Both pumping wells are screened in highly productive sand & gravel layers and are capable of pumping significant quantities of groundwater. In addition, five standard monitoring wells and one multi-level well were installed at the toe of the Cross Valley Dam (CVD).

Several pumping tests were carried out to determine the well yield of the newly completed pumping wells and to estimate the hydraulic properties (transmissivity, storage) of the Rose Creek aquifer. The results indicated that groundwater yields in the south-central portion of the aquifer are generally higher than on the north side, sustaining pumping rates in excess of 45 L/s (600 USGPM) compared to an estimated 23-30 L/s (300-400 USGPM) on the north side. The pumping tests indicated that groundwater pumping from extraction wells is a viable control strategy for intercepting contaminated groundwater in the Rose Creek aquifer.

The pumping test data were first interpreted using analytical solutions to obtain initial estimates of transmissivity (T) and storage coefficient (S) for the Rose Creek aquifer. Normalized drawdown plots showed a large scatter clearly indicating that the Rose Creek aquifer is not an "ideal aquifer", likely due to heterogeneity within the aquifer and/or leakage from semi-confining layers. Because of these non-ideal aquifer conditions only the drawdown in the pumping well and the nearest monitoring wells was interpreted using analytical solutions. The Cooper-Jacob analysis of the early drawdown data

suggested an average transmissivity of ~1,000 m^2/day in vicinity of PW1 (north side) and an average transmissivity of ~1,500 m^2/day in vicinity of PW2 (south-central side).

The estimated storage coefficient values for the PW1 test ranged from 0.02 to 0.04. These storage coefficients are higher than typical values for confined aquifers and are interpreted to be a composite result of pumping an unconfined upper layer (screened by the upper well screen) and a confined lower layer (screened with the lower well screen). Leakage effects may also influence the interpretation of the storage coefficient. Interpretation of the drawdown data for the PW2 test suggests storage coefficient values ranging from 0.0003 to 0.0015.

7.1.2 Groundwater Modeling Program

A three-dimensional groundwater flow model was developed for the lower reach of the Rose Creek aquifer using the finite difference model MODFLOW[96]. For the purpose of this study, the Rose Creek aquifer system was sub-divided into six hydrostratigraphic units: (i) mine waste material, (ii) glacio-fluvial sediments in the valley center; (iii) colluvial sediments (in selected areas) (iv) till in the valley center (v) slope till on the valley sides; (vi) slightly weathered to fresh bedrock underlying all overburden sediments. Most hydrostratigraphic units were represented by a single model layer of variable thickness (based on a geological model). The permeable glacio-fluvial sediments were further subdivided into three model layers.

The groundwater flow model was first calibrated assuming steady-state conditions. This initial calibration was then followed with a transient calibration using the results of the two constant discharge pumping tests. The calibrated model provided a very good overall fit with observed (steady-state) groundwater levels (r^2=0.98) and reproduced the observed (steady-state) seepage at the toe and downstream of the Cross Valley Dam very well. The calibrated model also reproduced the drawdown in groundwater levels and reduction in toe seepage observed during the pumping tests in PW1 and PW2.

The calibrated flow model suggests that the total flux of groundwater passing beneath the Cross Valley Dam is about 58 L/s. The net seepage from the Intermediate Pond and Polishing Ponds were estimated to be 9.3 L/s and 14.3 L/s, respectively. According to the model, leakage from the RCDC represents a major source of water to the southern portion of the aquifer (38.7 L/s). Particle tracking suggested that this leakage from the RCDC “forces” the underflow beneath the Cross Valley Dam towards the central and northern portion of the aquifer where most of the flow discharges to the drain system. The calibrated model suggests that only a small portion of this potentially impacted groundwater is currently continuing further downgradient along the far northern side of the Rose Creek aquifer.

The calibrated groundwater flow model was used to predict the performance of alternative groundwater interception scenarios at the toe of the Cross Valley Dam. For the purpose of this discussion we assumed that the groundwater interception system would require a capture efficiency of 99% (i.e. a by-pass of less than 1%) for “successful” performance. The key findings of this modeling work can be summarized as follows:

- the use of 5 pumping wells is predicted to require a uniform drawdown of at least 5m for successful groundwater interception (Scenario 1f); the predicted total amount of groundwater

pumped in this scenario would be 72.9 L/s with individual pumping rates up to 44 L/s (or 700 USGPM);

- the use of 10 pumping wells is predicted to require drawdown of at least 4m in the 4 central wells (high K sediments) and 5m in the remaining wells for successful groundwater interception (Scenario 2e); the predicted total amount of groundwater pumped in this scenario would be 72.0 L/s with individual pumping rates up to 22 L/s (or 344 USGPM);
- the use of shallow pumping wells (only screened in the upper third of the Rose Creek aquifer, Scenario 3a) is predicted to significantly reduce system performance whereas partial screening in the middle or lower third of the Rose Creek aquifer sediments (Scenarios 3b/c) was predicted to only marginally reduce system performance;
- the use of a shallow cut-off wall (Scenario 4a) is predicted to provide virtually no benefit to system performance whereas a fully penetrating barrier (Scenario 4b) significantly improves system performance; the primary benefit of installation of a "fully penetrating" cut-off wall is that a significantly lower drawdown is required in the pumping wells to achieve the same degree of capture;
- complete lining of the RCDC (to prevent leakage) is predicted to reduce the amount of groundwater collected (for a given drawdown) but would not change the system performance (i.e. total or relative by-pass flow) significantly;
- removal of the Cross Valley Dam and the Polishing Pond is predicted to result in significant flooding in this foot print area which would require the installation of a shallow collection system at the toe of the Intermediate Dam; the model predicts that toe seepage collected in this new drain system could be as high as 31 L/s; at the same time, the groundwater flow to be collected downstream of the (former) CVD is predicted to decrease to 13.9 L/s (a fourfold reduction);

It should be emphasized that the calibrated groundwater flow model is a simplified representation of highly complex subsurface conditions observed in the Rose Creek valley. The key uncertainties in the current version of the model include (i) the spatial distribution of hydrostratigraphic units (in particular towards the valley sides) and (ii) small-scale heterogeneity and its effect on vertical anisotropy. These uncertainties could significantly influence the design of a groundwater interception system (# of wells, screening interval, pumping rates). Additional field work and model calibration will therefore be required prior to final design of a groundwater interception system (see section 7.2).

7.1.3 Recommended Design of a Groundwater Interception System

For the purpose of this study it was assumed that the groundwater interception system would be installed at the toe of the Cross Valley Dam. This location is located downstream of all potential contaminant sources (tailings dams and ponds) and offers easy access for testing, installation and operation. This would be particularly true for the initial period of rehabilitation (until 2020), i.e. as long as the existing dam configuration will likely remain in place. For the purpose of this pre-feasibility design, it was further assumed that the Cross Valley Dam and the Polishing Pond remain in place.

An adaptive management plan is proposed for the design and implementation of the groundwater interception system (GIS) that would ensure that the required capture efficiency is met with a high

degree of confidence. The initial design would focus on the glacio-fluvial sediments in the Rose Creek valley which represents the primary pathway of future metal loading in the Rose Creek aquifer. An extensive monitoring network would be implemented to assess the performance of the initial interception system. If required, the initial design would then be upgraded using contingency measures that are clearly defined in the adaptive management plan.

For the purpose of this pre-feasibility design, we recommend the installation of ten (10) extraction wells at a horizontal spacing of 55 metes at the toe of the Cross Valley Dam. For costing purposes, we have assumed that four (4) extraction wells will be installed in high permeability (K) sediments and six (6) extraction wells will be completed in low permeability (K) sediments. The two newly completed extraction wells PW-1 and PW-2 (both screened in high K sediments) are included in the proposed fence of extraction wells. The proposed extraction wells would be screened, to the extent possible, across the entire depth of the Rose Creek valley sediments. If required, these pumping wells would be extended into the shallow (weathered and/or fractured) bedrock underlying the sediments.

During this initial phase of construction of the groundwater interception system, no cut-off wall would be installed. However, a detailed monitoring program would be put in place to monitor the performance of the GIS (see below). The performance of the GIS would be tested immediately after construction to ensure that the fence of extraction wells (as constructed) is capable of reversing groundwater gradients throughout the aquifer (and underlying bedrock) and can achieve the required capture efficiency. If this initial testing suggests that pumping can not achieve the required capture efficiency (e.g. due to significant heterogeneity) then a cut-off wall (possibly extending down to bedrock, if required) would be installed immediately downgradient of the fence of pumping wells.

Provisions have also been made in this initial design to install a shallow concrete sump downstream of monitoring station X13. In the short-tem, this sump would intercept any residual toe seepage that may still occur during initial optimization of the pumping system. In the long-tem, this sump would prevent the discharge of any toe seepage during a temporary failure of the primary groundwater extraction system (e.g. pump failure; power outage). This sump would be equipped with a pump that feeds into the pipeline to the water treatment plant

Water from all pumping wells and the sump would be directed to a pipeline leading to a water treatment plant.

The proposed monitoring network for the GIS includes:

- 15 deep multilevel monitoring wells (with ports throughout the sediments and up to 10m into weathered/fractured bedrock);
- 8 shallow conventional monitoring wells (completed to a depth of 15m);
- 8 (existing) regional monitoring wells;
- 1 toe seepage monitoring station, and
- 3 stream gauging stations in Rose Creek.

Performance of the GIS would be assessed regularly during operation using groundwater and surface water data from the monitoring system. If groundwater or surface water parameters were observed to have reached or surpassed monitoring triggers (parameter levels indicating constant or

rising contaminant concentrations or inappropriate hydraulic gradients), remedial actions would be implemented.

Potential remedial options to improve system performance include (i) installation of additional extraction wells along the proposed alignment, (ii) installation of additional extraction wells downgradient of the GIS, (iii) installation of cut-off walls (partial or fully penetrating) downgradient of the GIS and/or (iv) removal of any additional contaminant sources (e.g. historic tailings). As a final contingency, if initial remedial actions at the toe of the Cross Valley Dam and/or the downstream reach could not provide adequate capture to protect the downstream aquatic environment, Rose Creek could be isolated along a reach further downgradient (between RCDC and XRC) to prevent the discharge of impacted groundwater into the stream along this reach. This would provide additional room for the installation of additional groundwater interception components (extraction wells, cut-off walls etc.) further downgradient.

A preliminary cost estimate was completed for the installation of the recommended groundwater interception system (at the pre-feasibility level)). The total cost for installation of the initial design of the GIS (including monitoring network) was estimated at Can $2.3 million (including indirect costs and contingency). The estimated total cost for the installation of a complete cut-off wall (down to bedrock) was Can $11.1 million (including indirect costs and contingency)

7.2 Recommendations for Future Work

We recommend that the following future work be carried out prior to final design of a groundwater interception system:

- A 7-day pumping test should be conducted in the lower screen of PW-1 (the upper screen should be isolated using a packer system) to assess the influence of small-scale heterogeneity and vertical anisotropy on performance of a groundwater interception system. During the test, the drawdown and recovery in the sediments and underlying weathered/fractured bedrock should be monitored.
- Additional field work (drilling and hydraulic testing) should be carried out to better define the hydrogeological conditions along the sides of the Rose Creek valley near the CVD. For this purpose, one pumping well and two monitoring wells each should be installed on the north and south side of the proposed alignment of the fence of wells. On each side, one monitoring well should be screened in the overburden soils and one in the underlying weathered/fractured bedrock. A 24-hr pumping test should then be carried out at both new pumping wells to determine the hydraulic properties (transmissivity, storage) in the local sediments and underlying weathered/fractured bedrock and the hydraulic connection to the central portion of the valley.

The results of these two tests should be used to re-calibrate the groundwater flow model and, if required, update the design of the groundwater interception system. Once re-calibrated, additional sensitivity analyses should be carried out with the updated groundwater flow model to study the potential influence of (i) a weathered/fractured bedrock layer and (ii) recharge from the valley sides and underflow from upstream on the design and performance of a groundwater interception system.

In case, the removal of the CVD and the Polishing Pond (Scenario 6) was the preferred option for closure of the Rose Creek Tailings Facility, additional calibration work should be carried out to better

constrain the seepage rate from the Polishing Pond and the Intermediate Pond. The following activities are proposed for this work:

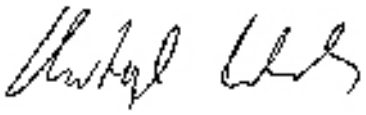
- Complete a water and load balance for the Polishing Pond and the Intermediate Pond for the inactive winter period (taking into account changes in the pond level and any inflows/outflows);
- Re-calibrate the groundwater flow model using transient monitoring results for the Cross Valley Dam area (incl. pond levels) observed for (i) a winter base flow period and (ii) a short-term pumping period.

8 CLOSURE

This report was prepared by Robertson GeoConsultants Inc. for the Faro Mine Closure Planning Office (FMCPO) and Deloitte & Touche Inc. in its capacity as Interim Receiver of Anvil Range Mining Corporation. No third-party engineer or consultant shall rely on any of the information, conclusions, opinions, or any other material contained in this report without the express written consent of the FMCPO, Deloitte & Touche Inc. and RGC.

We trust that the information provided in this report meets your requirements at this time. Should you have any questions or if we can be of further assistance, please do not hesitate to contact the undersigned.

Respectfully Submitted



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PHOTO LOG



Photo 1. Sonic drill rig setting up at P05-01.



Photo 2. Recovering soil core sample from the core barrel.



Photo 3. Drill core samples from P05-01 laid out for logging.



Photo 4. Assembly of multi-level piezometer P01-05 (showing port #5).



Photo 5. Development of monitoring well MW-1 using Waterra hydrolift pump.



Photo 6. Development of multi-level well P05-01 using peristaltic pump.



Photo 7. DR24 Foremost Dual air rotary rig setting up at PW2.

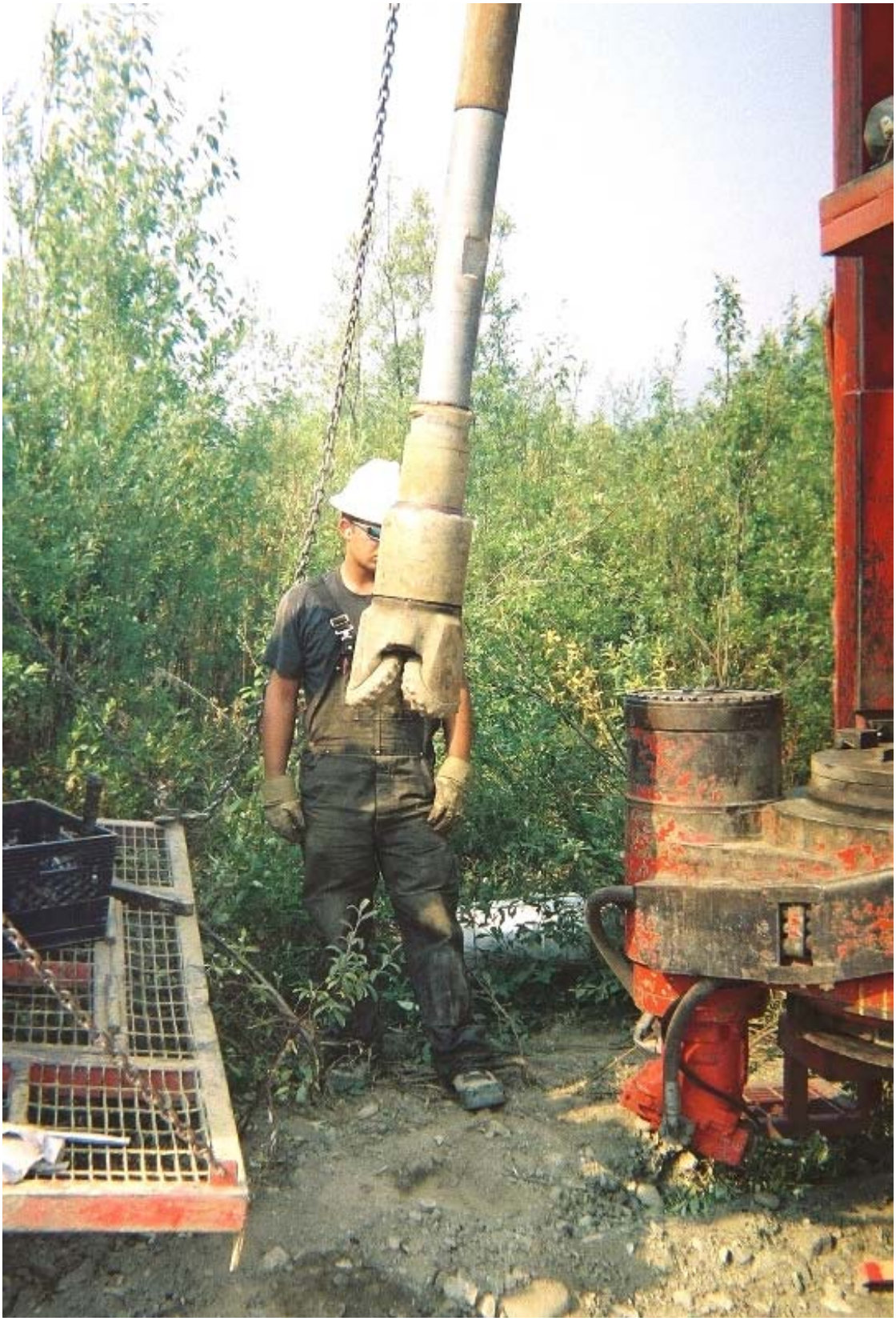


Photo 8. Tricone bit (9-7/8-inch) used for air rotary drilling.



Photo 9. Drill cuttings from PW2 laid for logging.



Photo 10. Welding of the 8-inch well screen assembly at PW2.



Photo 11. Installation of well screen assembly into 10-inch outer steel casing.



Photo 12. Jetting tool used for well development of pumping wells.



Photo 13. Submersible pump used for pump testing lowered into PW2.



Photo 14. Setup at well head for pump test at PW2.



Photo 15. Flow meter gauge at well head used for monitoring flow rate.



Photo 16. End of discharge line in seepage ditch downstream of X13.



Photo 17. Monitoring of pumping rate at end of discharge line.