



**RESPONSE TO IDAHO DEPARTMENT OF WATER RESOURCES MEMORANDUM:  
"Review of Hydrologic Data Analyses Submitted by RMEA  
in Support of Application for Transfer 80496"**

Bryce A. Contor  
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**Background**

On February 3, 2016, Sean Vincent, P.G., Hydrology Section Manager of Idaho Department of Water Resources (IDWR) provided a memorandum (Memo) to James Cefalo, Water Resource Program Manager of IDWR. The Memo was a review of technical analyses submitted by Rocky Mountain Environmental Associates, Inc. (RMEA) in support of application for transfer number 80496 (Report). The subject of the Report was a pumping test on the Boyd Smith Point of Diversion, which is a vertical-turbine with a constant-speed electric motor, set in an 18 foot deep excavated sump, 650 feet from a slough used as part of the distribution network of the Liberty Park Canal system.

The Memo also included policy findings and questions beyond the scope of the submitted Report.

At the invitation of Cefalo, and following a telephone conference with Vincent, RMEA has prepared this document (Response). The Response is organized as follows:

- Brief response to eight General Conclusions in the Memo
- Brief response to selected Specific Comments
- Brief response to policy findings of the Memo
- Brief description of the administrative background behind the test and transfer application
- Withdrawal of previously-submitted analyses
- Presentation of revised analyses
- Revised conclusions
- Figures
- Appendix A containing screenshots of Jenkins spreadsheet analyses
- Appendix B describing local surficial geology

**Brief Response to General Conclusions**

The IDWR General Conclusions are copied below in *italic text*, followed by RMEA responses in normal text.

- 1) *The existence of springs and a wetland in the upper reaches of Spring Slough is an indication that the slough is hydraulically connected to the shallow aquifer system.*

We agree that the slough is hydraulically connected to the shallow aquifer system.

- 2) *The degree and immediacy of the hydraulic connection between the proposed POD and Spring Slough is uncertain due to concerns about the design of the aquifer test, the construction of the test well, the quality of the test data, and irregularities and inconsistencies in the analysis and interpretation of the test data.*

We acknowledge that this real-world test does not adhere to every nuance of laboratory or text-book conditions. The brief description of the administrative history explains why the decision was made to not adhere to every formal requirement. Additional responses are provided below, to specific technical concerns.

We believe that the test and revised analyses document a substantial hydraulic connection between the Smith POD and Spring Slough.

- 3) *It appears that RMEA analyzed data from a different aquifer test that was of shorter duration than the one that was analyzed by Clearwater Geosciences. This introduces additional uncertainty because the shorter duration test was not documented in the submittals accompanying the transfer application.*

We apologize that the Report did not explicitly point out that both analyses were performed on different parts of the time series of data. We regret that this was not apparent from observation of the shapes of the data hydrographs.

- 4) *Irrespective of whether the proposed POD is in direct and immediate connection with Spring Slough, Spring Slough is not the original source and there does not appear to be an open water connection between Spring Slough and the original source. As a result, the issue of whether the proposed POD has direct and immediate hydraulic connection to Spring Slough arguably is immaterial to the transfer application.*

The Report was not a policy document and therefore did not detail the administrative history of why delivery of surface water to the slough was material, or why the transfer application was configured as it was. However, in concluding that "delivery of surface water or canal shares to the slough would fully mitigated for any extraction from the POD" it implicitly informed the

proposed mechanism for delivery of Liberty Park Canal water to the proposed POD.

At the request of Vincent, RMEA has walked the ditch that supplies surface water, from the point of injection to the lateral takeoff from the Liberty Park Canal, and mapped the GPS track points in Figure 14. Photos are provided in Figures 15-17.

- 5) *The submittals accompanying the subject transfer application do not include an evaluation of potential hydrologic impacts to other water right holders.*

Implicit in the Report was the assumption that since the POD extracts water from the slough, fully compensated by injections to it, there would be no hydrologic impacts to other right holders. We also point out that 2015 was the first year in more than two decades that the system had *not* operated as proposed by the transfer; the analyses were not a contemplation of the potential hydrologic effects from some new activity.

- 6) *Assuming the proposed POD is hydraulically connected to Spring Slough, delivery of canal water to the ditch that connects to the unnamed tributary would not necessarily fully mitigate for pumping because there are at least four on-slough diversions upstream of the confluence with the unnamed tributary.*

Figure 14 shows that one of the points of injection is upstream of the slough at its nearest point to the POD. Also, for more than two decades, the POD was used in its current configuration while the other noted pumps were used and not affected.

- 7) *Delivery of surface water from Liberty Park Canal to Spring Slough via any means other than the ditch that connects to the unnamed tributary would require constructing a conveyance that is longer than the conveyance that would be required to deliver water to the center pivot irrigation system directly from the canal. As such, establishing a permanent conveyance from Liberty Park Canal to the irrigation system should be considered as an alternative to mitigation for pumping using water from the canal.*

Figure 14 shows an existing conveyance that historically has been used to inject water upstream of the point on the slough that is nearest the Smith POD.

- 8) *The two PODs on Spring Slough that are nearest the Boyd Smith well do not have identification tags and are not in the IDWR spatial database. It is unclear whether there are water rights for these diversions.*

The Report was a technical rather than a policy document, and these pumps are

not subjects of the transfer application. However, it is RMEA's understanding from consultation with IDWR Eastern Region staff that the slough has been historically been used as an extension of the lateral system of the Liberty Park Canal system, and that the water rights and point of diversion for these pumps are the water rights and point of diversion of the Liberty Park Canal.

## **Brief Response to Selected Specific Comments**

Many of the Specific Comments relate to the Clearwater Geosciences analyses and report. We do not intend to rely on these moving forward, and so will not respond unless the Specific Comment also relates to the test itself or to the RMEA analyses. Specific Comments are not always reproduced in full below, but are generally abbreviated or paraphrased. Quotes or paraphrases from the Memo are in italic type and RMEA responses are in normal type. Underlining or lack thereof are preserved from the Memo.

### *Characterization of the Surface Source*

*None of the technical submittals... describe the nature and extent of the slough.... It does not appear... that the slough has an open water connection to the South Fork.... At least during the winter, the slough is not a substantial body of water in the vicinity of the POD....*

Figure 13 illustrates the geographic relationships between the Smith POD and other pertinent features. The visible surface expressions of the slough system arise in cultivated parcels south and east, and northwest, of the Smith POD. Two small branches of the slough combine just south of the Smith POD, into a larger slough that joins a northern branch approximately 7,500 feet in the north-northwest direction from the Smith POD. We agree that there appears to be no apparent open-water connection to the South Fork except for the mapped points where canal water is injected into the slough.

Two concentric circles, Radius 1 and Radius 2, are shown on Figure 14, centered on the Smith POD. Radius 1 is discussed below. Radius 2 is drawn at the distance of the irrigation or commercial groundwater point of diversion nearest to the Smith POD. Within Radius 2, we have traced the approximate course of the portions of the slough that are nearer to the Smith POD than any irrigation or commercial POD. The combined linear length of the mapped slough is calculated within a Geographic Information Systems project as 8,425 feet.

We agree with the IDWR characterization in the General Comments that the sloughs intercept groundwater, and point out that users have historically injected surface water into the slough system, to utilize it as a conveyance for Liberty Park Canal water. Two such injections are shown on Figure 13. During the period of proposed use, the slough

does contain water, and injections at the mapped points are adjusted to maintain adequate water levels for all re-diversions.

### *Aquifer Testing and Data Analysis*

*...the design of the Boyd Smith well test provides for uncertain hydraulic property estimates and precludes definitive conclusions about the hydraulic connectedness between the proposed POD and the slough. The uncertainties relate to three elements of the test design:*

We agree that this was less robust than a textbook example pump test, for reasons outlined in the brief outline of the administrative background later in the Response. We believe the uncertainty introduced by the departures does not preclude a determination of whether the POD has a substantial hydraulic connection to the slough.

#### *1) No provisions were made for measuring the flow rate during the test...*

The piping configuration at the pump head was not conducive to accurate flow measurements, and reconfiguration was cost prohibitive. Flow was measured at the pivot point for the southern center pivot, and near the pivot point for the larger northern center pivot. The northern flow measurement was upstream of a manifold which distributes water to both northern pivots and to stationary sprinklers on the north end of the place of use. These two measurements captured nearly all the discharge at the time of measurement, with approximately 100 gallons per minute added to the measured total based on a count of hand line sprinkler nozzles that were not captured by the two measurement points.

The test personnel (RMEA) and the system operator (Smith farm) understood the importance of maintaining constant discharge, to the extent possible, during the pumping test. The operator wrestled with the practicality of operating a complex system that includes two center pivots which overlap, requiring periodic shut-off of the overlapping section of one of the pivots. Physical obstructions prevent the operation of any of the pivots in a full-circle pattern, so all three must be periodically stopped and reversed in dry mode.

The Smith POD is fitted with a constant-speed vertical turbine pump. The physics of such a pump are that within a short enough period of time that impeller wear is insignificant, discharge will be constant if total dynamic head is constant. The three components of total dynamic head are pumping lift, column friction, and discharge pressure at the pump. Pumping lift is a function of aquifer properties, static head, and pumping rate. Column friction is assumed to be relatively constant over a narrow range of operating flows. On this system, discharge pressure is indirectly controllable by the operator, by adjusting the number of

nozzles operating at a given time.

The test relied upon the operator's experience from measurement of pumping levels prior to the 2015 hiatus, that drawdown quickly stabilizes and remains essentially constant throughout the irrigation season. Data presented in Figure 1 show that the operator's expectation was met for the duration of the test.

It is acknowledged that some uncertainty could be introduced from changes in pumping water level. Because equations for estimating transmissivity are linear relative to discharge rate, there is no amplification of flow-rate imprecision by exponential terms. The potential magnitude of the effects of changing pumping levels can be estimated by considering approximate total dynamic head (TDH), calculated as:

$$\begin{aligned}\text{TDH} &= [11 \text{ feet pumping lift} + 5 \text{ feet friction} + (72 \text{ psi} \times 2.31 \text{ ft/psi})] \\ &= 182 \text{ ft}\end{aligned}$$

If pumping lift were to vary by four feet, this would represent (4/182) or approximately 2% variation in TDH. While a pump curve was not available for this pump, within such a narrow range of TDH, discharge response would be expected to be generally proportional, and hence represent approximately a 1% to 3% uncertainty in flow rate in response to pumping-lift variation.

Greater uncertainty arises in the imprecision of adjusting a finite number of laterals of finite capacity, in order to maintain constant discharge pressure. Periods of different discharge are apparent in the record as periods of drawdown differing from the general linear hydrographs. Also, an attempt was made at a stepped-rate pumping test, so for an initial period, the flow rate was less than half the typical rate during the balance of the test.

The revised pumping-period analyses were applied to a brief period of time where the hydrographs appeared to show minimal disturbances, and the sprinkler configuration was known. There is good confidence in the flow rate used for these tests.

The revised recovery-period analyses are not affected by perturbations of discharge rate, but the average flow rate during the entire pumping period (required for recovery analysis) is less well estimated. Power consumption coefficients were measured during the initial lower-flow portion of the test, and a later higher-flow portion. The TDH and hence the power consumption coefficients for the two conditions differed significantly. A time-weighted average power consumption coefficient was estimated for the pumping period, using these two measured power consumption coefficients. The estimated average coefficient was applied to power-meter readings to estimate average discharge

over the pumping period, conservatively bracketed at +/- 25%. This implies an imprecision in those transmissivity estimates of approximately 25%, though this may be an over-estimate, due to the conservative subjective assignment of imprecision in the weighted-average power consumption coefficient.

*2) The discharge was run through a sprinkler system rather than piped away from the test site....*

This decision was made to control costs, and to avoid the necessary delay of constructing temporary piping to convey water beyond the spatial extent of the irrigation system. During the initial part of the test, to which the revised pumping-period analyses were applied, the pivot nearer the POD was operated in the northern half of its coverage area. The circle labeled Radius 1 in Figure 13<sup>1</sup> is set at the distance from the POD to the pivot point. Approximately 3,115 linear feet of the slough are within Radius 1, nearer to the POD than any recharge from the pivot. Signals from interception of the slough would be expected to exceed in magnitude any signals from percolation, and be expressed earlier in time. Further, the entire parcel was in alfalfa hay and was in a dry condition immediately following harvest. To reach the water table, percolation from the pivot would have to first fill a largely-depleted root zone of a deep-rooted crop. Approximating the parcel size at 300 acres, the 55 to 72 acre feet pumped would have produced an application depth of 0.18 to 0.24 feet, or approximately 2.2 to 2.9 inches of application depth. Using a rule of thumb of one foot saturation per inch of water, for loam or sandy loam soils, it is expected that the root zone would have accommodated nearly all applied water, given that depths to water at the beginning of the test were 2.1 to 2.5 feet below land surface.

It is acknowledged that later in the test, the southern pivot was operated near the POD. Any percolation that may have occurred below the root zone could have affected later-time drawdown and the recovery analyses.

*3) The test was conducted during the irrigation season....*

This decision was made to allow timely submission of the test results.

Radius 2 in Figure 13 is set at the distance of the nearest groundwater point of diversion large enough to potentially perturb the pumping test, an irrigation well not under the control of Smith. Approximately 8,425 linear feet of the slough are in nearer proximity to the POD than this nearby well. RMEA believes the influence of the slough would overwhelm any influence from the well in magnitude, and precede it in time. If there is an effect on the data from the nearby well, it would affect the revised recovery-period analyses more than it

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<sup>1</sup> The representation of the Liberty Park Canal in Figure 13 and Figure 14 was hand-digitized from a scanned image of a map in the Memo and is subject to some imprecision.

would the revised pumping-period analyses, which were applied to early-time data.

The parcel immediately south of the POD was in grain stubble and was not irrigated during the test. Other irrigated parcels are outside Radius 2 and more distant than the 8,425 linear feet of slough within Radius 2. Hence, their effects would be expected to be delayed in time and reduced in magnitude relative to the effects of interest.

The deficiencies are acknowledged, and defended in the context of a real-world test constrained by cost and timeliness considerations. These deficiencies are more likely to affect the revised recovery-period analyses than the revised pumping-period analyses, due to the greater period of time available for propagation of confounding signals.

#### *Site Evaluation*

*It appears based upon my review that most of Stallman's pre-test site evaluation criteria were not satisfied and important details... either were not documented or were not verified.... the open intervals for the test well were not documented and this information is likely not known... since no well driller's logs were provided...*

Appendix B contains a geologist's review of the site geology.

The POD sump (referred to as "well" in the Memo) does not meet the legal definition of a well and was not installed by a well driller; hence, there are no logs. However, the key information to be gained from logs in this context would be information informing the expected productivity of the aquifer and the efficiency of the POD. This 18-foot deep POD was known to be very productive. Further, the revised analyses omit consideration of pumping-POD data during pump operation, avoiding the need to estimate or adjust for well-efficiency factors.

#### *Test Execution*

*...inspection of Figure 6 from [the Clearwater] report reveals that there most likely were at least two significant pump outages, the first beginning approximately 20 minutes after the test started and lasting for approximately 45 minutes... the test was not restarted after the first outage.... Fluctuations in the pumping rate reduce the quality of the water level versus time data.*

There were two pump outages. As reported above, an attempt was made to obtain stepped-rate pumping data. This failed when approximately 20 minutes into the test, the system over-pressure switch stopped the pump. Following the rule of thumb of a recovery period exceeding the pumping period, the test was restarted approximately 45 minutes later, at a greater discharge rate allowing system pressures to be in the normal

operating range. The early time data were not used in the original RMEA analyses and are not used in the revised pumping-period analyses presented below. Early time volumes and elapsed times were included in the calculation of average discharge rate for evaluation of recovery data, however.

The second pump outage was due to a regional power-system failure. It occurred after the time period used for the revised pumping-period analyses, and does not affect them. The instructions RMEA provided to the operator apparently were not adequately clear, and the pump was restarted too soon following restoration of power. Because recovery after the second period did not meet the rule of thumb, no revised pumping-period analyses were performed on later data. Revised recovery-period analyses rely upon total pumping-period volume and not instantaneous rate.

In addition, the operator reported that there were heavy rains toward the end of the recovery period. Combined with a root zone already filled by irrigation, these potentially could have entered the aquifer and affected aquifer water levels.

Other variations in flow rate are discussed above.

#### *Documentation*

The Memo raises a number of criticisms, most of which apply to the analyses which are withdrawn by this response. This Response constitutes documentation of the test and the revised analyses.

#### *Hydrogeologic Conceptual Model*

This section also raises questions mostly applicable to the withdrawn analyses. The geologist's review attached as Appendix B documents that within the depth of the slough and the POD, the system is believed to be primarily unconsolidated coarse alluvial sediments. As the POD sump, piezometers and the slough all penetrate the aquifer to approximately equal depths, all are assumed to fully penetrate the portion of aquifer active in lateral flow to the POD. Given the productivity of the POD, the nature of the geologic materials and the large surface area of interconnection between the aquifer and the slough, the implied assumption of predominantly horizontal flow is reasonable.

#### *Initial Data Processing*

These comments address data processing for the withdrawn Clearwater analyses. The Clearwater adjustments were not applied prior to performing the revised analyses presented with this Response.

#### *Selection of Data for Analysis*

The period of data prior to the power outage were selected for revised pumping-period analysis for the following reasons:

- These are not affected by the shorter-than-standard recovery period from the second pump outage.
- The pumping rate is known for this period.
- These data are least likely to be affected by any percolation that may have occurred below the root zone.
- These data are least likely to be affected by any perturbation from the nearby irrigation well or from nearby incidental recharge from irrigation.

The data from the pumping POD were omitted from pumping-period analyses to avoid the confounding effects of well efficiency.

The recovery period data were analyzed because these are free from temporal effects of the two pump outages and the minor variations in flow implicit in the drawdown hydrographs.

*The calculated recovery method (Brown, 1953) requires that the measured recovery data be adjusted.... It could be argued that the drawdown had stabilized at the end of the pumping period [however there] are three other possible explanations for near steady-state conditions which are not discussed...:*

- 1) There likely was a decline in the discharge rate as a result of increased total dynamic head...*
- 2) The pumped water was discharged onto the ground in the vicinity of the pumping well...*
- 3) The occurrence of incidental recharge on adjacent irrigated lands also may have contributed to near steady state conditions...*

These concerns are addressed above under "Aquifer Testing and Data Analysis."

*When there are uncertainties about the pumping period water level trends, application of the Theis (1935) residual drawdown recovery method is more straightforward and produces a more reliable transmissivity estimate...*

#### *Assessment of Water Level Recovery*

#### *Selection of Analytical Solutions*

#### *Hydraulic Property Estimation*

The discussions of these sections apply mostly to the withdrawn analyses and no response is provided. We have used the Cooper-Jacob<sup>2</sup> straight line method, following the widely-used conventions of applying it to both pumping and recovery data, and to unconfined systems even though the mathematics properly apply to confined systems.

We acknowledge that consideration of the  $u < 0.05$  criterion was improperly neglected in the RMEA distance-to-boundary evaluations and disqualifies the analyses for the two piezometers more distant from the POD (P50 and P100). These are withdrawn, but the P20 piezometer analysis meets the criterion and is retained. It follows the figures generated for this Response.

### *Transmissivity – Uncertainty of Estimates*

The adjustments described above provide two sets of transmissivity estimates, with results presented below. The estimates from the revised pumping-period analyses are most reliable, due to better knowledge of the pumping rate, and more temporal isolation from distant potential confounding influences. Both these and the revised recovery-period analyses produce results very similar to those obtained by Vincent as reported in the Memo.

### *Storage*

The Clearwater storage coefficient estimates are withdrawn.

Storage coefficients calculated using the Cooper-Jacob method on pumping period data ranged from unrealistically low to unrealistically high, so RMEA instead relied upon textbook storage coefficient values which provided a narrower range of estimates, reported below.

### *RMEA Stream Depletion Analysis*

These analyses were repeated using the maximum, minimum, and median transmissivity values from Vincent's two estimates, the three revised pumping-period analyses, and the four recovery-period analyses. A range of textbook storage coefficients were used. Screenshots of the analyses are provided in Appendix A and the results are reported below.

### *RMEA Hydraulic Boundary Analysis*

*There are several internal inconsistencies in the RMEA boundary analysis:*

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<sup>2</sup> Cooper, H.H. and C.E. Jacob, 1946. A generalized graphical method for evaluating formation constants and summarizing well field history. Trans. Am. Geophys. Union, v. 27, p. 526-534

- 1) *The boundary analysis for P20 includes drawdown... from one to approximately 10,000 minutes.... The elapsed time on the P50 and P100 boundary analysis plots is only 800 minutes...*

As described above, due to the  $u$  criterion, only the P20 data are suitable for this analysis and the other analyses are withdrawn from consideration.

- 2) *The boundary analysis plot for P20 shows the same response to the suspected pump outage between 20 minutes and 65 minutes... as observed in all four wells on Figure 6 of the Clearwater Report. Without explanation, however, complete water level recovery between 20 minutes and 65 minutes is not seen on the boundary analysis plots for P50 and P100.*

The times for the P50 and P100 plots were adjusted to reflect the restart of the test, while the P20 plot and the Clearwater plots were not. However, the P50 and P100 plots are withdrawn from analysis.

- 3) *The RMEA analysis interpreted a break/decrease in slope at approximately 900 minutes in the P20 plot... and between 10 and 20 minutes on both the P50 and P100 plots...*

This is an artifact of violation of the  $u$  criterion. Upon further reflection, RMEA believes that the change in slope in the P20 plot is reflective of an encountered boundary, while the changes in slope in the other two plots represent the transition to a log-linear relationship when the  $u$  criterion was met. The P50 and P100 analyses are withdrawn.

#### *Inconsistencies between the Hydraulic Boundary and Aquifer Test Data Analyses*

The Clearwater analysis, the P50 boundary analysis and the P100 boundary analysis are withdrawn.

- *Whereas RMEA chose to analyze the pumping period data for the boundary analysis, the Clearwater analyst chose to analyze the water level recovery data because "The shut down and variation in pumping rate early in the test (day 1) are the most problematic for aquifer analysis."*

The revised RMEA Cooper-Jacob analyses rely on the data period after adequate recovery from the first pumping period and restarting the test, and before the second pump shutdown.

The RMEA boundary analysis does use data spanning the second shutdown period. The timing of the unplanned power outage was unfortunate, and it is a reason to rely more heavily on other analyses. RMEA believes that it still provides

insight and adds to understanding of the system, rather than detracting.

- *...individual data points are not discernable on the boundary analysis plots prepared by RMEA.*

Many of the revised figures include individual data points. For precision in analysis, some of the revised analyses still rely on plots prepared without the data points, because the time series traces are more distinct without the individual points. The P20 boundary analysis graphic was not reproduced with individual data points but Figure 10 shows the same data period plotted with the points.

- *The boundary analysis plots for both P20 and P50 indicate that the drawdown at an elapsed time of one minute was 0.00 feet. In contrast, Figure 6 in the Clearwater Report indicates that there was measureable drawdown in both of those piezometers at one minute.*

This may be an artifact of the data adjustments rejected earlier in the Memo, or an artifact of which of the non-pumping measurements were selected as the zero point. The Clearwater analyses are withdrawn. As the revised analyses rely upon differences in drawdown, the selection of datum is arbitrary and irrelevant.

- *Whereas the boundary analysis plot for P20 indicates that the maximum drawdown was approximately 1.2 feet, Figure 5 in the Clearwater report indicates that the maximum drawdown in P20 was approximately 3.6 feet.*

The vertical axis of the P20 plot does not include the full range of data. Data adjustment, selection of time period, and datum also may partly explain the difference. The Clearwater report is withdrawn.

- *As previously mentioned, complete water level recovery between 20 minutes and 65 minutes is not seen on... the plots for P50 and P100...*

As explained above, the P50 and P100 plots used a time scale relative to the restart of the test, and are withdrawn.

- a. Whereas the aquifer test documented in the Clearwater report involved 7 days of pumping, the... time for... P50 and P100 is only 800 minutes.*

As explained above, the RMEA analyses treated part of the time series of data that were considered by Clearwater.

- b. Comparison of the boundary analysis lots for P20 and P50... indicates greater drawdown in P50 after only 80 minutes of pumping... than*

*measured in P20 after seven days of pumping.*

This is likely an artifact of the adjustment to time series and perhaps to selection of zero datum for calculation of drawdown. The P50 boundary analysis is withdrawn.

*c. ...there is no mention of transducer removal and replacement in P100.*

All the transducers were removed and replaced at least twice. Additionally, the caps of the piezometers (with attached suspension lines) were lifted slightly and set aside for brief periods during two leveling surveys,<sup>3</sup> which could have introduced a few tenths of a foot of discrepancy for a few data points.

- *Whereas the RMEA analyst interpreted that there was a slope change indicative of a recharge boundary less than 20 minutes into the pumping period on both the P50 and P100 drawdown plots, the Clearwater report identified breaks... 700 minutes after pumping began... and 1,400 minutes after recovery began...*

As discussed above, RMEA believes that the apparent breaks in slope it identified in the P50 and P100 data were artifacts of violating the  $u$  criterion, and in fact represent the transition to a log-linear relationship expected at later times.

#### *Application of the Straight Line Method*

*The truncated infinite series approximation is only valid after sufficient time has elapsed for the so-called "u-criterion" to be met...*

We acknowledge the failure to consider the  $u$  criterion and for this reason we withdraw the P50 and P100 boundary analyses. The P20 analysis is retained as it meets the criterion. The criterion is met for the revised analyses, as can be seen by the transition to log-linear relationships before the data periods to which the analysis lines are fit.

#### **Brief Response to Policy Findings of the Memo**

The Memo also addresses policy issues, paraphrased by RMEA in italics, with responses in regular type:

*Is a water-right transfer of point of diversion the appropriate vehicle to describe the physical condition of injection of water into the slough, to replace water withdrawn from the slough by operating the POD?*

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<sup>3</sup> The leveling survey data were not used.

RMEA shares Vincent's concern. A point of diversion transfer was selected as perhaps the best selection from a limited suite of available administrative mechanisms. We invite recommendations for a better administrative approach.

*Is there a physical mechanism to deliver the applicant's surface water from shares held in Liberty Park Canal, to the slough from which the re-diversion occurs?*

RMEA personnel walked the ditch from near the point of injection, to the lateral headgate from the Liberty Park Canal. The path of the lateral is mapped in Figure 14, with photos of the infrastructure shown in Figures 15 through 17.

*What is the water-right status of pumps observed in the slough, without IDWR identification tags?*

From consultation with IDWR regional staff, long before preparing the transfer application, RMEA concluded that the slough is historically used as a lateral extension of the Liberty Park Canal and that these pumps administratively are the same as ditch pumps internal to a canal system's laterals.

### **Brief Description of Administrative Background**

The Smith POD was constructed by the applicant under the 42-228 "recovery well" statute more than 20 years ago, under guidance of competent legal counsel and in compliance with the then-current meaning of the statute. The pump was used every year through 2014, with injection of water into the slough system to maintain water levels in the slough. The entire place of use is within the Liberty Park Canal service area, and the transfer applicant owns more than enough shares in the canal to fully service this and all his other canal-company lands.

Prior to the 2015 irrigation season, a Notice of Violation was filed against the applicant, with the only change since development of the POD being reinterpretation of the statute. At short notice and significant expense, the applicant purchased a diesel pump and portable mainline, setting up a temporary pumping station in a ditch. Except for the pumping test, this diesel pump was used exclusively in 2015, at considerably greater expense and inconvenience than the electrical pump used previously at the Smith POD. Because of the short notice, and cost constraints of new pumps and new impellers, the used pump obtained was not properly matched to the requirements of the system. This reduced pumping capacity from historical enjoyment and added further inconvenience and cost to the 2015 farming activities.

In consultation with the regional office of IDWR, and with knowledge of the historical practice of injection at both points of injection documented in Figure 13, RMEA explored the hydraulic connection of the Smith POD to the slough immediately to the south of the county road, using aquifer parameters from ESPAM2.1. The positive results

encouraged the preparation of the transfer application, the pump test, and the documentation report.

Because of the great expense already expended by the applicant, in the face of a long history of good-faith compliance, RMEA attempted to minimize the cost of the pumping test. This was under an assumption that IDWR was fully aware of the history of the Smith POD, the status of the applicant's shares in the Liberty Park Canal system, the historical practice of injecting necessary quantities of water to maintain slough water levels, and the selected transfer mechanism. The departures from text-book ideals were judged to not be severe enough to preclude testing of the hypothesis of a substantial hydraulic connection between the Smith POD and the nearby slough.

From the limited suite of administrative tools available, RMEA selected a water-right transfer of point of diversion to represent the physical reality of the injection and re-diversion process. The effort was not an attempt to establish any new use or new stress upon the water-supply system, but to simply resume an activity undertaken in good faith, lawfully and under guidance of legal counsel, which had operated for more than two decades.

### **Withdrawal of Analyses**

We agree that additional analysis is in order. We hereby withdraw the Clearwater analyses, and all previously-submitted RMEA analyses except the P20 boundary analysis.

### **Revised Analyses**

A geologic summary was prepared and is attached as Appendix B. The upper 20 feet of the local shallow aquifer may include a few feet of loamy silt or sandy topsoil, underlain by sand and gravel. One well log suggests the possibility of clay in the topsoil, and one suggests the possibility of gravel and cobble rock. From informal observation during the test, RMEA believes there is no significant clay fraction in the topsoil at this location.

The 2016 irrigation season is fast approaching, and the applicant is anxious to avoid the significant cost, inconvenience and inferior capacity of continued use of the temporary diesel pump system. Time is of the essence, and Dr. Wood is constrained by a busy teaching, research and administrative load at University of Idaho and unable to repeat the Clearwater analyses in a timely manner. For this reason, the computerized analyses performed by Clearwater will not be repeated. Instead, RMEA has conducted Cooper-Jacob<sup>4</sup> straight-line analyses, conducted per procedures outlined by Domenico and Schwartz.<sup>5</sup> Pumping-period analyses were performed on data from all three

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<sup>4</sup> Cited earlier in the Response

<sup>5</sup> P.A. Domenico and F.W. Schwartz. 1998. Physical and Chemical Hydrogeology. John Wiley and Sons, New York. p. 108.

piezometers, for an early pumping period when pumping discharge rate was known and before the unfortunate regional power outage that interrupted the test. Pumping-period analysis was not performed on the POD sump data, to avoid the confounding influence of well-efficiency factors. Recovery-period analyses were performed on data from all three piezometers and from the POD sump. Copies of the graphical analyses are attached, and calculations of transmissivity are summarized in Table 1. IDWR estimates from the Memo are included as "Vincent" entries. Note that the change in drawdown ( $\Delta s$ ) was not calculated across the same log-time cycle for all tests,<sup>6</sup> so individual drawdowns are not comparable from test to test. The pumping-test drawdowns were extracted visually to two significant figures, while the recovery-period drawdowns were extracted digitally from the log-linear plot to six significant figures, rounded to three figures for Table 1.

Table 1. Summary of Transmissivity Calculations

| Test             | s <sub>1</sub> feet | s <sub>2</sub> feet | $\Delta s$ feet | Q gpm,<br>ft <sup>3</sup> /day | T ft <sup>2</sup> /day<br>(2.3 Q/4 pi<br>$\Delta s$ ) |
|------------------|---------------------|---------------------|-----------------|--------------------------------|-------------------------------------------------------|
| P20<br>Pumping   | 0.82                | 1.17                | 0.35            | 1988/382,691                   | 200,124                                               |
| P50<br>Pumping   | 1.76                | 2.04                | 0.28            | 1988/382,691                   | 250,154                                               |
| P100<br>Pumping  | 1.005               | 1.305               | 0.30            | 1988/382,691                   | 233,478                                               |
| Pump<br>Recovery | 0.476               | 0.9155              | 0.440           | 2150/413,876                   | 172,285                                               |
| P20<br>Recovery  | 0.544               | 0.997               | 0.453           | 2150/413,876                   | 167,226                                               |
| P50<br>Recovery  | 0.503               | 0.989               | 0.487           | 2150/413,876                   | 155,583                                               |
| P100<br>Recovery | 0.545               | 0.998               | 0.454           | 2150/413,876                   | 166,968                                               |
| Vincent<br>Low   |                     |                     |                 |                                | 170,000                                               |
| Vincent<br>High  |                     |                     |                 |                                | 220,000                                               |

From the values in Table 1, the maximum, minimum and median estimated transmissivity were tabulated at 250,154 ft<sup>2</sup>/day, 155,583 ft<sup>2</sup>/day and 172,285 ft<sup>2</sup>/day, respectively. Given the wide range of transmissivities that can occur in geologic materials, the vagaries of graphical analysis, and the admitted challenges affecting the

<sup>6</sup> Since this is a linear analysis, selection of which log-time cycle to use is simply a matter of convenience and has no effect upon results.

test and the data, RMEA considers it a positive diagnostic that the largest estimated transmissivity is only 1.6 times the smallest estimated value. It is also a positive diagnostic that the Vincent estimates lie within the bounds of the RMEA estimates.

As discussed above, RMEA attempted Cooper-Jacob analysis of storage coefficient, but obtained results that ranged from unreasonably low to unreasonably high. These were rejected and RMEA used textbook estimates from Fetter,<sup>7</sup> listed in Table 2. In addition to the tabled values, a coefficient of 0.10 was used in an attempt to replicate a reduction in porosity expected from potential poorly-sorted conditions, where finer grained particles occupy voids between larger grain sizes. The geology report (Appendix B) indicates a high-energy depositional environment during the late pleistocene, which commonly results in highly-permeable sediments. However, three of the four well logs attached by the geologist indicate mixed textures in the upper intervals of the wells, so the conservative storage coefficient of 0.10 was added to the calculation matrix.

Table 2. Storage Coefficient Estimates

| <b>Coefficient</b> | <b>Source</b> | <b>Materials Represented</b>                                |
|--------------------|---------------|-------------------------------------------------------------|
| 0.10               | RMEA          | Poorly-sorted gravels, minimum <sup>8</sup>                 |
| 0.12               | Fetter        | Coarse gravel, minimum                                      |
| 0.22               | Fetter        | Coarse gravel, average                                      |
| 0.26               | Fetter        | Coarse or medium gravel, maximum; also medium sand, average |
| 0.35               | Fetter        | Fine gravel, coarse sand or medium sand, maximum            |

The full matrix of five storage coefficients and three representative transmissivity values (maximum, minimum and median) were applied to the Jenkins<sup>9</sup> method spreadsheet<sup>10</sup> to estimate stream depletion at 24 hours. The median pumping-period transmissivity of 233,478 ft<sup>2</sup>/day and the median storage coefficient of 0.22 were also applied to the spreadsheet as the most likely values. The mean of the Vincent estimates (195,000 ft<sup>2</sup>/day) and the median storage coefficient were also applied. Screenshots of analyses are presented in Appendix A, and results are summarized in Table 3. Values greater than or equal to 50% are in italic type. The most probable value from the RMEA

<sup>7</sup> C.W. Fetter. 1994. Applied Hydrogeology. Third Edition. Prentice-Hall, New Jersey. p. 91

<sup>8</sup> This is also consistent with Fetter's estimate for "Fine sand, minimum."

<sup>9</sup> C.T. Jenkins. 1968. *Computation of Rate and Volume of Stream Depletion by Wells*. Chapter D1, Book 4. Techniques of Water-Resources Investigations of the United States Geological Survey.

<sup>10</sup> B. A. Contor. 2011. Adaptation of the Glover/Balmer/Jenkins Analytical Stream-Depletion Methods for No-flow and Recharge Boundaries. Idaho Water Resources Research Institute Technical Completion Report 201101.

analyses, and the most probable value using the Vincent estimates, are in bold, red type.

Table 3. Jenkins Depletion Percentage (q/Q) at 24 Hours

| Transmissivity<br>ft <sup>2</sup> /day            | S = 0.10 | S = 0.12 | S = 0.22 | S = 0.26 | S = 0.35 |
|---------------------------------------------------|----------|----------|----------|----------|----------|
| 250,154 (max.)                                    | 69%      | 66%      | 55%      | 52%      | 45%      |
| 233,478<br>(median of<br>pumping-period<br>tests) |          |          | 54%      |          |          |
| 195,000 (mean<br>of Vincent<br>estimates)         |          |          | 50%      |          |          |
| 172,285<br>(median)                               | 63%      | 60%      | 47%      | 44%      | 37%      |
| 155,583 (min.)                                    | 61%      | 58%      | 45%      | 41%      | 34%      |

The P20 boundary-distance analysis is retained and provided following Figure 17. Acknowledging the concerns with its data, we note that it does indicate interception of a recharge boundary at a distance compatible with the physical distance from the Smith POD to the slough. It indicates interception of a recharge boundary at a distance of approximately 670 feet, after 1,000 minutes or approximately 17 hours.

The recovery-period data also suggest the encounter of a recharge boundary. In later time, the hydrographs all depart from the log-linear trend expected if recovery were only from spatial redistribution of water stored in the aquifer. Visually-apparent departure points are noted with red arrows in Figures 2, 5, 7 and 9. These suggest intersection of a recharge boundary possibly within 600 minutes (10 hours) but certainly within 1,000 to 1,300 minutes (17 to 22 hours).

These distances and times to apparent recharge boundaries are not presented as primary evidence, but they are compatible with the other analyses and serve as additional indication of substantial hydraulic connection between the Smith POD and a recharge boundary at the approximate distance of the slough (650 feet), within a time period less than 24 hours.

### Revised Conclusions

We agree with Vincent that the proper policy standard is that *"diversion and use of water from the proposed point of diversion would have substantially the same effect on the hydraulically-connected source as diversion and use of water from the original point of diversion."* We acknowledge that IDWR has adopted a guideline to identify this

condition as being met when *"at least 50 percent depletion in original source [results from] from depletion at proposed point of diversion in one day."*

We acknowledge that recovery-period data are generally to be preferred over pumping period data. We also acknowledge that in general, a longer test period is to be preferred over a shorter period. However, the narrative above indicates that in this case, the pumping-period analyses from early in the test are preferred over the recovery-period results for the following reasons:

- If there were effects from percolation of applied water to the aquifer, these would have occurred later in the test and have greater influence on the recovery data;
- If there were effects from nearby well pumping, these would have had more time to propagate into the test area and have greater influence upon the recovery data;
- If there were effects from nearby irrigation, these would have had more time to propagate into the test area and therefore would have greater influence upon the recovery data;
- The recovery data may have been affected by heavy rains toward the end of the recovery period;
- The average pumping rate for the pumping-period test is much better known than the average pumping rate for the duration of the test;
- The data used for pumping-period analyses precede the power outage, and do not appear to exhibit large numbers of changes in discharge rate.

The most likely transmissivity estimates, combined with the most likely storage coefficient estimate, produce calculated 24-hour depletions that meet the 50% criterion. This is compatible with secondary analyses indicating interception of a recharge boundary at a distance of approximately 670 feet, within 10 to 22 hours.

We acknowledge that these transmissivity and storage coefficient estimates produce a distribution that does extend somewhat below the criterion, but we note that the center of mass of the distribution, and therefore the most probable value, is above the criterion. While the Report, the Memo and this Response should be technical and not policy documents, we propose that the test indicates that the proposed transfer does meet the policy requirement that *"diversion and use of water from the proposed point of diversion would have substantially the same effect on the hydraulically-connected source as diversion and use of water from the original point of diversion,"* even if perhaps the results are near the cusp of the criterion of 50% in 24 hours.

It is true that a subset of the results could be selected that would not reach this standard. However, the context for this particular transfer application includes not only the opportunity for any party to protest the transfer, but a history of more than two decades of pumping at the proposed condition. This gives any potentially injured party full opportunity to understand the implications of the injection and re-diversion. In this

particular context, it is the professional opinion of RMEA that the hydrogeologic setting, the ESPAM2.1 aquifer model calibrated properties, and the pump test all confirm that pumping from the Smith POD has substantially the same effect as pumping from the slough.

Figure 1. Raw Data - Full Time Series - Smith Pump Test

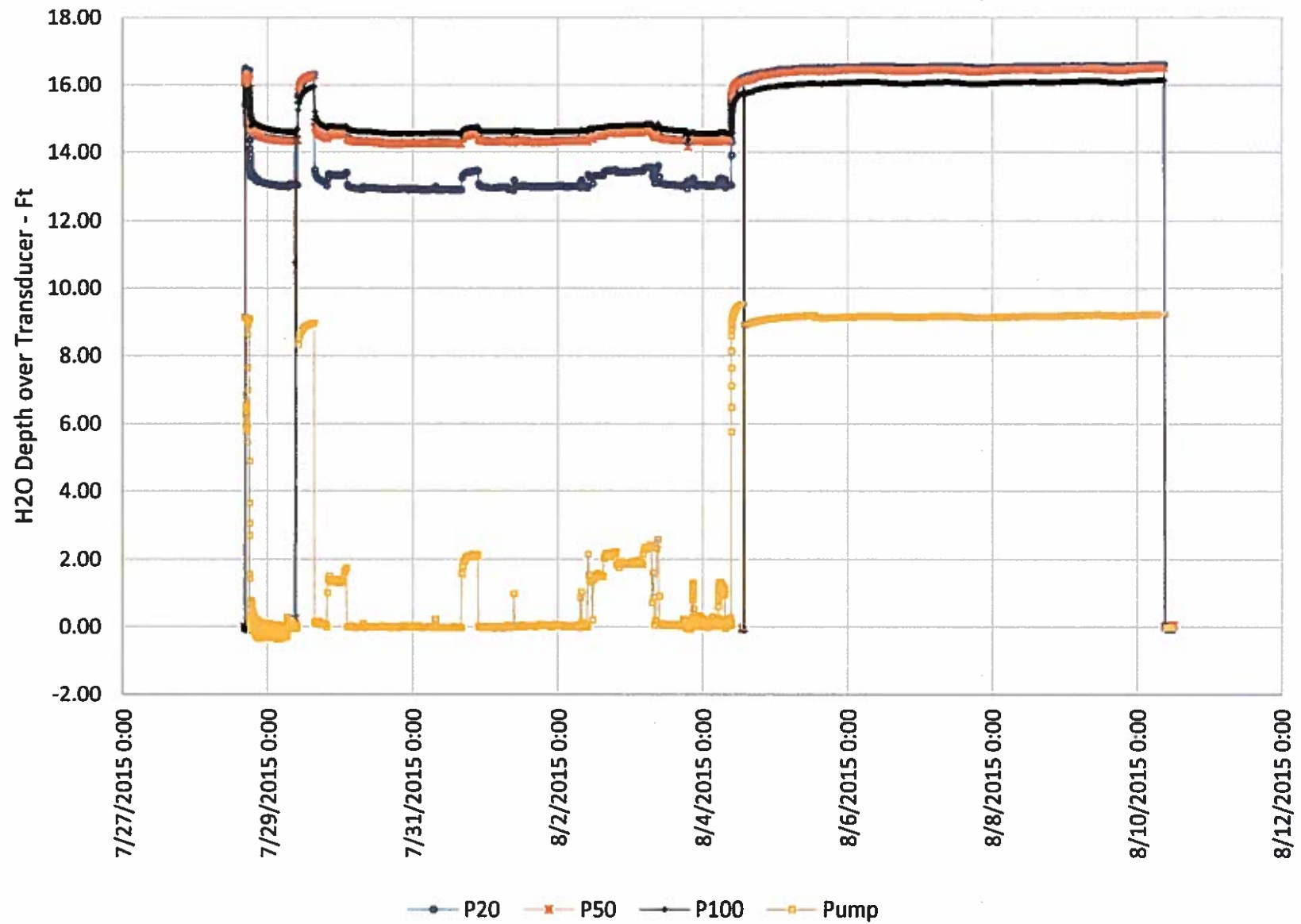


Figure 2. Pumped Well Recovery Data

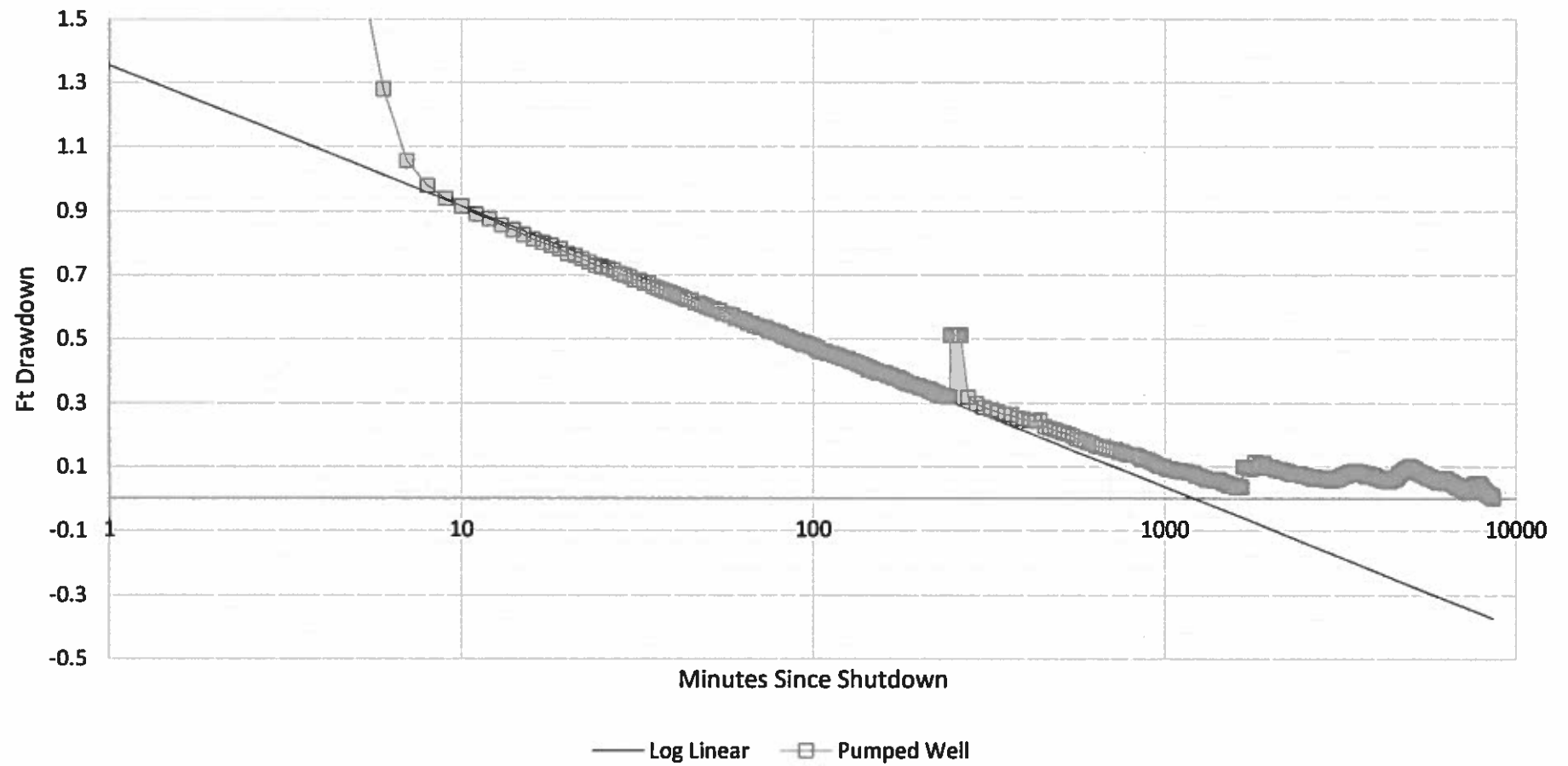


Figure 3. Pumped Well Recovery Graphical Analysis

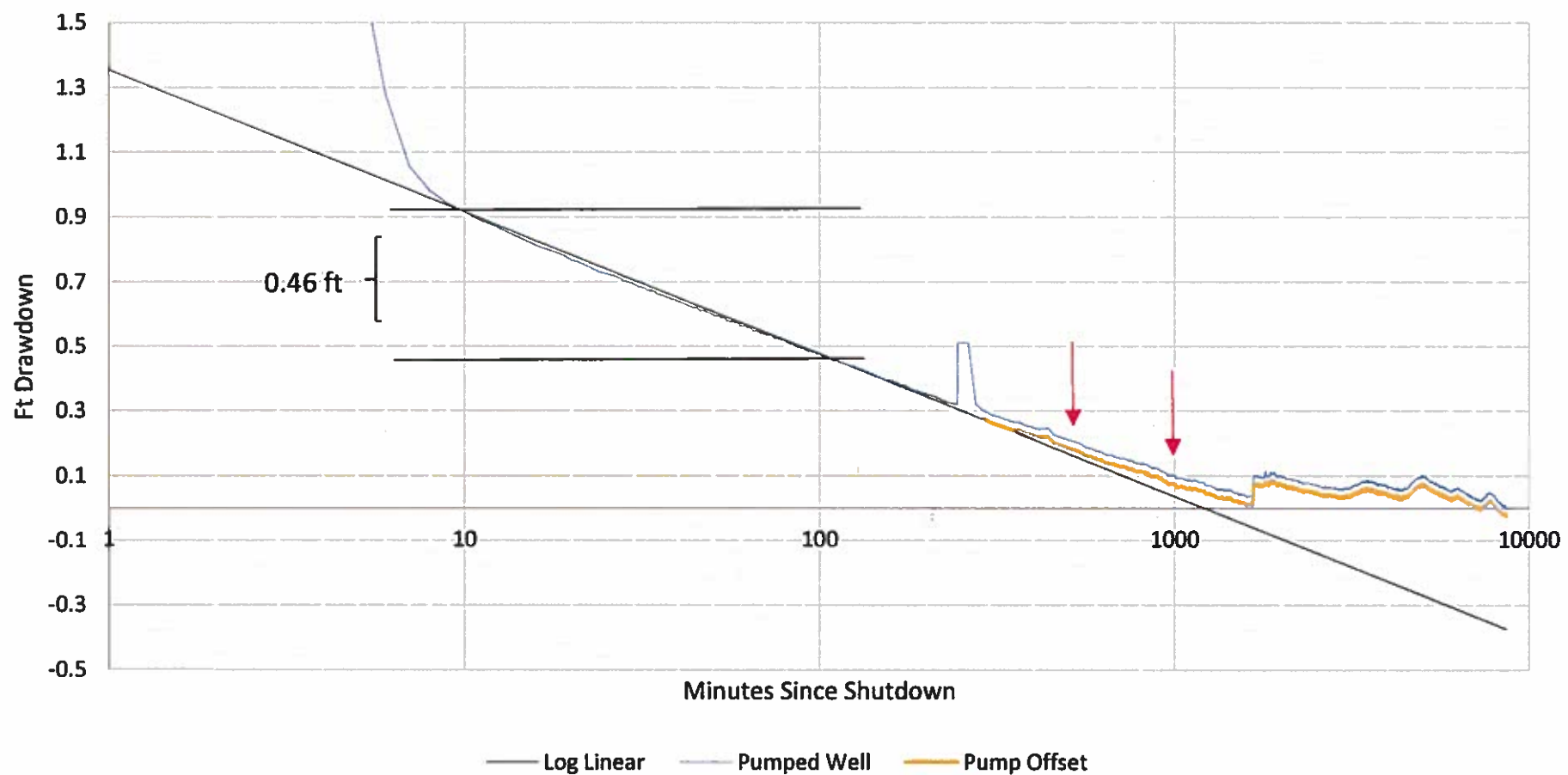


Figure 4. P20 Recovery Data

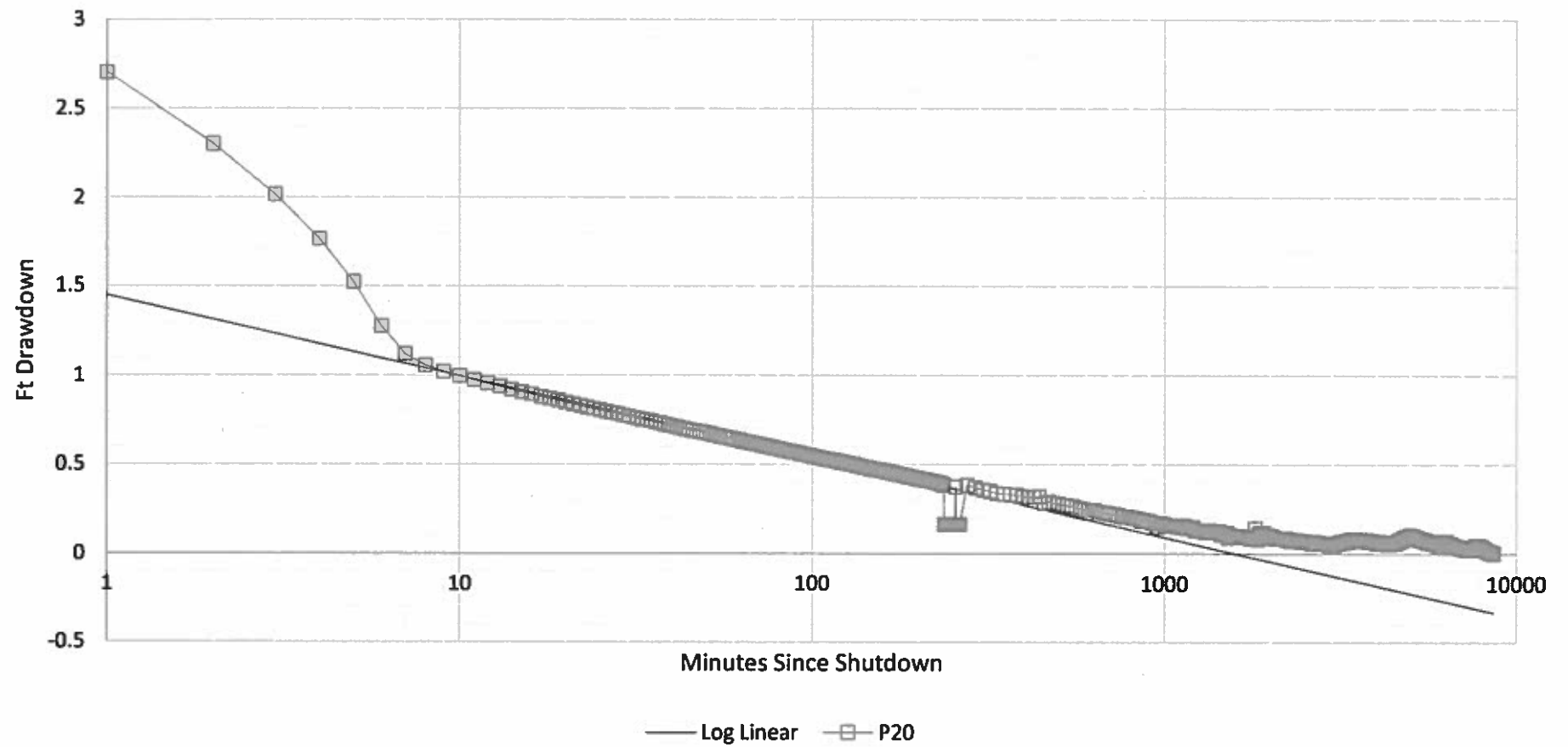


Figure 5. P20 Recovery Graphical Analysis

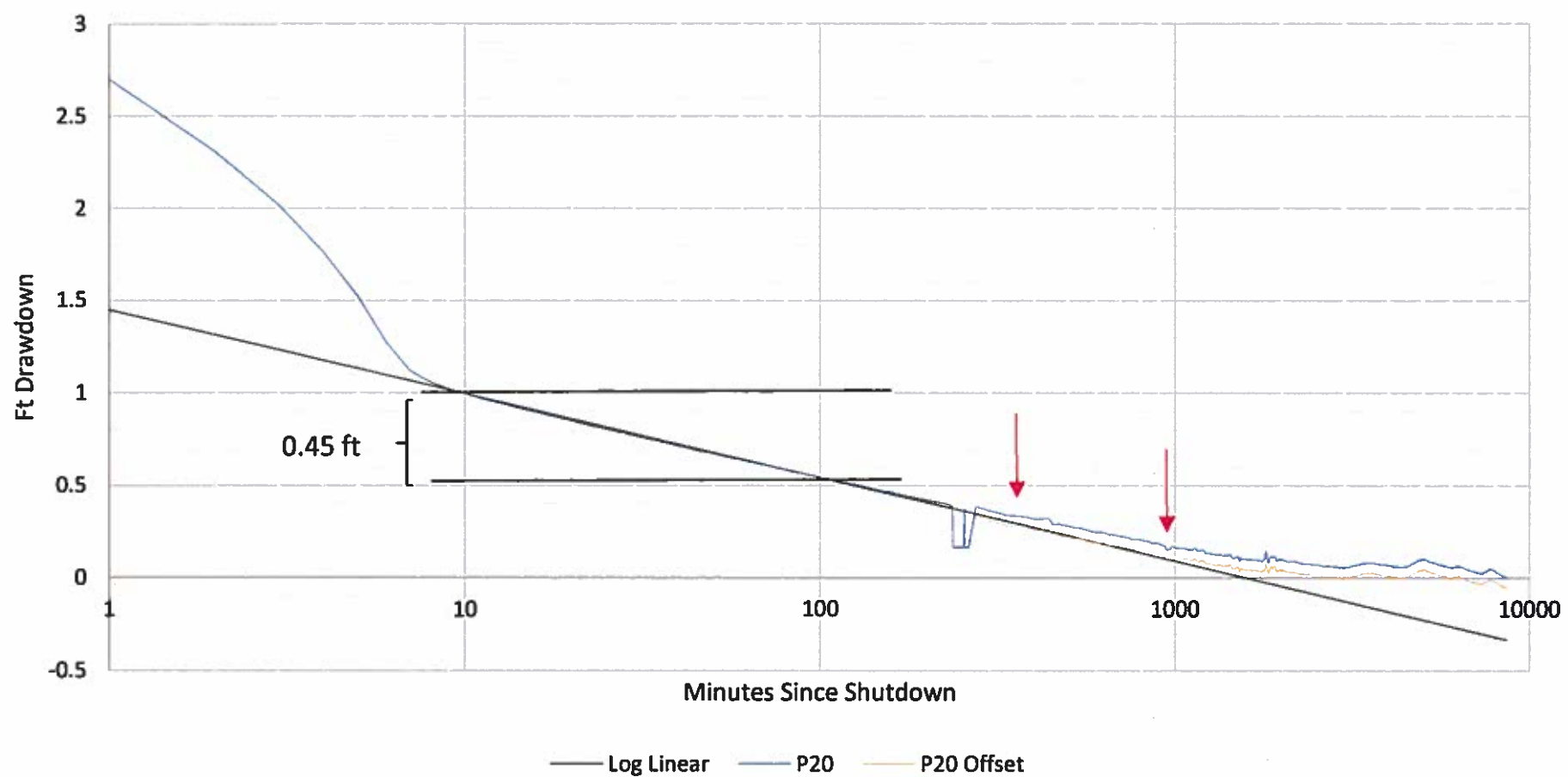


Figure 6. P50 Recovery Data

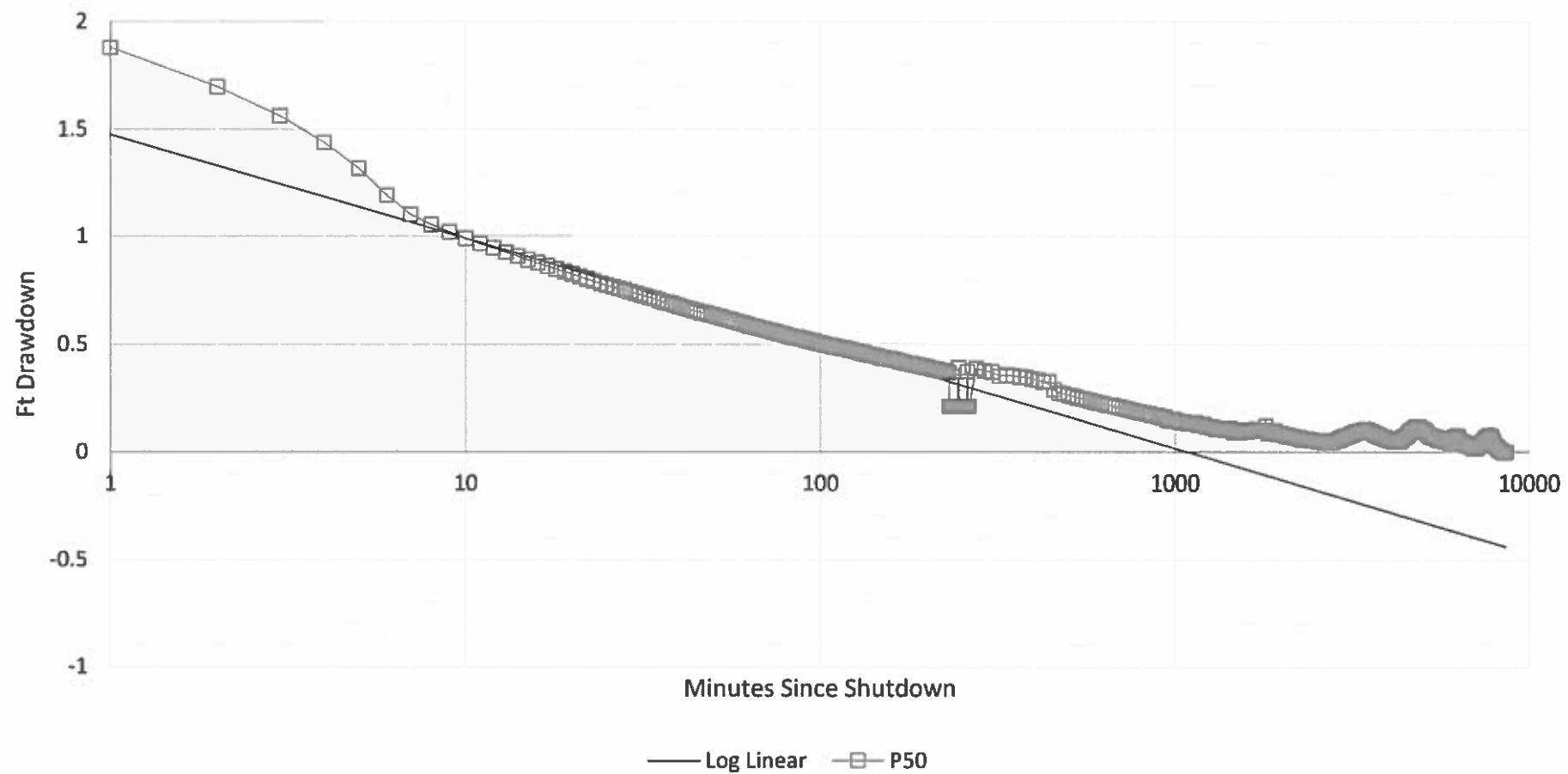


Figure 7. P50 Recovery Graphical Analysis

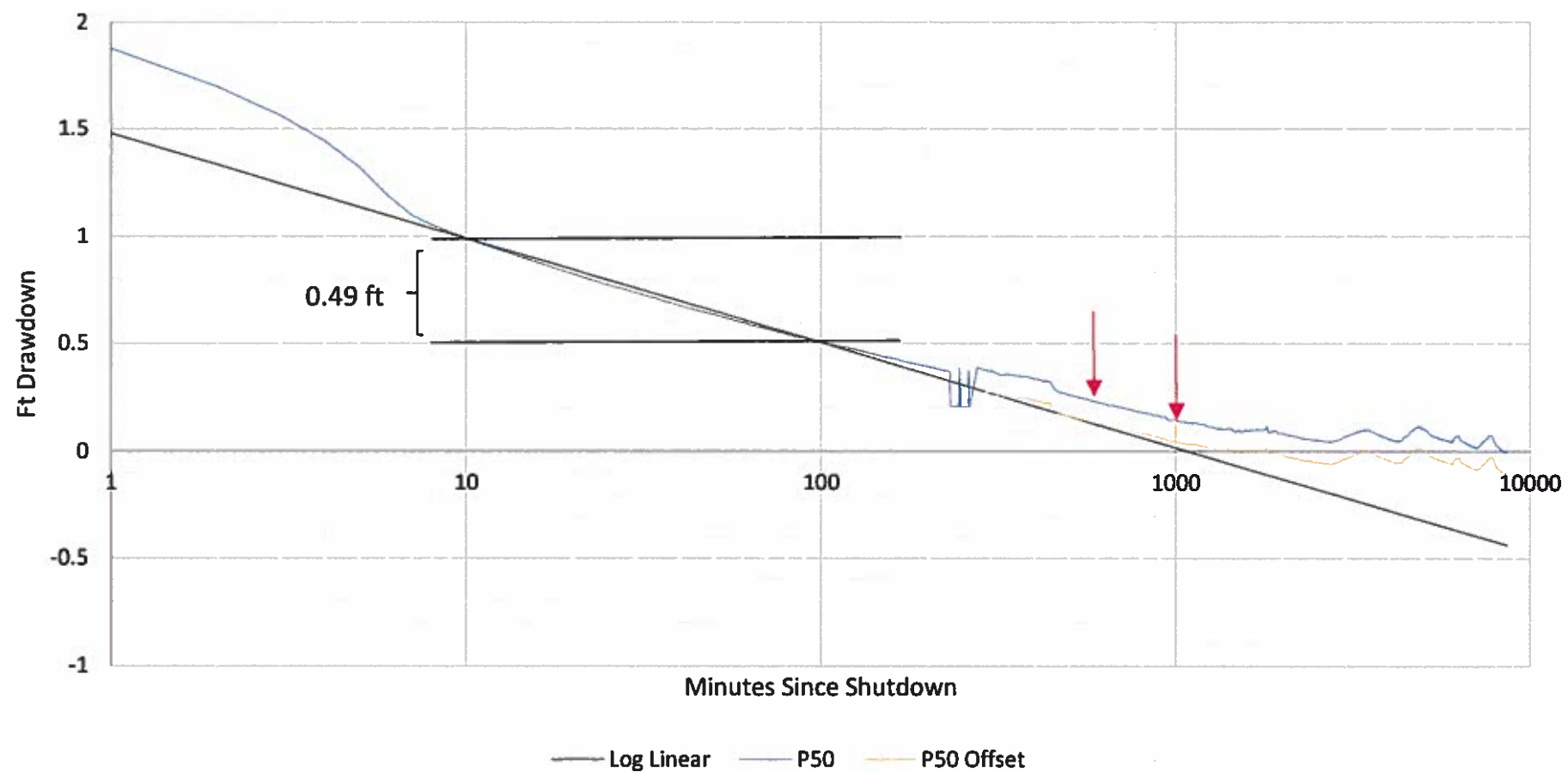


Figure 8. P100 Recovery Data

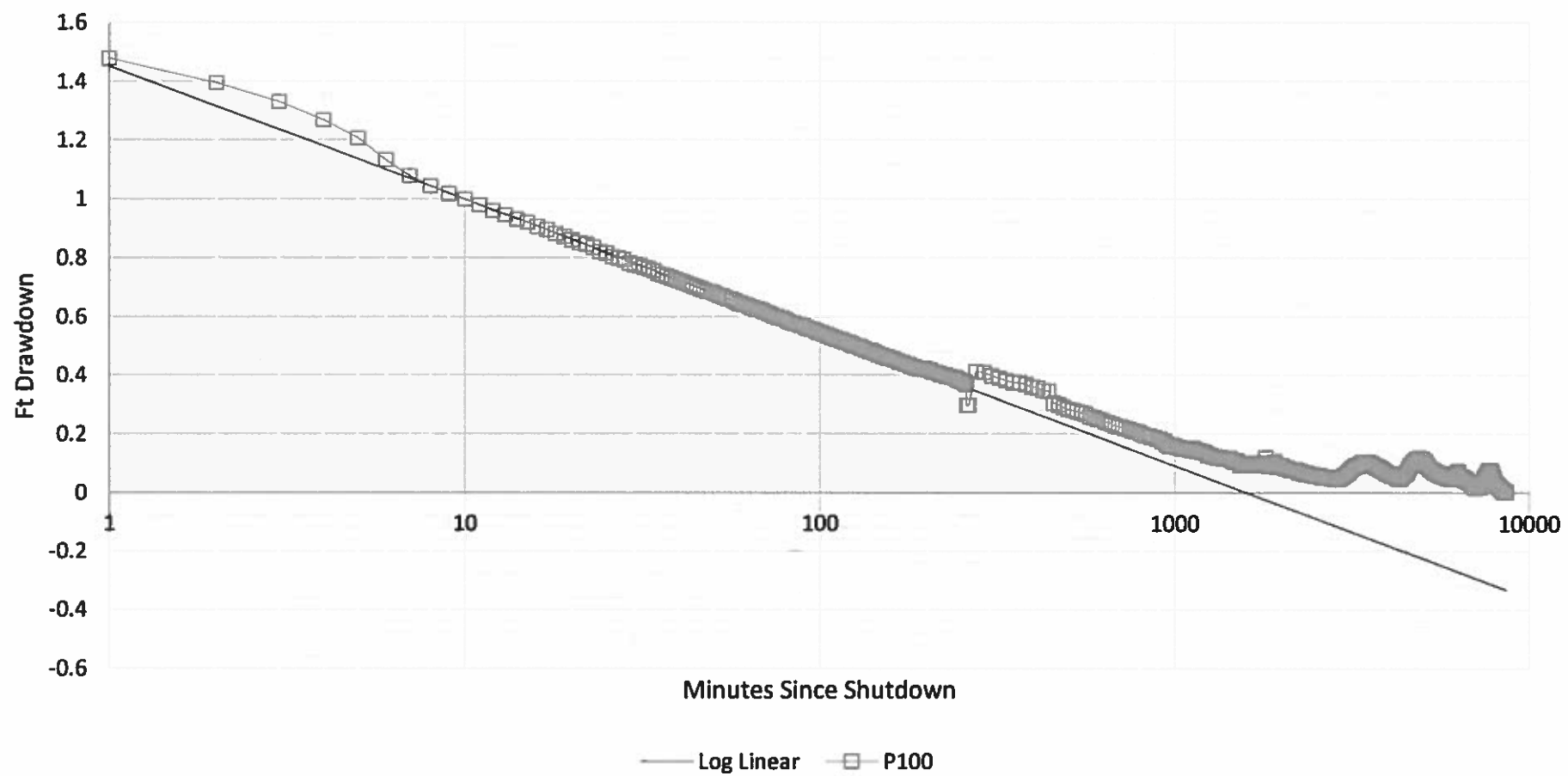


Figure 9. P100 Recovery Graphical Analysis

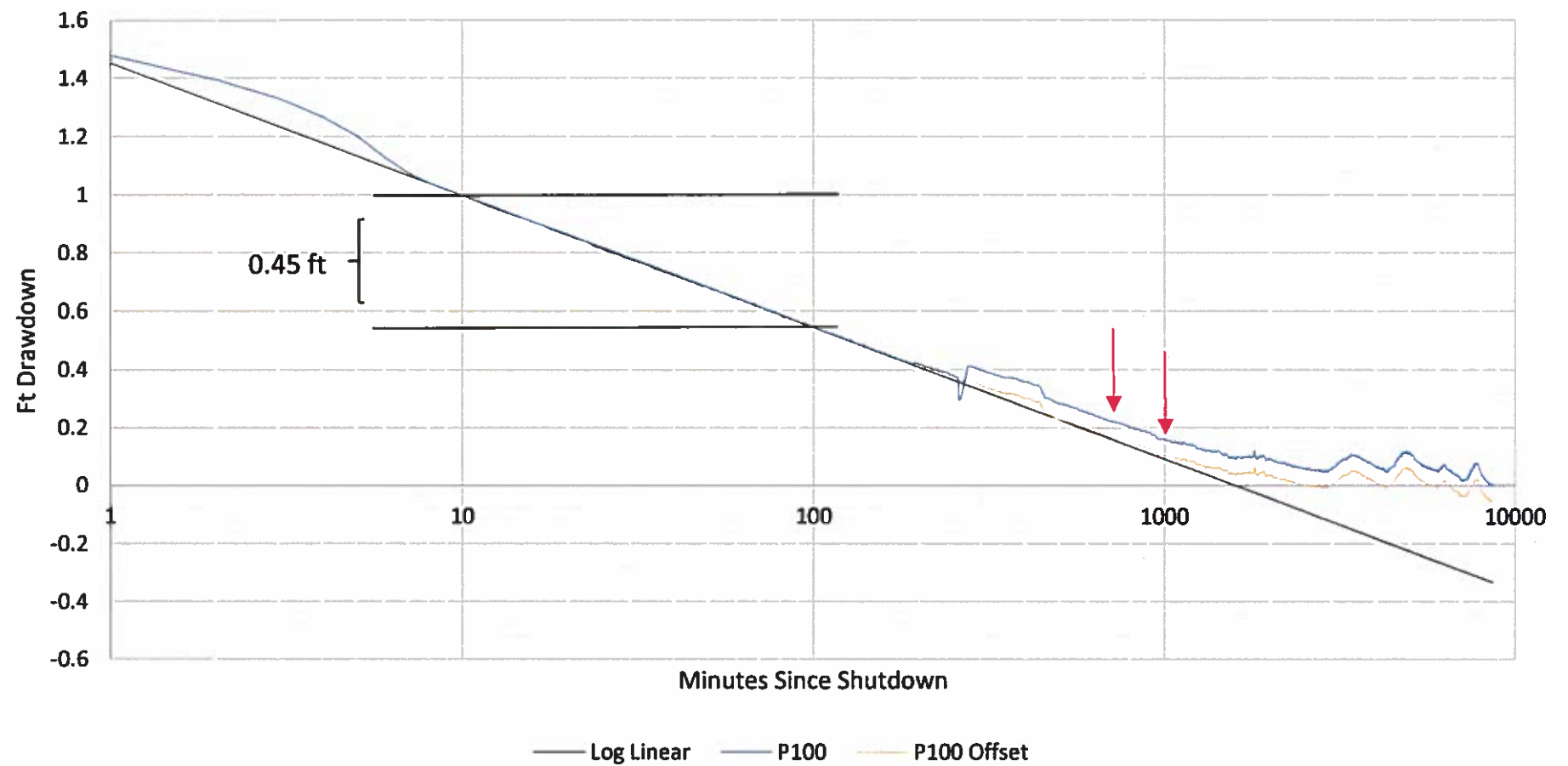


Figure 10. P20 Early Pumping Graphical Analysis

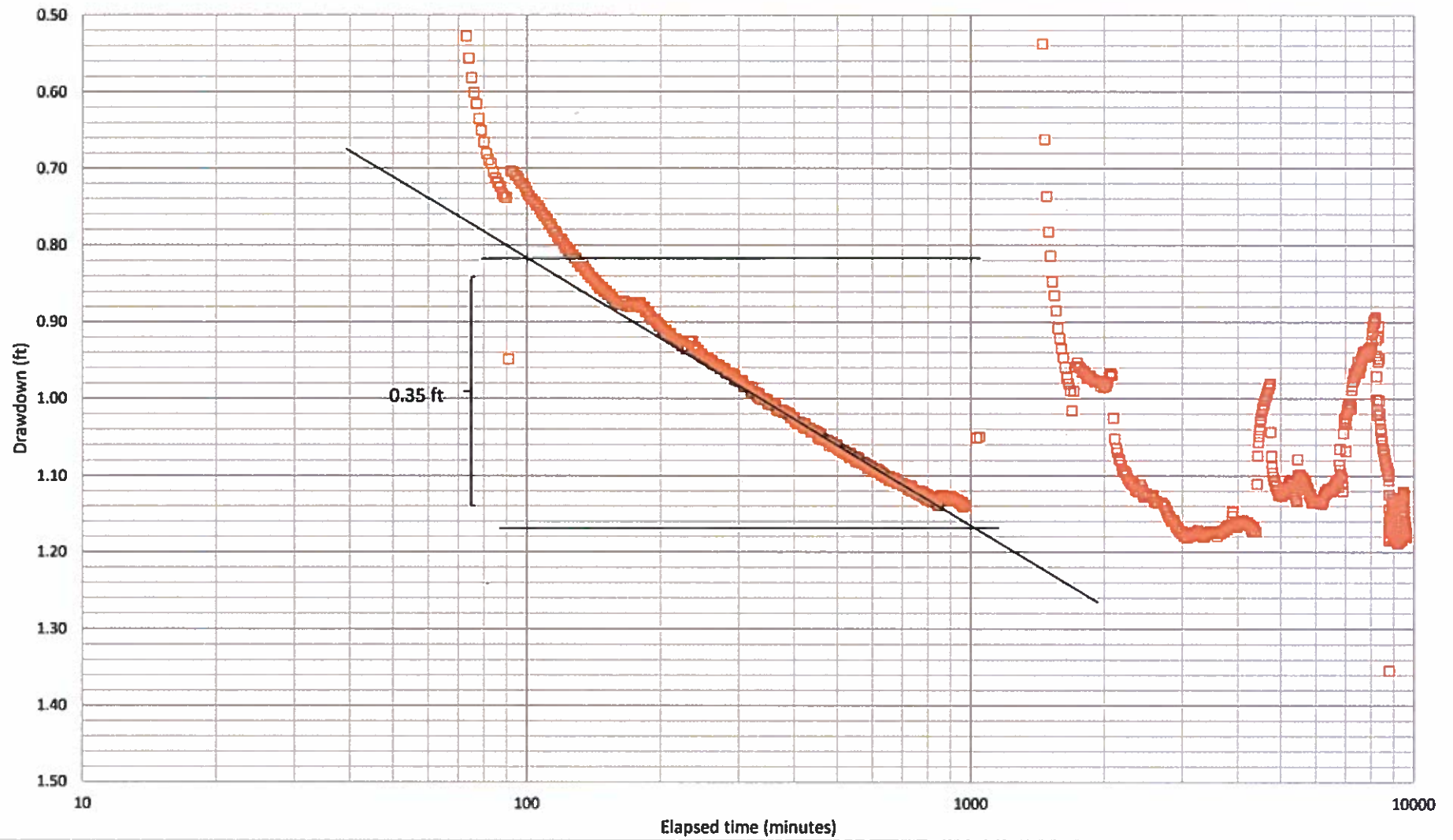


Figure 11. P50 Early Pumping Graphical Analysis

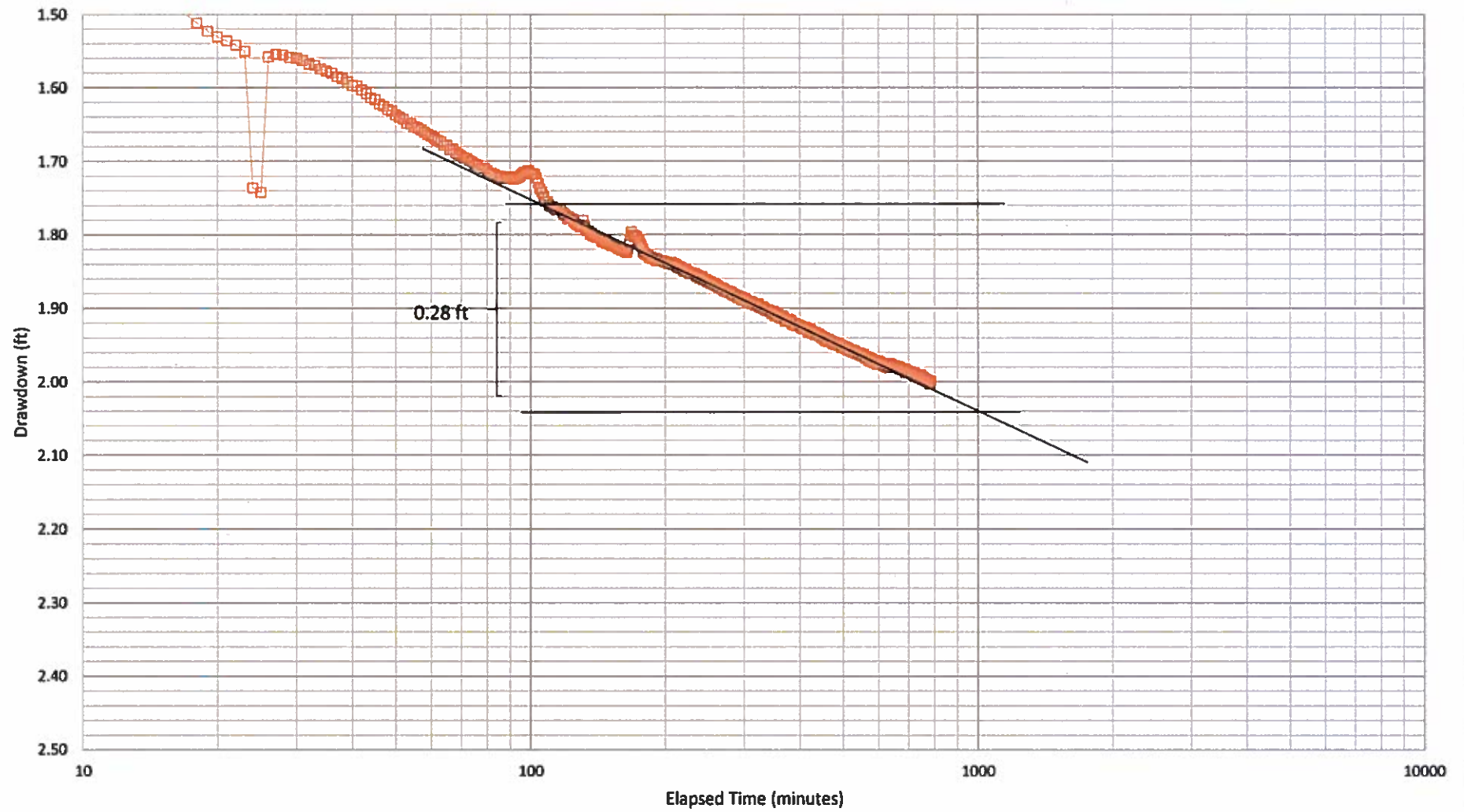
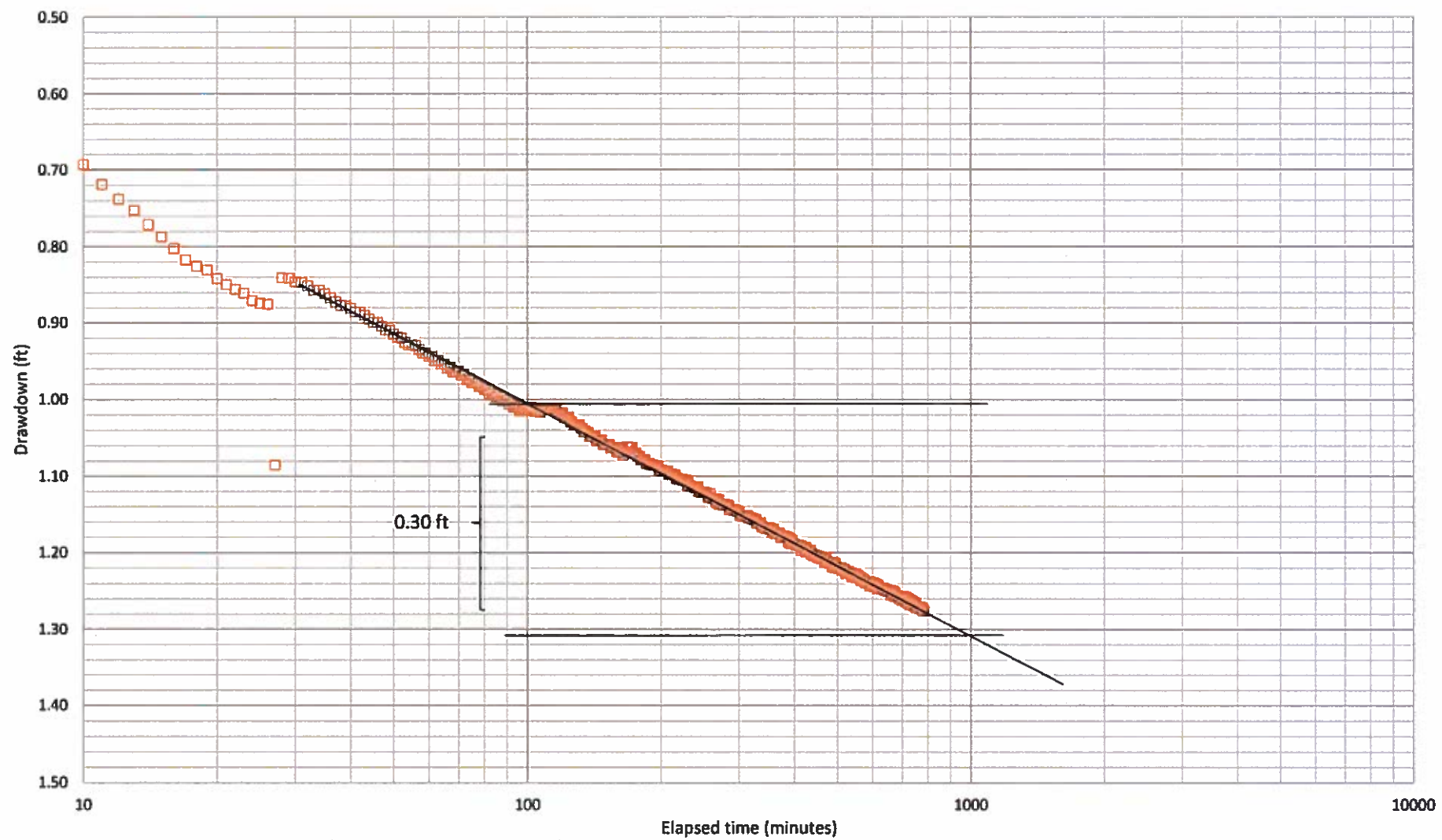
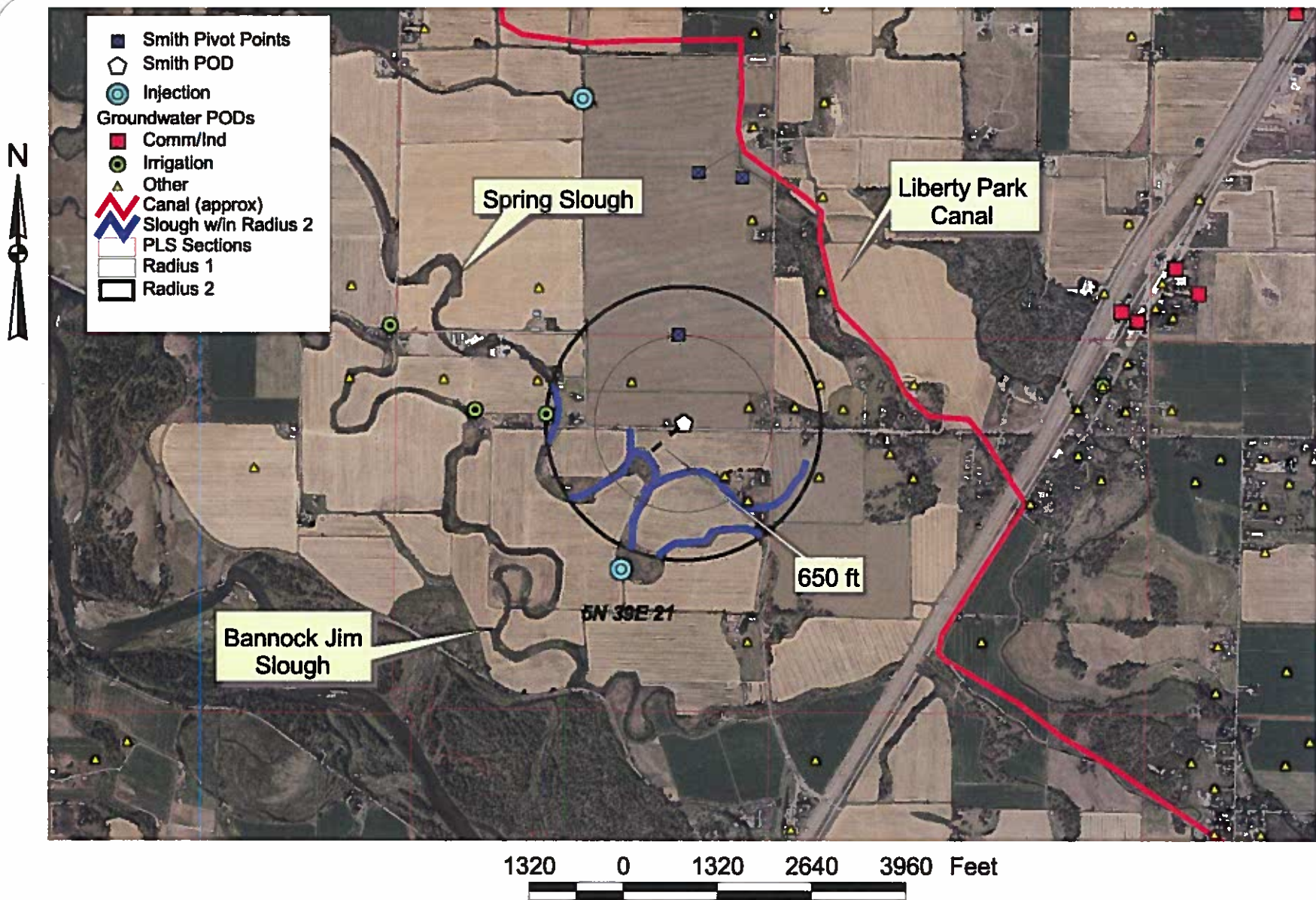


Figure 12. P100 Early Pumping Graphical Analysis

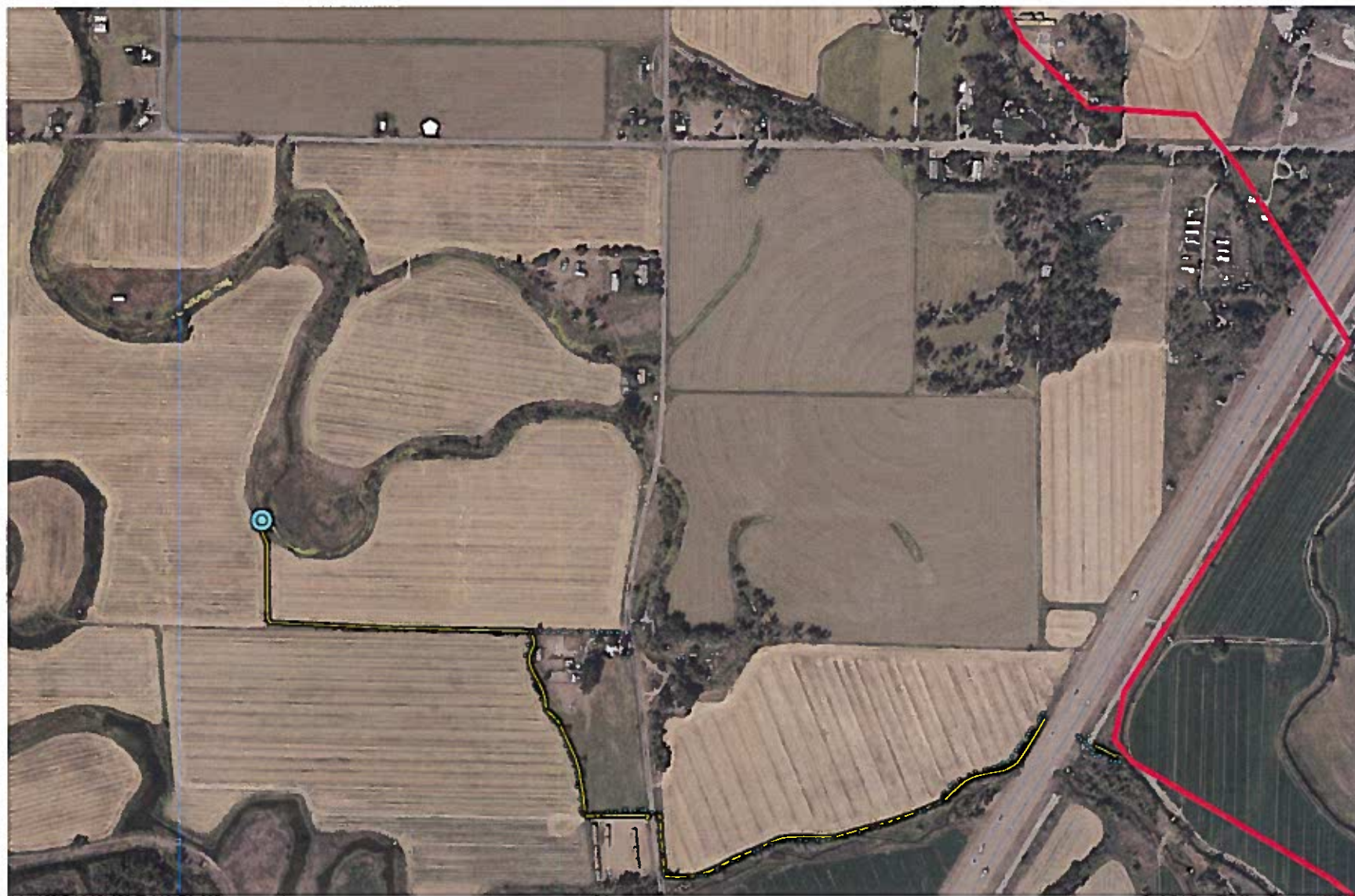




482 Constitution Way, Idaho Falls, ID 83402  
208-524-2353

Figure 13. Geographic Overview

Project 13-0052



660 0 660 1320 Feet



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Figure 14. Ditch Trace

Project 13-0052

- GPS Trace
- ◻ Smith POD
- ◉ Injection
- ≡ Ditch
- ≡ Canal (approx)



Figure 15. Inscription on lateral headgate from Liberty Park Canal.

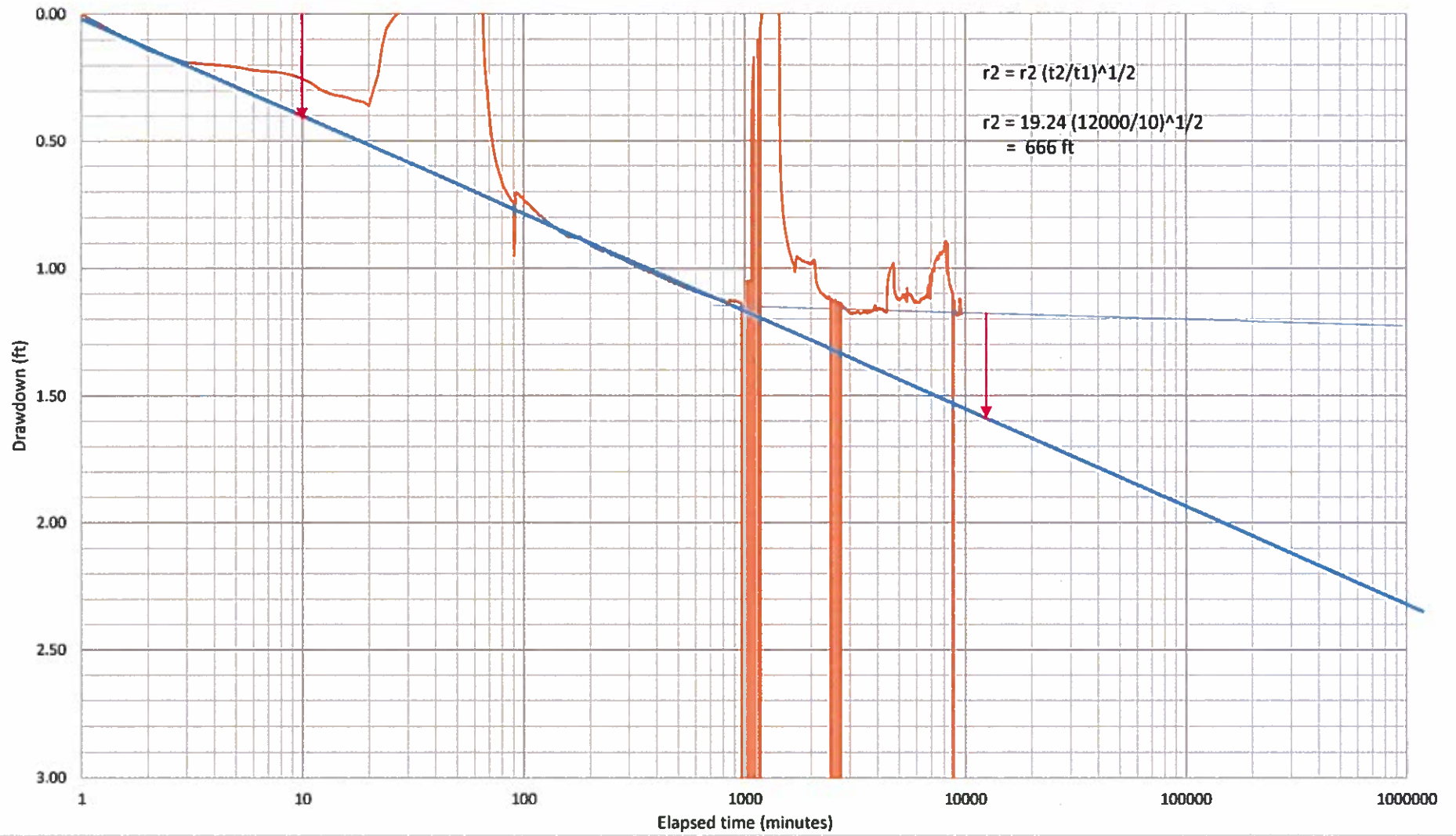


Figure 16. Lateral headgate from Liberty Park Canal, discharging to the west (photo left).



Figure 17. Lateral culvert under US Highway 20.

P20 Time/Drawdown





## **Appendix B**

### **Brief Geologic Description of Surface Geology of T5N, R39R, Sec. 21 & Vicinity**

Chad Jensen

February 26, 2016

During the course of at least two glaciations at approximately 140 ka and 25-13 ka (Licciardi and Pierce, 2008), the Snake River transported enormous quantities of gravel onto the Snake River Plain (Scott, 1982). In the subject area, the two forks of the Snake River filled a subsiding basin with hundreds of feet of alluvium (Phillips and Welhan, 2011). The attached well logs indicate the top approximately 3 to 10 feet consists of "topsoil". Based on a review of soil descriptions in the area, a typical profile consists of loamy silts, sand, and lesser gravels and clay.

The topsoil is underlain by a unit of terrace alluvium of the Snake River and tributary streams from the late Pleistocene. It is interpreted to have been deposited during a period of waning discharge and stream incision during termination of Pinedale glaciation at approximately 13-14 ka (Noe, 1981; Jorgensen, 1979). This unit is dominated by sand and gravel. The sand consists of quartz, black obsidian, and rhyolitic lithic grains. The gravel consists of rhyolite, basalt and lesser quartzite, sandstone, and granitic clasts. Well logs indicate horizons of mixed of sand and gravel. The thickness is uncertain because the unit cannot be distinguished from older or younger alluvial units in water well logs (Phillips and Welhan, 2011). This unit is noted to be poorly drained with water levels approximately 6 feet from surface.

Below this terrace alluvium is alluvium of Snake River outwash, deposited during the Pinedale glaciation by meltwaters (Scott, 1982), also from the late Pleistocene. This unit is gravel and sand composed dominantly of very hard pink, purple, and gray quartzite with lesser rhyolite, basalt, sandstone, gneiss, and granitic rocks (Phillips and Welhan, 2011). Exposures of this unit can be observed at local gravel pits. These exposures indicate thickly planar to cross-bedded strata separated locally by thin, cross-bedded sand layers. Gravel is mostly pebble to cobble-sized, clast supported, locally normally graded and imbricated. Gravel framework is filled by fine to medium sand composed of subangular black obsidian, quartzite, quartz and feldspar crystals, muscovite, and fragments of basalt and rhyolite. Sand beds are locally black because of high obsidian content (Phillips and Welhan, 2011). Minimum thickness is approximately 164 feet though this is uncertain because possible older units cannot be reliably separated in water well logs.

The Geologic Map of the Ririe Quadrangle, Jefferson, Madison, and Bonneville Counties, Idaho, includes a cross section in the northwest quadrant, which indicates a depth to bedrock of approximately 280 feet. This cross section is approximately 2 miles from the subject area.



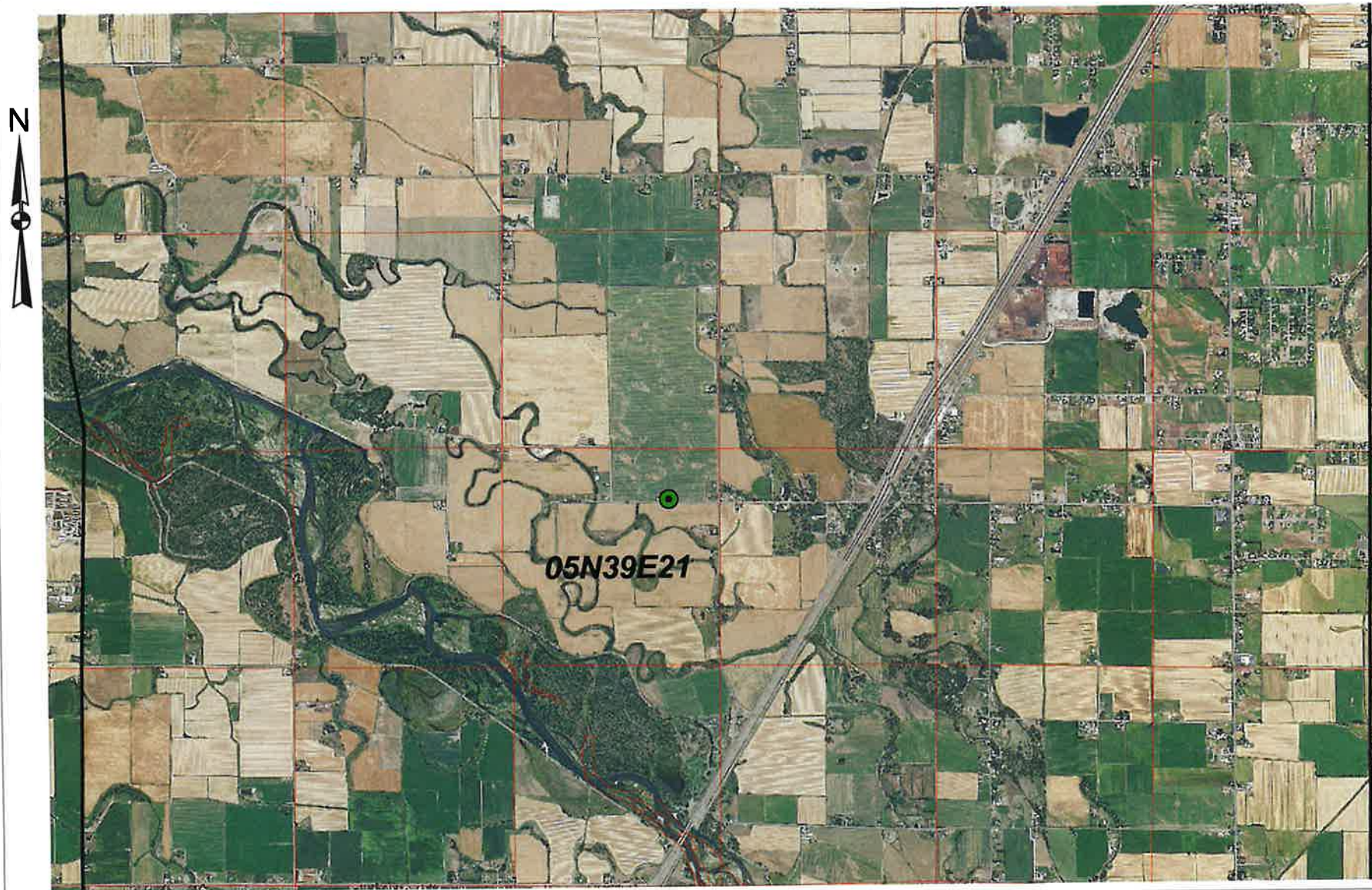
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FORWARD WHITE COPY TO WATER RESOURCES





**Rocky Mountain  
ENVIRONMENTAL™**  
ASSOCIATES, INC.

482 Constitution Way, Idaho Falls, ID 83402  
208-524-2353

**Figure 1: Site Location Map**

*Boyd Smith Well Interaction Analysis -  
Jenkins Methodology*

*Project 13-0052*

-  POD
-  Township/Range
-  Sections



200 0 200 400 Feet



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Figure 2: Site Detail

Boyd Smith Well Interaction Analysis -  
Jenkins Methodology

Project 13-0052

- Features
- Township/Range
- Sections
- QQ



200 0 200 400 Feet



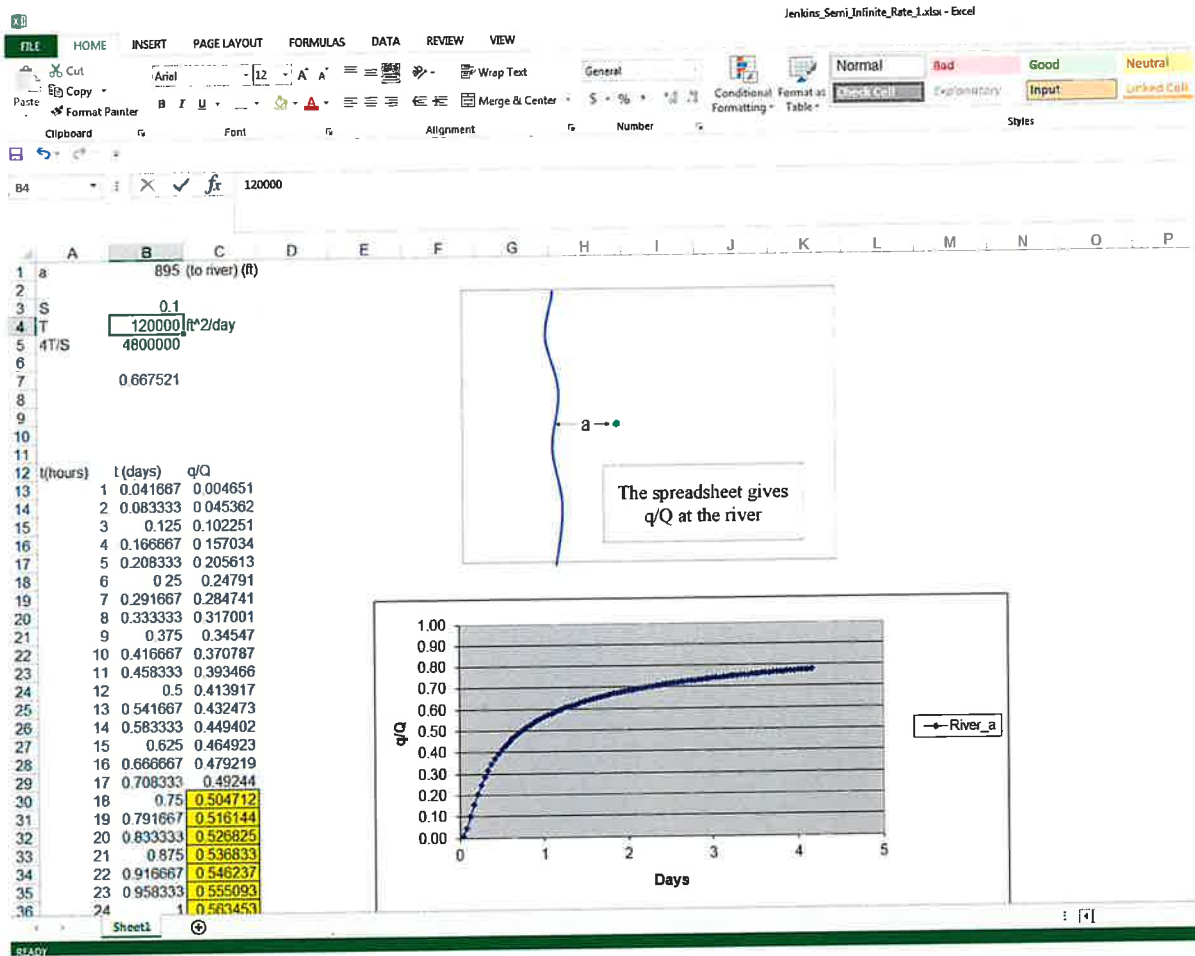
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208-524-2353

**Figure 3: Geometry for Jenkins Calculation**  
**Boyd Smith Well Interaction Analysis -**  
**Jenkins Methodology**

Project 13-0052

-  Linear Approximation
-  Pod.shp
-  Township/Range
-  Sections
-  QQ

**Figure 4**  
**Screenshot of Jenkins Calculation**





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*Ground Water Development and Exploration*

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September 11, 2015

Rocky Mountain Environmental, Inc.  
Mr. Bryce Contour, Senior Hydrogeologist  
482 Constitution  
Idaho Falls, ID 83402-3537

Subject: Letter Report on Pumping Test Analysis for the Boyd Smith Well

Dear Mr. Contour,

I have completed my analysis of the pumping test data for the Boyd Smith Irrigation Well near Thornton, Idaho. Since you conducted the pumping test and collected the data you will have the specifics on the test. Briefly, a 7 day pumping test and 7 days of recovery monitoring were conducted in an existing irrigation well. Three piezometers were installed at radial offsets of 19.24 ft, 48.83 ft and 98.79 ft. Water levels were monitored using automated pressure transducers and all water level data were corrected for barometric pressure fluctuations. Rocky Mountain Environmental Associates (RMEA) performed a careful QA/QC analysis of the data and provided me with excel spreadsheets of the water level data. Flow was estimated based on the discharge capacity of the sprinkler system that the pumped water supplied and by calculating the flow based on the power consumed during the pumping test. You provided me with a range of volume discharge of 55 to 72 ac-ft. This means that on average the flow rate during pumping ranged between 1,860 and 2,436 gpm. The pumped well experienced a maximum drawdown of approximately 9 ft and the observation wells recorded between 1 and 4 ft of drawdown depending upon the radial offset from the pumped well. This is a very well documented test with a relatively clean data set (minus some starts and stops of pumping). It provides a very good measurement of aquifer parameters in this area.

The data were initially processed and put in terms of elapsed time from the start of pumping, feet of drawdown from static and missing lines in the data string were removed (lines had been inserted for comments). At an elapsed time of 9881 minutes the transducer was removed and then reinstalled in the pumped well. Upon reinstallation the transducer got hung up and did not return to its original setting. A length of 0.631 ft was added to the drawdown after elapsed time 9637 minutes to make the recovery match the earlier data. This has no impact on the data quality as the offset did not affect the relative changes in water levels during the recovery test and the transducer was fully submerged. In addition to the barometric correction made by RMEA it was apparent that during the test water levels in the aquifer were impacted by regional fluctuations. This is probably related to irrigation pumping occurring during the pumping test. The corrections were small, +0.1, +0.17, +0.21 and +0.3 ft in wells Pump, P20, P50 and P100, respectively. The fully processed data sets for the wells are presented in Figure 1. As can be seen in Figure 1, the flow rate of the pump was interrupted or altered five times during the pumping test. The shut down and variation in pumping rate early in the test (day 1) are the most problematic for aquifer analysis. The recovery test data is much better because the pumped well was left alone for the entire 7 day period. Figure 2 is a plot of the recover test data.

Several analytical solutions were attempted to find the best match to the aquifer test data. Initially, the Theis Solution was attempted. A suitable match for the Theis curve could not be found. This is not surprising because the aquifer in this location is very shallow and almost certainly unconfined or leaky confined. The best match to the data was to the leaky confined analytical solution shown in Figure 3. A good match could not be found to the unconfined analytical solution as shown in Figure 4. The better match to the leaky confined solution was no



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doubt driven by the ability of this analytical solution to yield water from an overlying or underlying aquifer, which mimics the effects of an aquifer source boundary. In this case the source boundaries are the sloughs located southwest of the pumping field. The best estimate of the leaky confined aquifer transmissivity (2,436 gpm) is  $2.9 \times 10^4$  and the specific yield is very high, 0.3, which is unrealistically high probably by a factor of 5 to 10. The match to the data for 1860 gpm yields a transmissivity of  $2.02 \times 10^4$  ft<sup>2</sup>/day. For the Cooper-Jacob straight line method (Figure 5) for a confined aquifer the transmissivity ranges from  $1.5$  to  $2.2 \times 10^5$  ft<sup>2</sup>/day for 1860 and 2436 gpm, respectively. The storativity is more realistic for this method approximately 0.0001.

It is difficult to quantify the uncertainty of these calculations because the automated type curve matching routine in AQTESOLV failed to converge. Additionally, the two different aquifer type analytical solutions yield values that are roughly an order of magnitude apart. Roughly speaking the estimate of transmissivity ranges from  $2.0 \times 10^4$  ft<sup>2</sup>/day to  $2.2 \times 10^5$  ft<sup>2</sup>/day. The calculated storativity is unreliable. Problems with the match to the analytical solutions are driven mainly by the presences of an interfering hydraulic boundary that impacts the shape of the recovery curve (discussed below). Based on these types of materials, the anticipated range of storativity is between 0.0001 and 0.1. The wide range is driven by the natural variation in aquifer confinement conditions in this area.

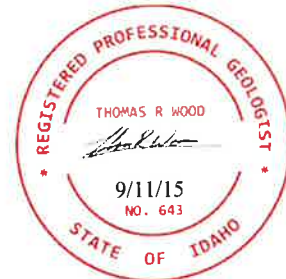
A detailed examination of the drawdown and recovery curves on semi-log scale plots illuminates the fact that the water levels in wells came to effective or quasi-steady state conditions relatively early in the pumping test. Figure 6 shows all data plotted at a semi-log scale with solid lines indicating trends in drawdown (left side) and a near horizontal line indicating near quasi-steady state when pumping is being satiated by a recharge boundary. Figure 7 is a plot of the recovery data at a semi-log scale. There is a bit of scatter or noise in the data most likely attributed to several factors: 1) other irrigation wells turning off and on; regional and diurnal changes in water levels; and variations in the pumping rate of the test well. However, the test results are conclusive in that they demonstrate that pumping of the Boyd Smith creates a cone of depression that quickly comes into equilibrium with a nearby source of surface water. This is demonstrated by the stabilization of the drawdown curves in less than 1,000 minutes. Additionally, the aquifer test drawdown and recovery data could not be matched to an analytical solution not containing an inherent supply of water, i.e. the field data departed significantly below the Theis Curve. The Theis solution assumes an infinite aquifer without a source of recharge. The behavior of the drawdown and recovery curves for this test are characteristic of aquifers with a recharge boundary. I know that you have performed calculations for the distances to a boundary and I will not repeat them here. I reviewed your calculations and found them to be accurate. The main point is that the pumping test demonstrates that there is a definite connection between surface water and ground water in this location. The cone of depression from the well is established very quickly and it draws water from the slough. There is no doubt that water pumped from the well primarily comes from the nearby surface water bodies.

Please call or email me if you have questions about this letter report.

Sincerely,

A handwritten signature in black ink, appearing to read 'Tom Wood', is written over a horizontal line.

Thomas R. Wood, Ph.D., P.G.  
Idaho Registered Geologist, #643



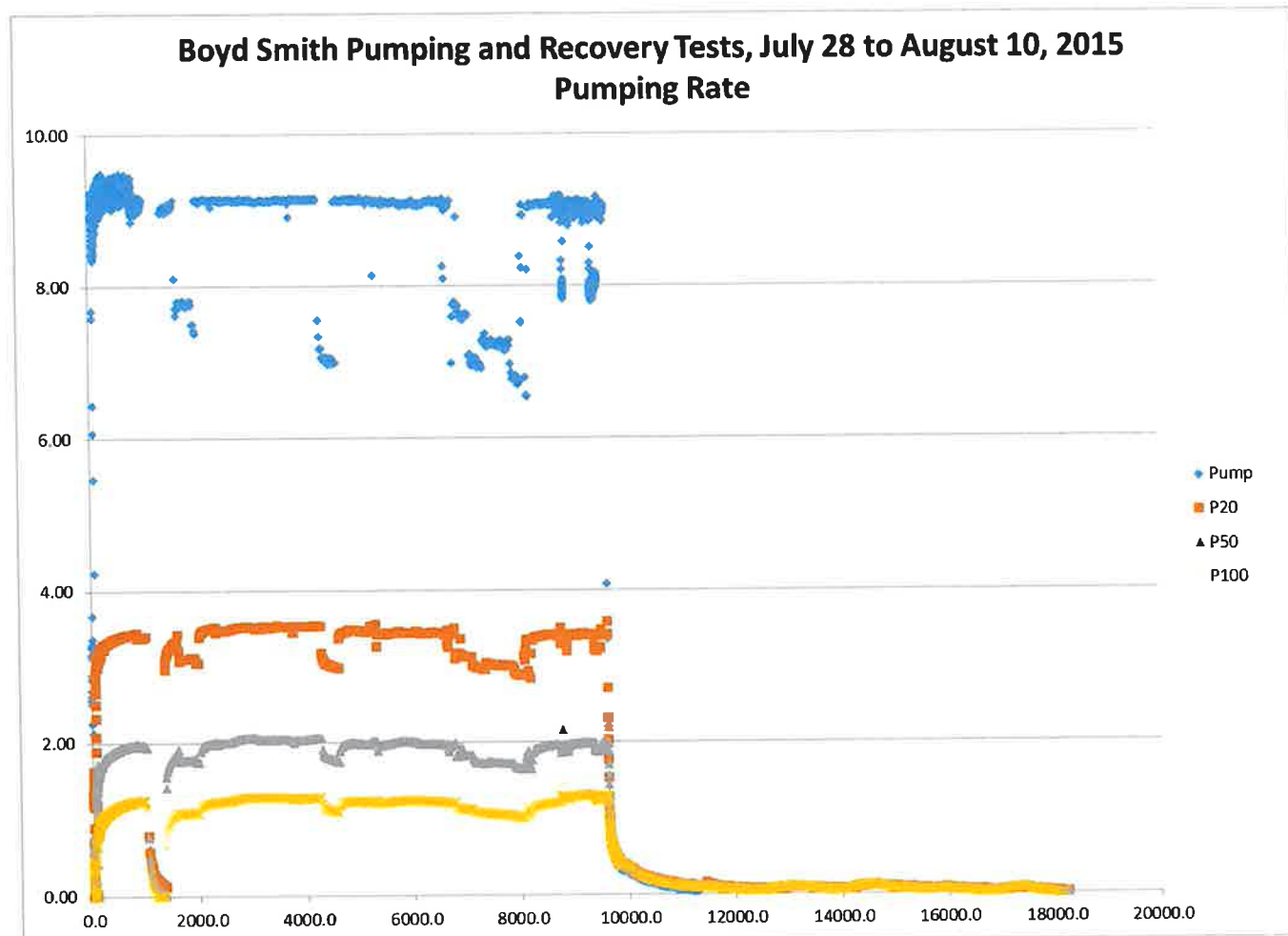


Figure 1. Arithmetic plot of pumping and recovery test data.

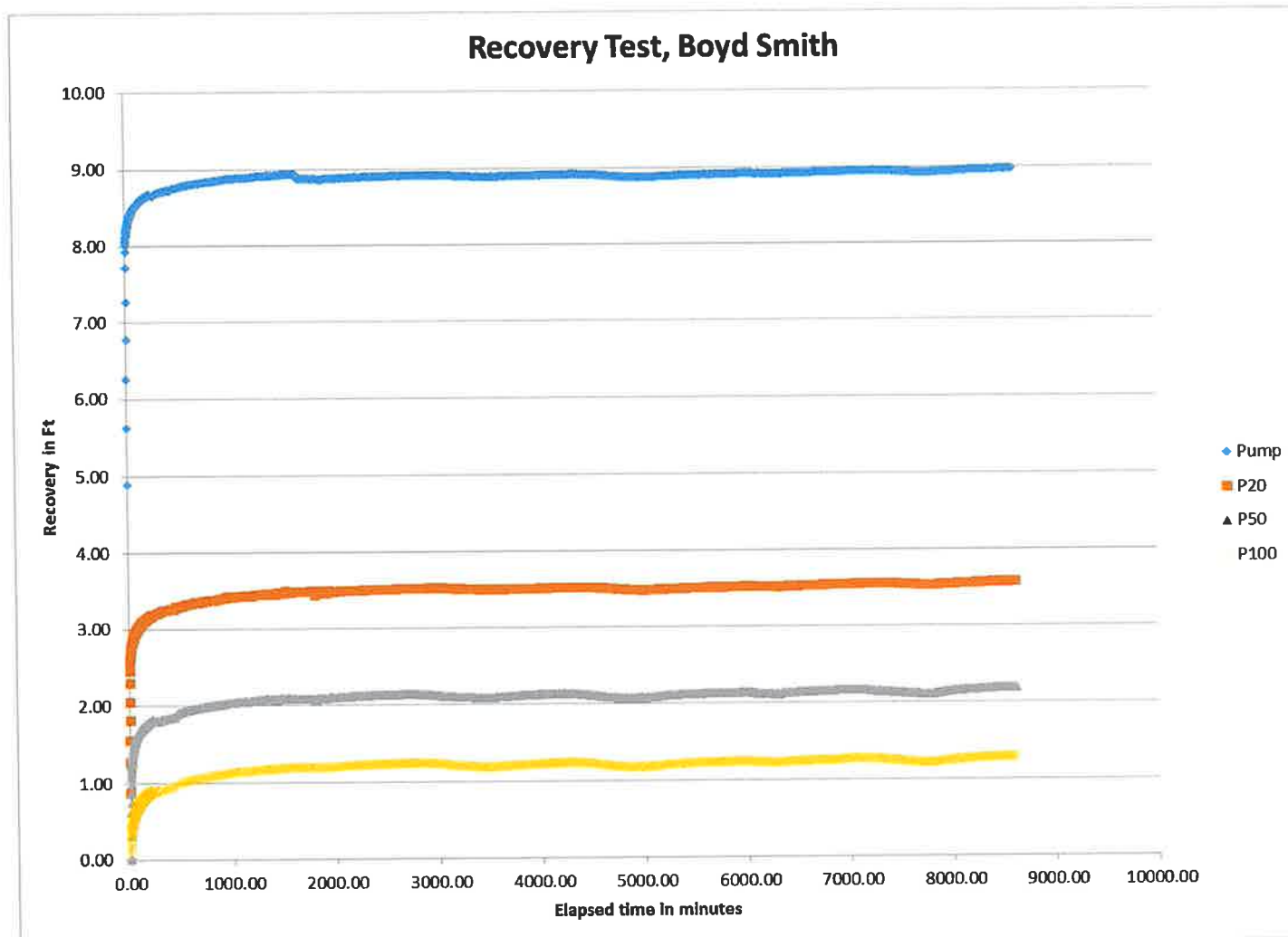


Figure 2. Arithmetic plot of water level recovery after pumping ceased.



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Boyd Smith Recovery Test Plot

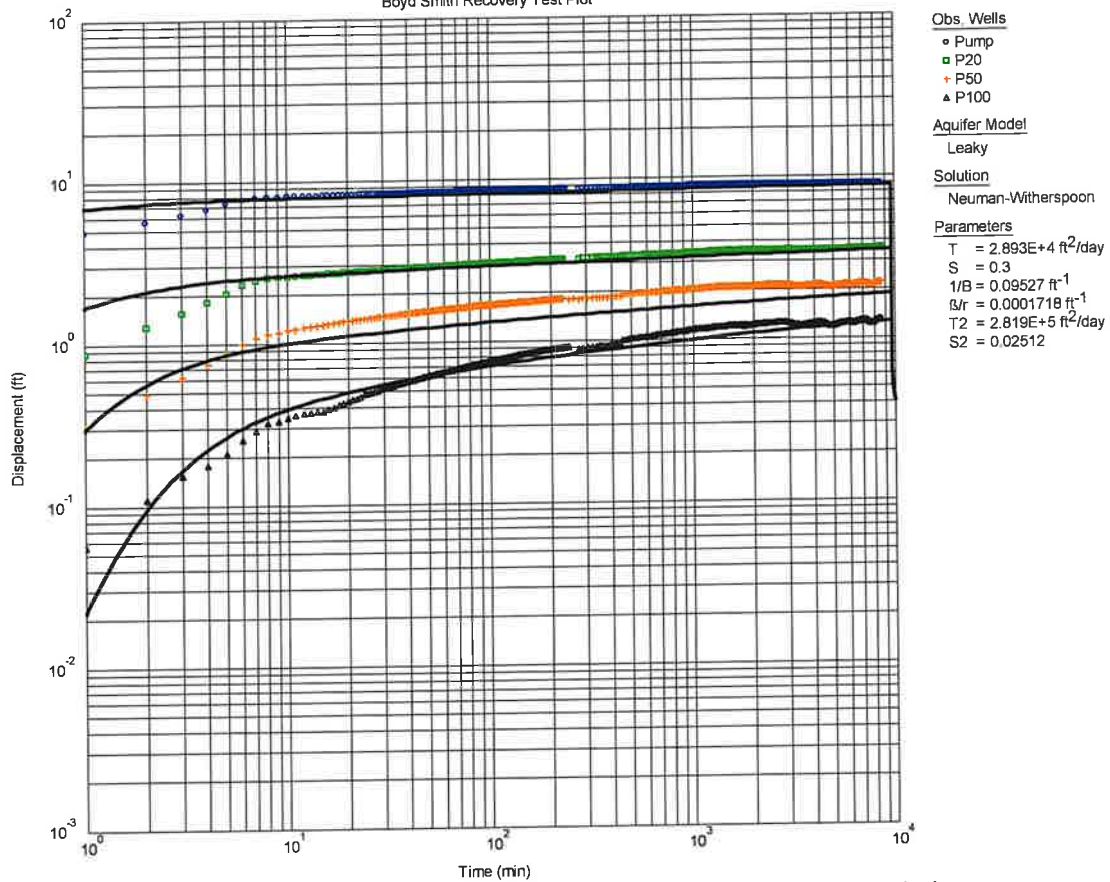


Figure 3. Data match to the Neuman-Witherspoon leaky confined analytical solution.



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Boyd Smith Recovery Test Plot

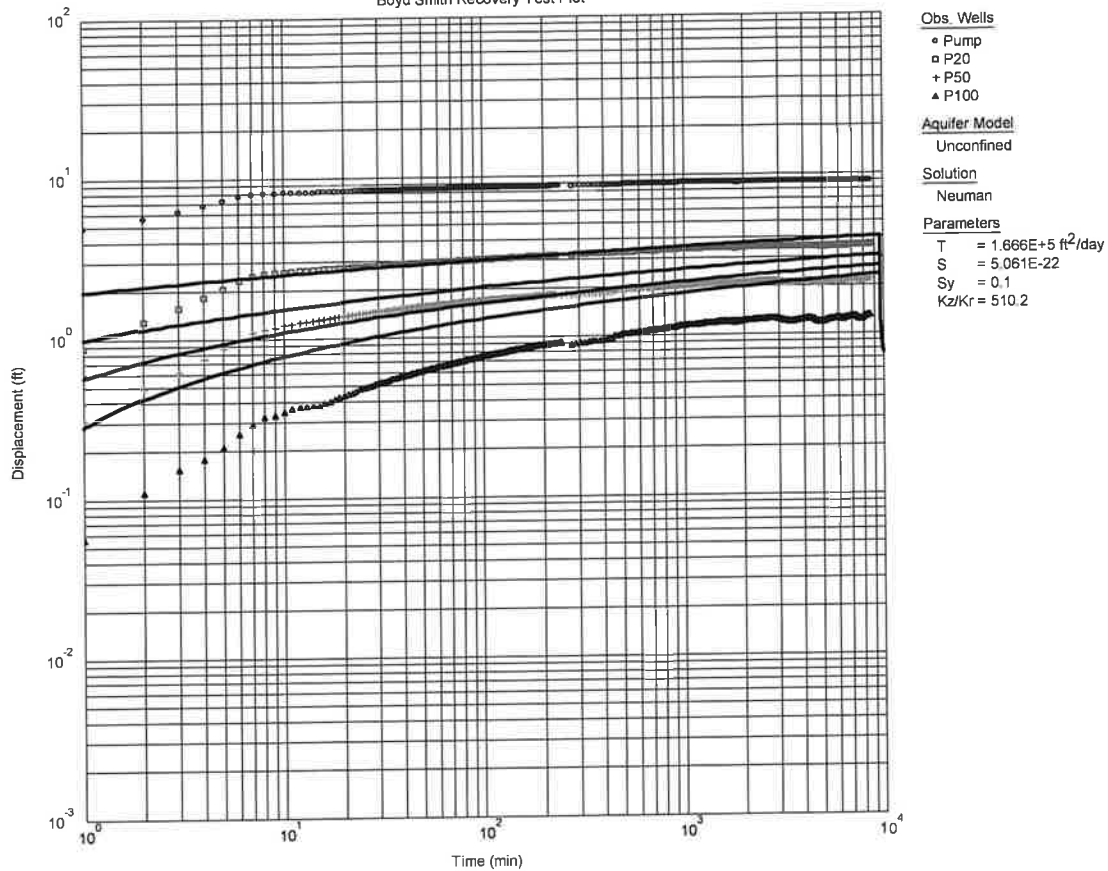


Figure 4. Data match to the Neuman unconfined solution.



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Boyd Smith Recovery Test Plot

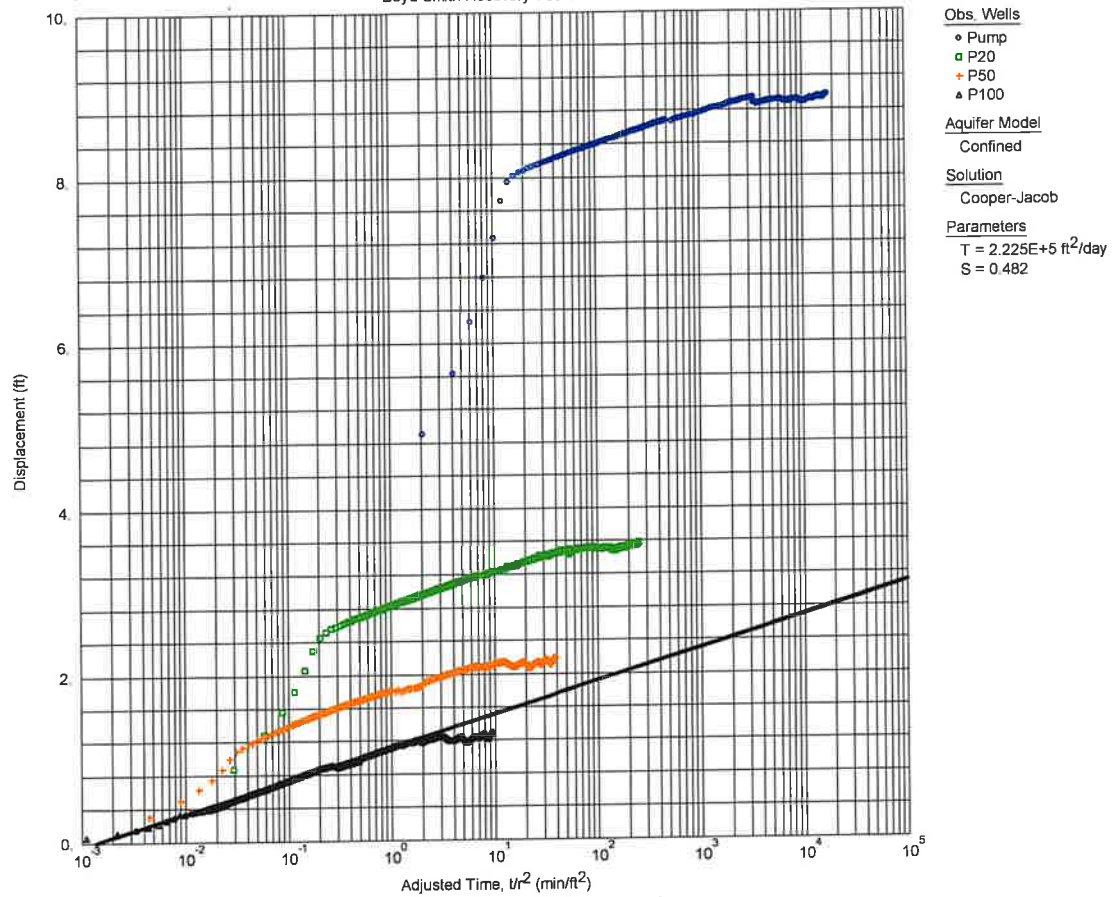


Figure 5. Data match to the Cooper-Jacob confined solution.

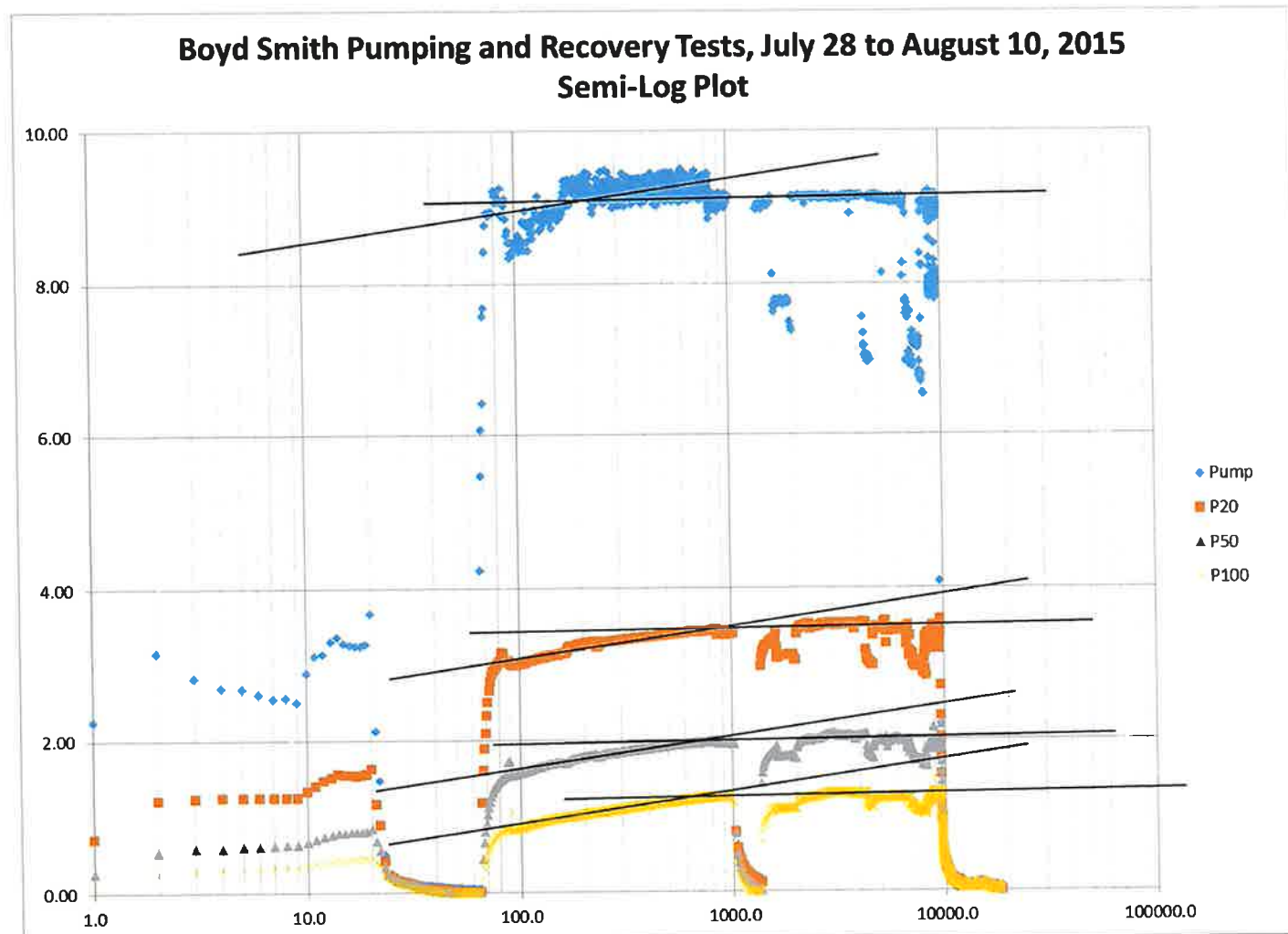


Figure 6. Semi-log plot of both drawdown and recovery data with lines showing the break in slope 700 minutes indicating the onset of near steady state drawdown.

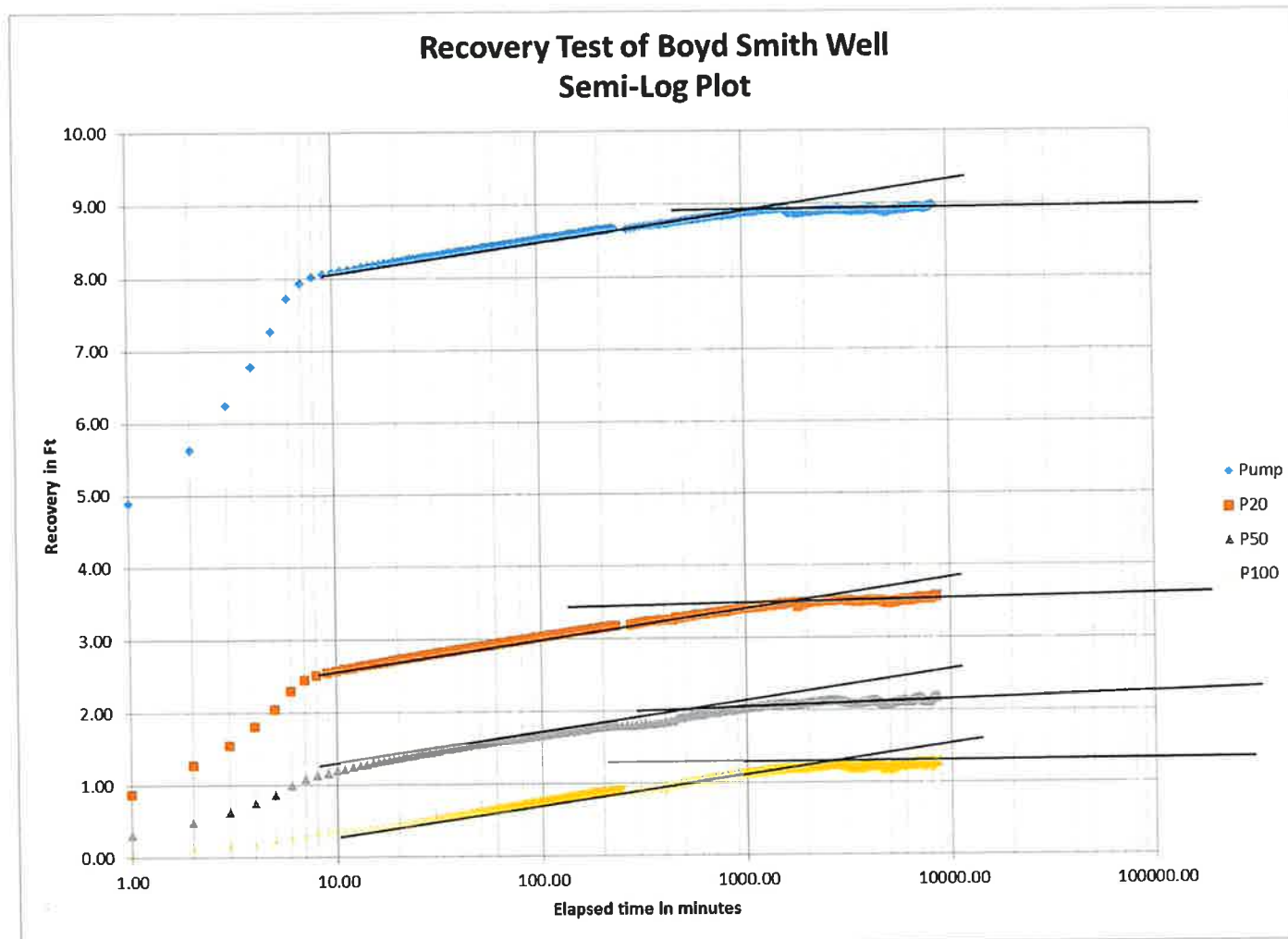


Figure 7. Semi-log plot of recovery data with lines showing the break in slope ~1400 minutes indicating full aquifer recovery.



**Attachment: Graphical Analysis of Distance to Boundary**

1. Methodology
2. Piezometer P20 analysis
3. Piezometer P50 analysis
4. Piezometer P100 analysis

# ***Physical and Chemical Hydrogeology***

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***Second Edition***

***Patrick A. Domenico***

*David B. Harris Professor of Geology  
Texas A&M University*

***Franklin W. Schwartz***

*Ohio Eminent Scholar in Hydrogeology  
The Ohio State University*



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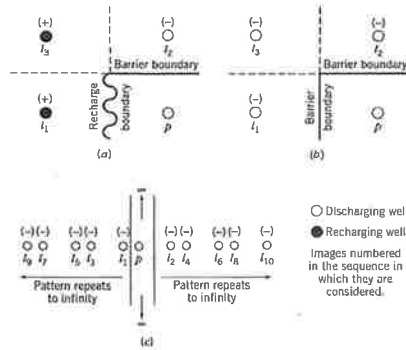


Figure 6.24 Image well systems for (a) wedge-shaped aquifer with both barrier and recharge boundaries, (b) wedge-shaped aquifer with barrier boundaries, and (c) infinite strip.

extent. In other words, graphic models such as given in Example 6.7 can be used to find the drawdown in response to one or several wells pumping simultaneously. Once the distance from real and imaginary wells to the point of interest has been determined, drawdown (or recovery) may be obtained from the graphs. For example, the drawdown at any point in the real aquifer of Figure 6.24 is the algebraic sum of the effect of one real well, one discharging imaginary well, and two recharging imaginary wells. For the infinite array (Figure 6.24c), image wells are added until the most remote pair has negligible influence on water-level response.

#### Example 6.8

A 60-cm-diameter well is located in an aquifer with a transmissivity of  $125 \text{ m}^2/\text{day}$  and a storativity of  $5 \times 10^{-2}$ . A fault (barrier) is located 300 m from the well. The drawdown at the midpoint between the fault and the barrier at the end of one year's pumpage at a rate of  $2700 \text{ m}^3/\text{day}$  is determined

$$s = s_p + s_i = \frac{Q}{4\pi T} [W(u)_p + W(u)_i]$$

$$(u)_p = \frac{r_p^2 S}{4Tt} = \frac{(150 \text{ m})^2 5 \times 10^{-2}}{(4)(125 \text{ m}^2/\text{day})(365 \text{ day})} = 6.16 \times 10^{-3}$$

$$(u)_i = \frac{r_i^2 S}{4Tt} = \frac{(450 \text{ m})^2 5 \times 10^{-2}}{(4)(125 \text{ m}^2/\text{day})(365 \text{ day})} = 5.55 \times 10^{-2}$$

From Table 6.1,  $W(u)_p = 4.52$ ,  $W(u)_i = 2.37$ .

$$s = \frac{2700 \text{ m}^3/\text{day}}{4 \times 3.14 \times 125 \text{ m}^2/\text{day}} (6.89) = 11.85 \text{ m}$$

#### Example 6.9

The accompanying figure depicts a time-drawdown response in an observation well affected by one pumping well and a barrier boundary. By construction,  $s_1 = s_2$ . It thus follows that  $W(u)_1 = W(u)_2$  and

$$u_1 = \frac{r_1^2 S}{4Tt_1} \quad \text{and} \quad u_2 = \frac{r_2^2 S}{4Tt_2}$$

where  $u_1 = u_2$ . Thus

$$r_2 = r_1 \left( \frac{t_2}{t_1} \right)^{1/2}$$

where  $r_1$  is the distance from the pumping well to the observation well, which is known, and  $r_2$  is the distance from the observation well to the imaginary pumping well causing the inflection, which is readily calculated. However, because the exact location of the barrier is not known,  $r_2$  merely defines a set of points that form a circle of radius  $r_2$  with the observation well located at its center.

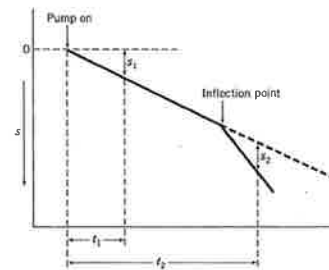
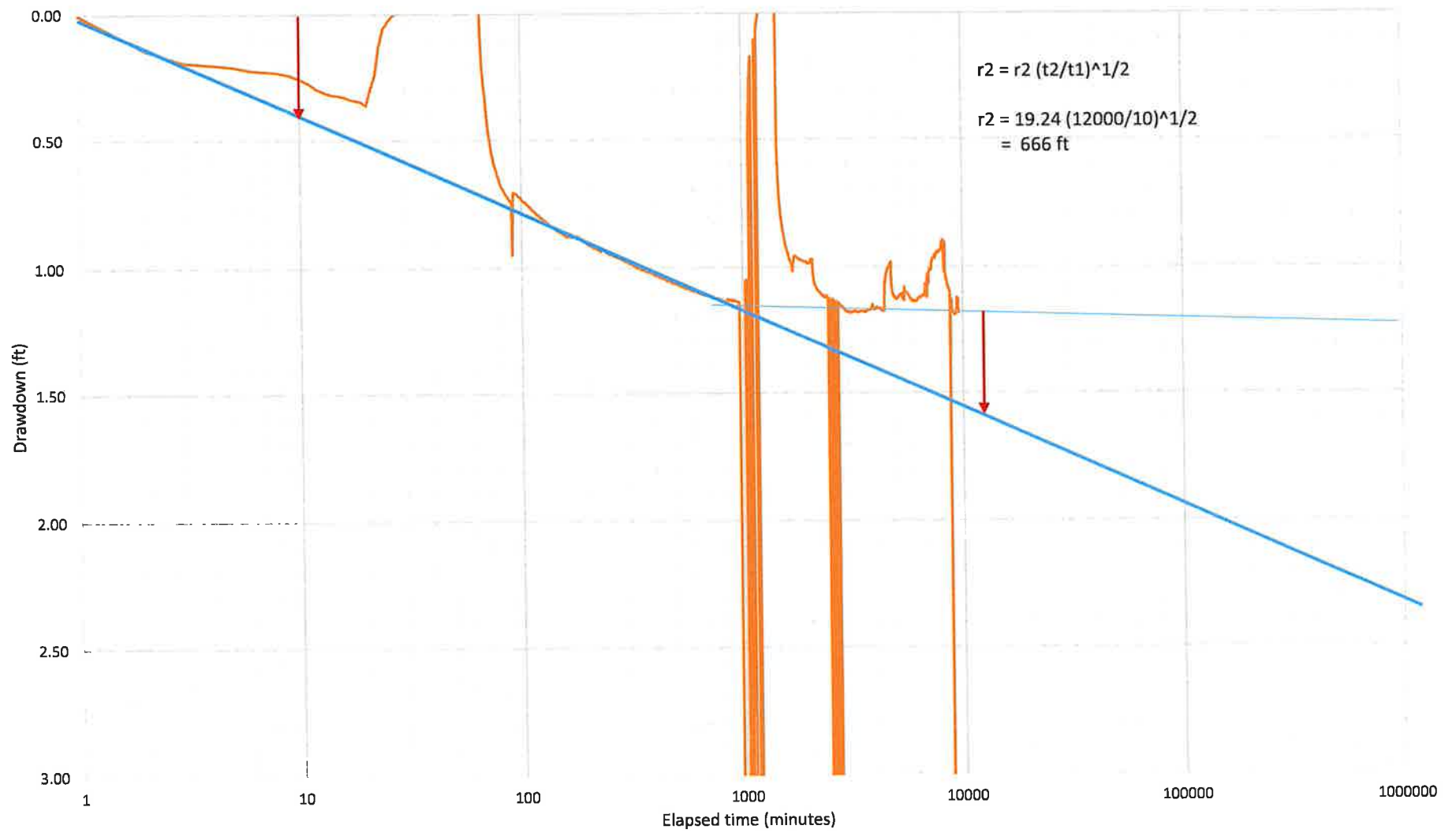


Figure 6.E9

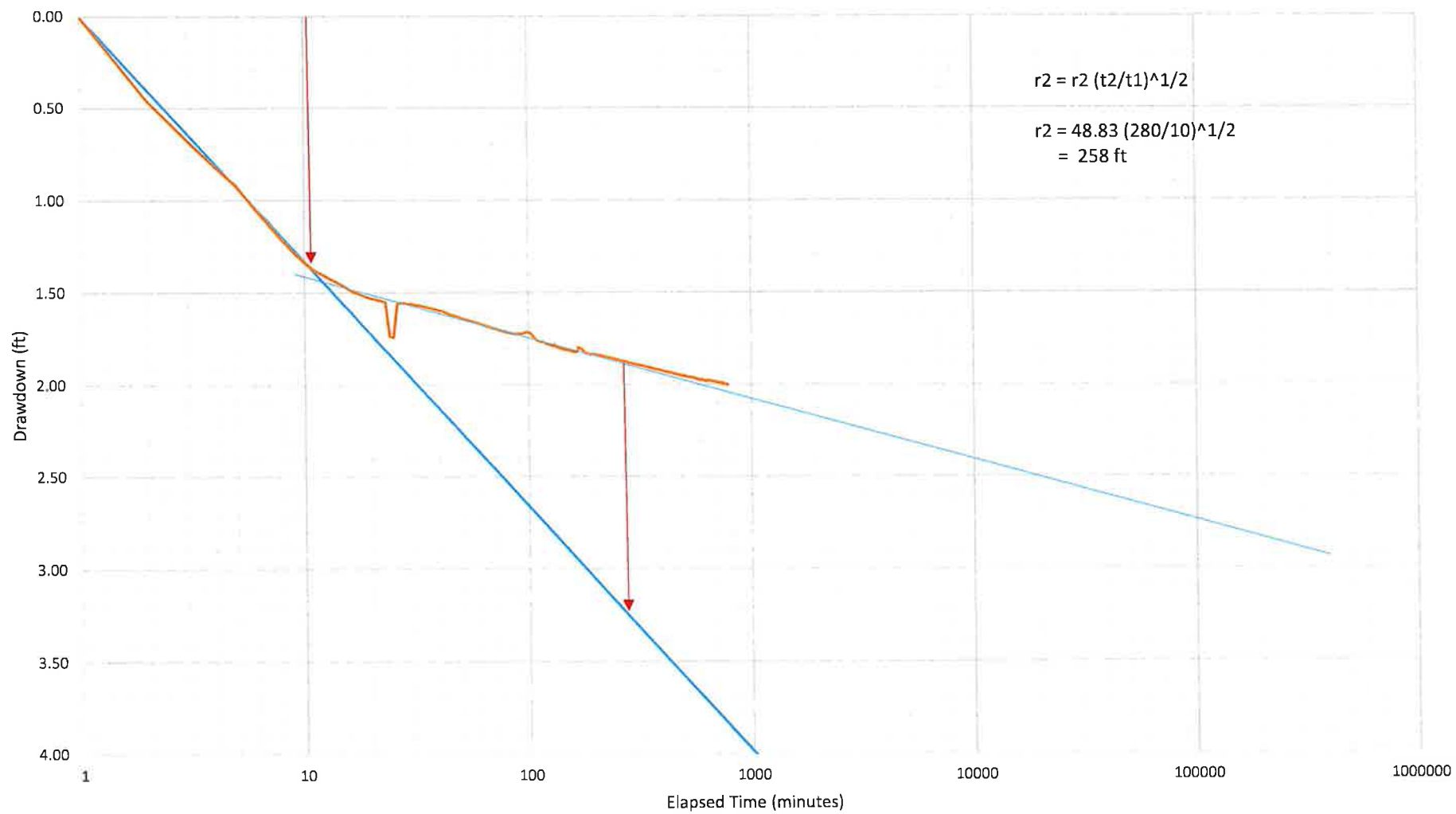
#### 6.5 Hydraulic Testing in Fractured or Low-Permeability Rocks

Advances in hydraulic testing over the past decade have been born out of the practical necessity of evaluating rocks as hosts for chemical or nuclear wastes. In some cases, this requires testing in deep boreholes up to a few thousand feet in depth, often in rocks that are of a low permeability, and frequently in rock that is fractured. In many cases, the emphasis has been on the single-borehole test. These tests generally require special equipment such as packers to isolate testing zones, feed in apparatus whereby a slug of water may be removed from the isolated zone or the zone quickly pressurized, and pressure transducers to measure the response. With deep boreholes, temperature or salinity effects on fluid density must sometimes be accounted for. Hydraulic testing in

P20 Time/Drawdown



P50 Time/Drawdown



# P100 Time/Drawdown

