

MEMO

State of Idaho

Department of Water Resources

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cc: Scott Bergendorf, Senior Water Resource Agent

Subject: Review of Hydrologic Data Analyses Submitted by RMEA in Support of Application for Transfer 80496

Introduction

This memorandum summarizes my review of the report entitled "*Technical Evaluation of Physical Water Source for the Boyd Smith Diversion, 5N, 39E, Section 20 (sic), NE Quarter, Madison County, Idaho*" (Technical Evaluation) dated September 24, 2015. The report was prepared by Rocky Mountain Environmental Associates, Inc. (RMEA) in support of Application for Transfer 80496. Included as an attachment to the Technical Evaluation is a letter addressed to RMEA from Clearwater Geosciences, LLP with the subject line: "*Letter Report on Pumping Test Analysis for the Boyd Smith Well*" (Clearwater Report). The aquifer test is the basis for the stream depletion analysis in the Technical Evaluation so the scope of my review includes the Clearwater Report. Lastly, I reviewed information presented in "*Narrative in Support of Application for Transfer, Liberty Park Irrigation Co. Inc., Water Right Nos. 01-104, 01-105, 01-106, 01-107, 01-108, 01-253, and 01-316*" (Narrative in Support) and a map showing the locations of the existing and proposed points of diversion (PODs), which is labeled "*Attachment #7*" in the Application for Transfer.

Description of Transfer

The existing surface water POD is located on the South Fork of the Snake River (South Fork) and the proposed POD is an existing shallow well¹ located approximately 1.1 miles northeast of the main channel of the South Fork (see Figure 1 in Attachment A). The proposed off-channel POD is located approximately 650 feet northeast of Spring Slough. The proposed transfer involves moving the existing surface water POD for Liberty Park Irrigation Company Inc. about five linear miles in the approximate downstream direction (northwest). The applicant seeks to

¹ As pointed out in the Technical Evaluation, the diversion works for the proposed POD may not meet the statutory definition of a well because it reportedly is only 18 feet deep.

divert a maximum of six cubic feet per second (cfs) at the proposed POD in order to irrigate 300 acres.

The transfer application is the result of an enforcement action that was brought against Boyd Smith on May 13, 2015 for failing to have a water right to divert from his well. The Narrative in Support indicates that Mr. Smith has adequate canal shares in Liberty Park Irrigation Company, Inc. to irrigate the 300-acre parcel and that he used a diesel pump and portable pipeline during the 2015 irrigation season to bring water from Liberty Park Canal to the parcel. Approval of the transfer would allow Mr. Smith to resume using the well.

Surface Water Hydrology

Spring Slough is located between the proposed POD and the main channel of the South Fork (see Figure 2 in Attachment A). The slough channel bifurcates approximately 0.1 miles upstream (southeast) from the point on the slough closest to the proposed POD. The furthest point upstream on the main channel abuts irrigated cropland that is approximately 0.75 miles upstream (southeast) of the proposed POD.

Bannock Jim Slough separates the tributary branches of Spring Slough from the main channel of the South Fork. Spring Slough joins with Bannock Jim Slough approximately 2.5 linear miles (~four river miles) downstream of the proposed POD (see Figure 1 in Attachment A). Bannock Jim Slough enters the Henry's Fork of the Snake River (Henry's Fork) in Section 14 of Township 5N, Range 38E. The confluence with the Henry's Fork is approximately 5 linear miles (~eight river miles) downstream (northwest) of the proposed POD. Based on inspection of aerial photos going back to 1946, there does not appear to be an open channel connection between Spring Slough and the South Fork.

The nearest open channel connection between Liberty Park Canal and Spring Slough is via an unnamed tributary that intersects Spring Slough approximately two river miles downstream (northwest) of the proposed POD (see Figure 2 in Attachment A). During a site visit on December 9, 2015, I verified the existence of a ditch located along the west side of the irrigated parcel that could be used to route water from Liberty Park Canal into the unnamed tributary (see Photo 1 in Attachment B). The connecting ditch is approximately 620 feet long.

There are two pumping stations on the slough at the approximate location closest to the Boyd Smith well (see Photo 2 in Attachment B). These diversions do not have IDWR identification tags but are used to irrigate lands adjacent the slough. Immediately downstream from that location, the water in the slough was approximately six inches deep during the site visit (see Photo 3 in Attachment B). Further downstream, there are two more on-slough diversions in the IDWR spatial database. These diversions are located upstream (southeast) of the confluence with the unnamed tributary to Spring Slough.

During the site visit, the main slough channel was dry upstream of a road that crosses the channel less than 1,000 feet upstream of the point on the slough closest to the Boyd Smith well (see Photo 4 in Attachment B). Abundant cattails and a well defined channel upstream of the road are indication that the slough channel upstream from the road is a wetland during part of the year.

On the day of the site visit, water level measurements that were made by IDWR in the three piezometers used for the Boyd Smith aquifer test indicated that the water table was between 4.7 and 5.1 ft below ground surface at the test site. In combination with field observations, the water level measurements strongly suggest that the water in the slough is sourced from springs that occur where the slough channel intersects the water table.

Description of the Irrigation System

The Boyd Smith well is reportedly 18 feet deep and is equipped with a 200-horsepower vertical turbine pump (see Photo 5 in Attachment B). Prior to 2015, the well was used to irrigate a 300-acre parcel. Water to most of the parcel is delivered through two center pivot sprinkler systems. The parcel is wholly within the Liberty Park Irrigation Company service area. The Liberty Park Canal transects the northeast side of the property (see Figure 2 in Attachment A). At its closest, the Liberty Park Canal is 590 feet from the center of the northern pivot.

According to the IDWR Water Management Information System (WMIS) database, the total system discharge on July 9, 2013 was 2,190.4 gal/min (gpm) while the well was being used to irrigate through both center pivots. It appears likely that both pivots were in operation during the aquifer test because the range of discharge rate estimates provided in the first paragraph of the Clearwater Report is 1,860 to 2,436 gpm. Given its shallow total depth, the Boyd Smith well is highly productive.

Issues for Consideration

At issue is whether the proposed transfer would cause injury to other water rights. Because the proposed POD is a well, special consideration must be given to whether the change from a surface water POD to a groundwater POD constitutes a change in source. According to Department policy, *"the application for transfer... is not approvable unless the ground water and surface sources have a direct and immediate hydraulic connection (at least 50 percent depletion in original source from depletion at proposed point of diversion in one day)."* (IDWR, 2009, p. 26). Moreover, *"The existing point of diversion and proposed point of diversion must be proximate such 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."* (IDWR, 2009, p. 26).

Summary of Technical Evaluation

The Technical Evaluation summarizes hydraulic boundary and stream depletion analyses that were conducted for the proposed POD by RMEA. Although not explicitly stated in the report, the analyses presumably were performed to demonstrate that the proposed groundwater POD has direct and immediate hydraulic connection with the surface source at the original POD. The stream depletion analysis relied on hydraulic property estimates that were developed by Clearwater Geosciences based, in part, on analysis of data from an aquifer test that was conducted by RMEA. The aquifer test involved pumping the proposed POD through a sprinkler system for seven days. The test began on July 28, 2015. Discharge was not measured during the test but rather a range of average rates was estimated based on the total power consumption during the test and the capacity of the sprinkler system. Water levels in the pumping well and three piezometers were monitored using pressure transducers during pumping and also during a seven-day water level recovery period. The aquifer test data analysis was performed by

Clearwater Geosciences using AQTESOLV, a computer program that facilitates the analysis of aquifer test data using a variety of different analytical solutions.

Despite a POD move of approximately 5 linear miles and a potential change in source, possible impacts to other water users are not discussed in the Technical Evaluation, the Clearwater Report, or the Narrative in Support. Although there was no assessment of injury, the potential need for mitigation is acknowledged in the concluding remark of the Technical Report:

“Hydrologically, we conclude that effectively this POD extracts water from the nearby slough, and that delivery of surface water or canal shares to the slough would fully mitigate for any extraction from the POD.”

General Conclusions

To facilitate understanding, the general conclusions of my review have been placed before specific comments concerning the technical submittals. The general conclusions are as follows:

- 1) The existence of springs and a wetland in the upper reaches of Spring Slough is indication 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.
- 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.
- 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.
- 5) The submittals accompanying the subject transfer application do not include an evaluation of potential hydrologic impacts to other water right holders.
- 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.
- 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.

- 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.

Specific Comments

Specific comments based on my review of the technical submittals address the following topics: 1) characterization of the surface source, 2) aquifer testing and data analysis, 3) the RMEA stream depletion analysis, and 4) the RMEA hydraulic boundary analysis.

Characterization of the Surface Source

None of the technical submittals that were prepared in support of the transfer application describe the nature and extent of the slough. This appears to be a significant omission given that the primary thrust of the aquifer test and stream depletion analyses was to demonstrate a direct and immediate hydraulic connection between the proposed POD and Spring Slough.

As previously mentioned, it does not appear based on review of aerial photos going back to 1946 that the slough has an open water connection to the South Fork. Instead, the slough heads in agricultural land less than a mile upstream of the POD and it eventually drains into the Henry's Fork upstream of the confluence with the South Fork.

During the site visit, the slough channel was shallow at the point closest the POD and dry less than 1,000 feet upstream of the proposed POD. At least during the winter, the slough is not a substantial body of water in the vicinity of the POD and this raises questions about whether the slough constitutes an effectively infinite source of recharge to the proposed POD/well and whether existing water users relying on spring water in the slough could be injured by the proposed groundwater extraction.

Aquifer Testing and Data Analysis

As described in the first paragraph of the Clearwater Report, the aquifer test was designed and conducted by RMEA and the data analysis was performed by Clearwater Geosciences.

Aquifer Test Design

According to Stallman (1971, p. 1), "An aquifer test is a controlled field experiment made to determine the hydraulic properties". Moreover, proper design requires that "The geologic and hydraulic setting of the flow system is known in sufficient detail so that the boundary conditions are known explicitly." (Stallman 1971, p. 2). During the design phase, a "careful examination of available information" should be conducted "to decide whether a proposed test site and associated equipment are capable of accurately determining the hydraulic properties of the aquifer and whether the expected results are worth the effort" (Stallman, 1971, p. 2).

Positive elements of the Boyd Smith well aquifer test include the fact that three piezometers were installed and equipped with electronic pressure transducers for measurement of water levels during the pumping and water level recovery periods (see Figure 2 in the Clearwater Report). The non-uniform spacing and varied orientation of the piezometers in relation to the pumping well allows for quantitative analysis of leakage, anisotropy, and hydraulic boundaries. Also, the pump was operated at high capacity (four to five cfs) and it was operated for a relatively long period of time (seven days). Since the stress on the aquifer constitutes a large fraction of the maximum diversion rate for the transfer (six cfs), the measured response provides a good indication of the response to pumping if the transfer was approved.

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

- 1) No provisions were made for measuring the flow rate during the test. The use of either sprinkler system capacity or total power consumption during the pumping period to estimate the average flow rate during an aquifer test is non-standard practice and it significantly reduces the reliability of the test data for determination of aquifer properties and hydrologic boundary conditions.
- 2) The discharge was run through a sprinkler system rather than piped away from the test site. This potentially compromised the aquifer test and the hydraulic property and stream depletion analyses that were based on interpretation of the test data because, after seven days of more-or-less continuous pumping, it is quite possible that a portion of the discharge infiltrated back to the *“very shallow aquifer”* (paragraph 3, Clearwater Report). Wanting to avoid this closed-loop phenomenon is why *“The pumped water should be discharged in a way that prevents its return to the aquifer”* (Kruseman and de Ridder, 1990) and why *“The control well (i.e., pumping well) should be equipped with a pipeline or conveyance structure adequate to transmit the water away from the test site, so that recharge is not induced near the site”* (ASTM D 4050, 1991, Section 7.1.2).
- 3) The test was conducted during the irrigation season. As a result, uncertainty was introduced into the data analysis because the water levels potentially were affected by incidental recharge on adjacent agricultural lands and/or by pumping in nearby irrigation wells.

In summary, it does not appear that the Boyd Smith well aquifer test was designed as a controlled field experiment because there were no provisions for measuring the flow rate during the test or for isolating the measured response to test well pumping from incidental recharge at and near the test site or from pumping in nearby irrigation wells. Conclusions about hydraulic connection with the slough are made uncertain by the fact that the slough was not the only potential recharge source that may have affected the test data.

Site Evaluation

As part of the design process, Stallman (1971, p. 6), recommends that a pre-test site evaluation be conducted. Criteria for the site evaluation include that *“the diameter, depth, and position of all intervals open to the aquifer in the control well (i.e., pumping well) should be known...as*

should total depth" (Stallman, 1971, p. 8). Also, the pumping well *"should be equipped with reliable power, pump and discharge-control equipment"* (Stallman, 1971, p. 8) and the *"Depth to, thickness of, and areal limits of the aquifer should be known"*. Stallman goes on to emphasize *"The importance of making the pretest site evaluation cannot be overstressed. To ignore such an evaluation is to invite failure of the test."*

It appears based upon my review that most of Stallman's pre-test site evaluation criteria were not satisfied and important details concerning the setup of the aquifer test either were not documented or were not verified. For example, the open intervals for the test well were not documented and this information very likely is not known by the data analyst since no well driller's logs were provided in any of the submittals accompanying the transfer. Moreover, the second paragraph of the Technical Evaluation indicates that the pumping well/proposed POD was *"represented by the owner to be installed to a depth of 18 feet below ground surface"*. In other words, there doesn't appear to be any written documentation describing the construction of the pumping well/proposed POD and the total depth was not verified by measurement.

Test Execution

The second paragraph of the Clearwater Report indicates *"the flow rate of the pump was interrupted or altered five times during the pumping test"*. Indeed, inspection of Figure 6 from that 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. Although few data and not much time would have been lost, the test was not restarted after the first outage. The fact that the test wasn't restarted is unfortunate because the hydraulic property estimates that are the basis of the stream depletion analysis were developed by application of analytical solutions for constant discharge aquifer tests. For constant discharge tests, ASTM advises that *"long-term variation of discharge from the beginning to end of test generally should be less than 5%"* (ASTM D 4050, 1991, Section 8.1). Unfortunately, compliance with this guidance cannot be evaluated at other times during the test because no discharge measurements were made during pumping. Failing to make regular discharge measurements and pumping rate adjustments as necessary to approximate a constant discharge rate is contrary to standard practice. As explained in one of the most commonly referenced textbooks on aquifer testing and data analysis, *"the discharge should be measured at least once every hour, and any necessary adjustments made to keep it constant"* (Kruseman and de Ridder, 1990, p. 41). Suffice it to say that fluctuations in the pumping rate reduce the quality of the water level versus time data. Moreover, it appears that *"starts and stops of pumping"* (paragraph 1, Clearwater Report) significantly affected water levels during the pumping period as can be seen by the erratic drawdown pattern that was observed during the pumping period (see Figure 1 in the Clearwater Report).

Documentation

In describing the aquifer test conducted by RMEA, the first paragraph of the Clearwater Report indicates *"This is a very well documented test"*. However, citations are not provided for the different analytical solutions that were employed for hydraulic property determination. Similarly, the source of the barometric pressure data that were used by RMEA to correct the water level data for barometric pressure fluctuations is not identified. And although adjustments were made to the test data to account for water level fluctuations that were *"probably related to*

irrigation pumping" (paragraph 2, Clearwater Report), the locations, completion intervals, and pumping schedules and rates of nearby irrigation wells are not identified or discussed. Also not presented are the data and calculations used to compute the average flow rate, which was 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"* (paragraph 1, Clearwater Report). Similarly, the Technical Evaluation indicates *"The pump was operated for approximately seven days"* but the exact duration is not documented so calculations of the average pumping rates based on the range of estimates for the total pumping volume cannot be verified. Lastly, well logs and information about the hydrogeologic setting are not provided in any of the submittals made on behalf of the applicant. This information is fundamental to aquifer test data analysis and, in accordance with standard practice, should be described in the report documenting the data analysis and used to justify selection of analytical solutions for analyzing the aquifer test data (see, for example, ASTM D 4105, 1991; ASTM D 4106, 1991).

Hydrogeologic Conceptual Model

Without reference to geology and well construction, the data analyst is unable to justify analytical solution selection. In fact, the analyst seems unsure as to whether the aquifer is confined or unconfined based on the following statement: *"the aquifer in this location is very shallow and almost certainly unconfined or leaky confined"* (emphasis added).

The proposed leaky confined aquifer conceptualization is surprising given that the high capacity well is thought to be less than 20 feet deep and the primary conclusion of the Clearwater Report is *"there is a definite connection between surface water and ground water"*. Under normal circumstances, direct connection with shallow surface water would be difficult unless the pumped aquifer was unconfined because a confining layer overlying the pumped aquifer would reduce the hydraulic connection to surface water. In fact, the stream depletion analysis that was performed on behalf of the applicant by RMEA (Jenkins, 1970) assumes the well is pumping from an aquifer that is in direct communication with surface water.

The effects of partially penetrating pumping and observation wells were not evaluated because the thickness of the aquifer is unknown, or at least it is not described in any of the technical submittals. As stated earlier, no review of driller's logs for nearby wells was described in the submittals. However, pumping out of a well that only partially penetrates the full thickness of the aquifer is known to affect drawdown in both the pumped well and observation wells (Kruseman and de Ridder, 1990, p. 159). For aquifer test data from an unconfined aquifer, *"the importance of gravity effects increases as the depth of penetration decreases"* (Neuman, 1974, p. 307). Because the pumping and observation wells are shallow, the lack of an evaluation of the potential impacts of partial penetration on the aquifer test data introduces uncertainty into the conclusions that are based upon analysis of those data.

Initial Data Processing

The second paragraph of the Clearwater Report indicates: *"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"* (emphasis added). While the existence of pumping-

induced fluctuations in the water table is entirely plausible, the use of non-uniform, well-specific corrections for *“regional fluctuations”* is not explained and is somewhat difficult to justify given the close proximity of the wells to one another. For example, P20 is less than 20 feet away from the pumping well but the magnitude of the data correction that was applied for *“regional fluctuations”* at P20 is 1.7 times larger than the correction that was applied to the pumping well data. Also, the correction of 0.3 ft that was applied to the P100 data is not *“small”* in comparison to maximum corrected drawdown of 1.2 ft (see Figure 5 in the Clearwater Report). Since the adjustments are positive, the uncorrected drawdown at P100 presumably was only 0.9 ft and the *“small”* correction was actually 33% of the maximum uncorrected drawdown. This is noteworthy because the slope of the trend of the P100 recovery data was used by the Clearwater analyst to develop the higher of the two transmissivity estimates that were considered by RMEA when determining input for the stream depletion analysis.

The smallest water level adjustment for *“regional fluctuations”* was made for the pumping well, which is in the middle of the monitored well array. For a homogenous and isotropic aquifer, it is difficult to imagine a scenario for which a perturbation in the water table that was caused by irrigation pumping some distance away from the tightly spaced well array impacted the centrally located pumping well appreciably less than the three piezometers that radially surround it (see Figure 2 in the Technical Evaluation). In other words, because of geometric considerations, it would be unexpected for the pumping well location to be a local minimum for regional scale, pumping-induced drawdown. The application of well-specific adjustments to the test data appears to be arbitrary because it is not justified in the documentation.

The second paragraph of the Clearwater Report indicates that the transducer in the pumping well was removed at *“an elapsed time of 9881 minutes”* and, because the transducer *“got hung up and did not return to its original setting”*, a correction of *“0.631 ft was added to the drawdown after elapsed time 9637 minutes to make the recovery match the earlier data.”* It's unclear from the documentation why the 0.631-foot correction was applied to data that were collected between elapsed times of 9,637 and 9,881 minutes since that timeframe preceded removal of the transducer.

Selection of Data for Analysis

Because of discharge variations during the period of pumping, the analyst chose to analyze the data collected during the period of water level recovery. This was a reasonable decision because recovery data are considered *“more reliable than pumping data because recovery occurs at a constant rate”* (Kruseman and de Ridder, 1990, p. 193). However, it's difficult to analyze or interpret the data plots because the data do not appear to have been processed for graphical analysis using either of the two standard methods for analysis of recovery data (i.e., the Theis (1935) residual drawdown recovery method and the calculated recovery method (Brown, 1953)). The analyst chose not to apply the Theis (1935) residual drawdown recovery method, possibly because it precludes estimation of aquifer storage and this is a necessary input for a stream depletion analysis using the method of Jenkins (1970). However, the aquifer storage estimate that was used in the RMEA stream depletion analysis (0.1), though reasonable, is not based on the analysis of test data but rather on the professional judgment of the analyst.

The calculated recovery method (Brown, 1953) requires that the measured recovery data be adjusted “*by extending the drawdown curve beyond the time that pumping stopped, and calculating the difference between the projected water level drawdown and the measured water level*” (Hargis, 1979, p. 15). However, there is no mention in the documentation for the aquifer test analysis of having attempted to make this adjustment and it doesn’t appear that the data were corrected for use of the calculated recovery method prior to analysis.

It could be argued that the drawdown had stabilized at the end of the pumping period and, as a result, the unadjusted recovery data for this data set would be the same as the calculated recovery. However, because the aquifer test was not a controlled field experiment, it is difficult to establish whether stabilization at the end of the pumping period occurred with the available information and, if so, why it occurred. In fact, the analyst offered two possible explanations for “*near steady state drawdown*” at the end of the pumping period (see Figure 6 in the Clearwater Report) and each explanation requires a different hydrogeologic conceptual model. The first explanation is that the pumped well intercepted a recharge boundary (the slough) and the second is that the pumped well began deriving water from a leaky confining layer. However, there are three other possible explanations for near steady-state conditions which are not discussed in the Clearwater Report:

- 1) There likely was a decline in the discharge rate as a result of increased total dynamic head caused by drawdown during the test. This possibility cannot be evaluated because discharge was not measured during the test.
- 2) The pumped water was discharged onto the ground in the vicinity of the pumping well and this water infiltrated through a shallow vadose zone back to the well. This possible explanation also cannot be evaluated because the static depths to water, well construction details, and geology were not documented in any of the submittals.
- 3) The occurrence of incidental recharge on adjacent irrigated lands also may have contributed to near steady state conditions because the test was conducted during the irrigation season. This possibility cannot be evaluated, at least not by an outside reviewer, because the method for correcting water level data to account for “*regional fluctuations*” was not documented.

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 than the calculated recovery method (Brown, 1953).

Assessment of Water Level Recovery

The title of Figure 7 of the Technical Evaluation is “*Semi-log plot of recovery data with lines showing the break in slope ~1,400 minutes indicating full aquifer recovery*”. While most of the water level recovery had occurred at 1,400 minutes, the trend lines for the later period are slightly upward for P20 and P50 suggesting that full aquifer recovery might not yet have occurred. In order to determine whether full recovery has occurred, it’s more common and straightforward to simply compare the recovery water level with the pre-pumping water level. Under normal circumstances, full recovery is indicated when the water level in the well recovers to the pre-pumping level (i.e., the residual drawdown is zero), not when there is a change in

slope. Because the test was conducted during irrigation season, however, it's difficult to know what constitutes full recovery or why the late-time piezometer trends were slightly upward; upward trends would not be expected after full aquifer recovery since corrections were made to compensate for "regional fluctuations".

Figure 6 of the Technical Evaluation shows that the residual drawdown at the end of the recovery period was approximately zero in all wells. This supports the analyst's conclusion that the aquifer fully recovered during the period of monitoring.

A more important question in the context of demonstrating direct and immediate hydraulic connection is whether the aquifer recovered earlier than expected in the absence of the effects of a nearby recharge boundary. Although it was not used for analysis of water level recovery data from the Boyd Smith well aquifer test, the Theis (1935) residual drawdown method facilitates this determination. The method involves plotting residual drawdown versus the logarithm of the ratio time since pumping started (t) divided by the time since pumping stopped (t'). As described in Driscoll (1986, p. 259), *"If the graph indicates zero drawdown at a t/t' value of 2 or more, it is concluded that some recharge water reached the aquifer during the pumping period. The result of the recharge is to bring about full recovery to the original static level during a relatively short recovery period"*.

Selection of Analytical Solutions

The automatic curve matching feature in AQTESOLV was applied by the Clearwater analyst to analyze semi-log and log-log plots of water level recovery versus time. The relative goodness of fit of the type curve matches was used by the Clearwater analyst to select the Cooper and Jacob (1946) confined aquifer straight line method and the Neuman and Witherspoon (1969) leaky confined aquifer solution and to reject the Theis (1935) confined aquifer and Neuman (1974) unconfined aquifer solutions. Derivative analysis, a diagnostic technique for interpretation of aquifer test data that is generally considered to be less subjective than a qualitative goodness of fit assessment, is supported by AQTESOLV but was not applied by the Clearwater analyst.

The Neuman and Witherspoon (1969) solution was developed for a boundary value problem involving a pumped aquifer that is separated from a second aquifer by a confining layer (Attachment D). Although the Clearwater report provides no geologic data to support the existence of a three layer system (two aquifers separated by a confining layer), the analyst feels that its use is justified because *"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 doubt driven by the ability of this solution to yield water from an overlying or underlying aquifer"* (paragraph 3, Clearwater Report). The conclusion that the pumped aquifer might be leaky confined undermines RMEA's effort to demonstrate that the water pumped out of the aquifer at the proposed POD is sourced from the slough rather than from an overlying or underlying aquifer. Moreover, the write-up implies that the leaky confined aquifer analytical solution was selected, and the Neuman (1974) unconfined aquifer solution was rejected, primarily based on the goodness of fit of the type curves rather than on knowledge of the hydrogeology at the test site. Disregard of the hydrogeologic setting is contrary to guidance and can be expected to yield poor hydraulic property estimates.

Confining layer leakage has a different diagnostic signature than a linear recharge boundary and is appropriate for a significantly different hydrogeologic conceptual model. Confining layer leakage generally increases gradually with time (see, for example, Neuman and Witherspoon, 1972, p. 1291) and fully logarithmic plots of drawdown versus time are generally used to evaluate leaky confined aquifer conditions. On the other hand, *"The semilog method is very well suited to the analysis of bounded aquifers"* (Freeze and Cherry, 1979, p. 349) and an abrupt decrease in slope on a semi-logarithmic drawdown versus time plot is diagnostic of a linear recharge boundary (see, for example, Lohman, 1979, p. 61).

As pointed out in Neuman and Witherspoon (1972, p. 1291), the relative impact of confining layer leakage is a function of radial distance from the test/pumping well and *"the closer one is to this well (i.e., the pumping well) the smaller the deviations of drawdown from the Theis curve are"*. Although the recovery data do not appear to have been processed in accordance with standard practice, a relatively good match to the leaky confined analytical solution for piezometers with different radial distances from the pumping well using a single set of parameters suggests that the data may have been affected by an areally-distributed source of recharge and also implies that the aquifer is relatively homogeneous and isotropic at the scale of testing. Since geologic evidence to support the existence of a leaky confining layer is absent, the observed response suggests that irrigating with the pumped well discharge and/or incidental recharge on adjacent irrigated lands may have recharged the aquifer during the test.

In summary, the selection of an analytical solution for a leaky confined aquifer is inconsistent with the conceptualization of a linear recharge boundary which is the basis of the RMEA boundary and stream depletion analyses. The inconsistent and seemingly arbitrary use of analytical solutions for the aquifer test and stream depletion analyses adds uncertainty to the conclusions that were made on behalf of the applicant based on those analyses.

Hydraulic Property Estimation

Using the computer program AQTESOLV, the selected analytical solutions were applied to estimate transmissivity and a range of storage estimates was developed based on the professional judgment of the analyst. Specific comments related to the development of hydraulic property estimates are provided in the indented paragraphs below.

Transmissivity... Straight Line Method

Figure 5 in the Clearwater Report is a semi-logarithmic plot of *"Displacement"* versus *"Adjusted Time, t/r^2 "*. The plot is labeled *"Boyd Smith, Recovery Test Plot"*. The plot was used by the Clearwater analyst to estimate transmissivity using the Cooper and Jacob (1946) straight line method. The ratio t/r^2 on this plot is the recovery time divided by the square of the radial distance from the test/pumping well. The higher of the two transmissivity estimates that were developed using data from the Boyd Smith well aquifer test is based on the slope of the line of best fit that was drawn through the recovery data for P100, a piezometer located 98.79 feet from the pumping well. The t/r^2 value of the last data point is 10^1 (i.e., 10) min/ft². For a radius of 98.79 feet, a t/r^2 value of 10 min/ft² corresponds to an elapsed time of 97,595 minutes, which is approximately 68 days (Calculation 1, Attachment C). However, according to the fourth paragraph of the Technical Evaluation, *"The pump was operated for*

approximately seven days" and *"Data were collected for the pumping period and also a seven-day recovery period"*. These statements cannot be reconciled with an x-axis value that corresponds to an elapsed time of 68 days. As such, it appears that the recovery data on Figure 5 in the Clearwater Report were incorrectly processed and/or plotted and this raises concerns about the validity of the straight line analysis that was used to determine the higher of the two transmissivity estimates that were considered by RMEA when determining input for the stream depletion analysis.

Figure 5 in the Clearwater Report is commonly referred to as a *"composite plot"* (i.e., a plot of drawdown versus t/r^2 for multiple observation wells). In theory, the use of t/r^2 as the x-axis variable normalizes the data from different observation wells so that a single line can be fitted through the entire data set (see, for example, Kruseman and de Ridder, 1990, p. 66). However the data for the different observation wells do not trend along a single line and the analyst decided to only fit the P100 data. The decision to analyze one of the piezometer data trends while ignoring the trends for the other piezometers is not discussed in the Clearwater Report and appears to be arbitrary. Perhaps more importantly, the fact that a single line cannot be fitted through all of the data makes it once again appear that the data that were used to develop the higher of the two transmissivity estimates were not processed and/or plotted correctly.

The third paragraph of the Clearwater Report indicates *"For the Cooper-Jacob straight line method (Figure 5) for a confined aquifer the transmissivity ranges from 1.5 to 2.2×10^5 ft²/day for 1860 and 2436 gpm, respectively."* However, transmissivity is directly proportional to the pumping rate and the ratio of 2,436 to 1,860 is only 1.31, whereas the ratio of 2.2 to 1.5 is 1.47. In other words, the indicated range of transmissivity estimates is too wide for the range of pumping rate estimates.

By back-calculating, I determined that the slope of the line of best fit on Figure 5 in the Clearwater Report is 0.3857 ft/log cycle. The low transmissivity estimate for that slope for a pumping rate 1,860 gpm is 1.699×10^5 ft²/day (Calculation 2, Attachment C). Rounded to two significant digits, the indicated transmissivity range for the straight line method is 1.7×10^5 to 2.2×10^5 ft²/day rather than 1.5×10^5 to 2.2×10^5 ft²/day as reported in the third paragraph of the Clearwater Report.

Transmissivity – Leaky Confined Analytical Solution

Inspection of the upper right hand corner of Figure 3 in the Clearwater Report reveals that AQTESOLV generated two transmissivity estimates for the leaky confined analytical solution of Neuman and Witherspoon, (1969); one labeled "T" and the other labeled "T2". T is the transmissivity of the pumped aquifer in the Neuman and Witherspoon (1969) boundary value problem (see Attachment D) and T2 is the transmissivity estimate for the second aquifer. The T estimate is discussed in the Clearwater Report but the T2 estimate is not. Similarly, the analyst did not report or attempt to interpret the parameters "1/B", " β/r ", and "S2". In my opinion, the analyst was precluded from a more rigorous interpretation of the curve-fitting results because geology and well construction information supporting a leaky confined aquifer conceptualization is lacking.

The transmissivity estimates that were reported based on the leaky confined solution of Neuman and Witherspoon (1969) are $2.02 \times 10^4 \text{ ft}^2/\text{day}$ and $2.9 \times 10^4 \text{ ft}^2/\text{day}$. This range of estimates is too wide for the indicated pumping rate range because the ratio of $2.9/2.02$ is 1.44 and the ratio of pumping rate estimates ($2,436 \text{ gpm}/1,860 \text{ gpm}$) is only 1.31.

Transmissivity – Uncertainty of Estimates

With reference to the automatic curve matching procedure, the Clearwater analyst concludes *“It is difficult to quantify the uncertainty of these calculations because the automated curve matching routine in AQTESOLV failed to converge”* (paragraph 4, Clearwater Report). While the uncertainty of the automatic curve matching-derived transmissivity estimates is difficult to quantify, it is at least an order of magnitude because, as acknowledged by the analyst, the transmissivity estimates obtained with the Neuman and Witherspoon (1969) solution ($2.02 \times 10^4 \text{ ft}^2/\text{day}$ and $2.9 \times 10^4 \text{ ft}^2/\text{day}$) are an order of magnitude lower than the estimates based on application of the Cooper and Jacob (1946) straight line method ($1.5 \times 10^5 \text{ ft}^2/\text{day}$ and $2.2 \times 10^5 \text{ ft}^2/\text{day}$). Even though hydraulic conductivity varies by more than 13 orders of magnitude in natural systems (Freeze and Cherry, 1979, p. 28), an order of magnitude difference is relatively significant for estimates based on analysis of the exact same test data.

As a potential explanation for transmissivity estimate uncertainty, the fourth paragraph of the Clearwater Report indicates *“Problems with the match to the analytical solutions are driven mainly by the presences (sic) of an interfering hydraulic boundary”*. Alternatively, however, the failure to converge could be attributed to the previously described data processing issues. This seems to me a more plausible explanation because it appears water level data that were interpreted by Clearwater to be collected prior to recharge boundary effects (i.e., before the occurrence of a *“near horizontal line”*) were used to determine hydraulic properties with the straight line method. That would explain why the later-time, horizontally trending test data from P100 fall below the line of best fit on Figure 5 in the Clearwater Report.

Although it is uncertain from the documentation, it appears the Clearwater analyst included water level data from the pumping/test well in the curve matching procedure for the fully logarithmic (i.e., log-log) plots of drawdown versus time (Figures 3 and 4 in the Clearwater Report), and this is another possible explanation for the failure to converge. In fact, it is not normal practice to analyze fully logarithmic data plots for the pumping well to determine hydraulic properties. According to the U.S. Bureau of Reclamation, it is often the case that *“Because of head losses inherent in flow to a discharging well, the drawdown in the well may greatly exceed the drawdown in the aquifer immediately adjacent to the well and result in erroneous test data.”* (USBR, 1985, p. 114). In such cases, the Bureau advises *“use of the straight-line solution is preferable because a log-log data plot may be so flat it would be difficult to fit to the type curve”*.

Storage

The Clearwater analyst considers the storage estimate from application of the Neuman and Witherspoon (1969) solution (0.3) to be *“unrealistically high probably by a factor of 5 to 10”* even though the surface geology in the area of the proposed POD is mapped as Quaternary alluvial deposits comprising gravel, sand, and silt (Lewis et al., 2012) and 0.3 is within the normal range of values for gravel or sand (see, for example, Fetter, 1980, p. 68; Driscoll,

1986, p. 67). Based on professional judgment, the analyst instead concluded that a storativity of 0.0001 is *“more realistic”* and *“Based on these types of materials, the anticipated range of storativity is between 0.0001 to 0.1”* (paragraph 4, Clearwater Report). This conclusion is lacking foundation since, other than the cryptic phrase *“these types of materials”*, it was made without reference to the geologic setting or the hydrogeologic literature. Note also that a *“more realistic”* value of 0.0001 is more than two orders of magnitude less than the data-analysis derived estimate of 0.3 divided by *“a factor of 5 to 10”* (i.e., 0.03 to 0.06).

The term *“storativity”* is a dimensionless term that is generally used to describe the storage of a confined aquifer. Confined aquifer storage values typically range from 5×10^{-5} to 5×10^{-3} (Freeze and Cherry, 1979, p. 60; Kruseman and de Ridder, 1990, p. 23). The storage term for an unconfined aquifer is known as the *“specific yield”*. According to Freeze and Cherry (1979, p. 61), *“The specific yields of unconfined aquifers are much higher than the storativities of confined aquifers”*. Specific yield usually ranges from 0.01 to 0.3 (Freeze and Cherry, 1979, p. 61; Kruseman and de Ridder, 1990, p.23). According to the fourth paragraph in the Clearwater Report, however, *“the anticipated range of storativity is between 0.0001 and 0.1.”* Use of the terminology *“range of storativity”* in the Clearwater Report is confusing because the range 0.0001 to 0.1 spans the non-overlapping ranges of storage that are typical for confined and unconfined aquifers.

In describing the three order of magnitude range in storage estimates, the Clearwater analyst explains *“The wide range is driven by the natural variation in aquifer confinement conditions in this area”* (paragraph 4, Clearwater Report). In the next paragraph, the analyst concludes *“The cone of depression from the well is established very quickly and draws water from the slough”*. While these statements are not strictly incompatible, aquifer confinement would be expected to delay the hydraulic connection between the well and slough. Moreover, geologic evidence to support the existence of variable aquifer confinement conditions is not presented in any of the technical submittals.

Although not discussed in the Clearwater Report, the estimate of storage generated by AQTESOLV using the Cooper and Jacob (1946) straight line method is 0.482 (see Figure 5 in the Clearwater Report). As mentioned above, this value is higher than typical for unconfined aquifers and orders of magnitude higher than typical for confined aquifers.

RMEA Stream Depletion Analysis

RMEA used the upper end of Clearwater’s storage estimates (0.1) for the Jenkins (1970) stream depletion analysis. The Technical Evaluation indicates *“Using a larger storage coefficient is conservative, producing a longer capture time...”* While this is generally a true statement, it is not a valid conclusion in this case because 0.1 is appreciably less than the aquifer test-derived storage estimates of 0.3 (Figure 3 of Clearwater Report) and 0.482 (Figure 5 of Clearwater Report).

The RMEA-computed stream depletion ratio after 24 hours of pumping is 56% using the arithmetic average of Clearwater’s transmissivity estimates (1.2×10^5 ft²/day) and a specific yield of 0.1. I verified this value using the assumed input (Calculation 3, Attachment C). However, using a specific yield of 0.3, which is the only Clearwater estimate that is both based

on quantitative analysis of the test data and also within the typical range of specific yields for gravel or sand aquifers, reduces the 24-hour depletion ratio to 32% (Calculation 4, Attachment C). Even if the slough was the original source, this 24-hour depletion ratio fails to satisfy the direct and immediate hydraulic connection criterion.

As described previously, there appears to be several problems with the processing of the data that were used by the Clearwater analyst to develop the high transmissivity estimate of 2.2×10^5 ft²/day. The use of the low transmissivity estimate for the stream depletion analysis with all other input unchanged results in a stream depletion ratio of only 16% after 24 hours (Calculation 5, Attachment C). Even if the slough was the original source, this 24-hour depletion ratio fails to satisfy the direct and immediate hydraulic connection criterion.

Whereas the high end of the transmissivity range (2.2×10^5 ft²/day) was computed using the higher of two pumping rate estimates, the low end of the transmissivity range (2.0×10^4) was computed with the lower pumping rate estimate. Because the low transmissivity estimate is an order of magnitude less than the high estimate, a percent variation in the low estimate has little effect on the magnitude of the arithmetic average but the same percent variation in the high transmissivity estimate has a much larger impact on the arithmetic average. The impact of using the low pumping rate estimate to compute the low transmissivity estimate while using the high pumping rate estimate to compute the high transmissivity estimate instead of using the arithmetic average pumping rate for both was to increase the arithmetic average transmissivity by 14.5% (Calculation 6, Attachment C). The 24-hour stream depletion ratio with the average transmissivity for the arithmetic average pumping rate is 53.6% (Calculation 7, Attachment C). This depletion ratio is 2.7% less than computed by RMEA (56.3%).

In combination with the use of arithmetic averaging, the existence of uncertainty in the pumping rate increased the stream depletion ratio above what it would be for the expected/average pumping rate estimate. In my opinion, the existence of uncertainty that was the direct result of not making accurate field measurements in accordance with standard practice should not be used to make a more favorable case on behalf of the applicant.

RMEA Hydraulic Boundary Analysis

The second-to-last attachment to the Technical Evaluation is a hydraulic boundary analysis that was performed by RMEA. The graphical method of analysis is based on the principle of superposition (Freeze and Cherry, 1972, p. 328) and is informally referred to as "*the method of times*". Although the analysis was not described in the Technical Evaluation, RMEA provided a photocopy of a sample problem from a text book and posted their calculations on the data plots. The unstated hypothesis of the RMEA boundary analysis is that the slough approximates the hydraulic functioning of a linear recharge boundary. As recognized by the RMEA analyst, the boundary analysis is directly relevant to the transfer application. As described below, however, there are inconsistencies both within the boundary analysis and between the boundary analysis and the aquifer test analysis documented in the Clearwater Report.

Inconsistencies within the Hydraulic Boundary Analysis

There are several internal inconsistencies in the RMEA boundary analysis:

- 1) The boundary analysis plot for P20 includes drawdown data ranging in elapsed time from one to approximately 10,000 minutes, which is roughly seven days and corresponds to the duration of the pumping period. However, the maximum elapsed time on the P50 and P100 boundary analysis plots is only 800 minutes, which is less than one day. The different durations are not documented.
- 2) The boundary analysis plot for P20 shows the same response to the suspected pump outage between 20 minutes and 65 minutes (i.e., full water level recovery to zero drawdown) 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.
- 3) The RMEA analyst interpreted a break/decrease in slope at approximately 900 minutes on the P20 plot and a break/decrease in slope between 10 and 20 minutes on both the P50 and P100 plots. According to the theory upon which the method of times was developed, the break/decrease in slope represents the effect on the drawdown data of the cone of depression intercepting a recharge boundary. Because the distances from each of the piezometers to the slough are not substantially different, ranging from approximately 630 feet to 740 feet (see Figure 2 in the Technical Evaluation), and because P20 is the closest piezometer to the slough, it is difficult to explain why the recharge boundary would not affect the drawdown trend in P20 until 900 minutes when, based on the RMEA analyst's interpretation, it impacted the drawdown trends in P50 and P100 in less than 20 minutes. Moreover, the computed distance from P20 to the slough based on the boundary analysis (666 ft) is significantly greater than the computed distance for P50 (258 ft).

As a result of these discrepancies, the hydraulic boundary analysis is not compelling support for the hypothesis that the slough approximates the hydraulic functioning of a linear recharge boundary.

Inconsistencies between the Hydraulic Boundary and Aquifer Test Data Analyses

There are also several inconsistencies between the hydraulic boundary analysis performed by RMEA and the aquifer test analysis performed by Clearwater:

- 1) 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*" (paragraph 2, Clearwater Report).
- 2) Unlike all figures in the Clearwater Report, individual data points are not discernible on the boundary analysis plots prepared by RMEA.

- 3) 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 measurable drawdown in both of those piezometers at one minute.
- 4) 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.
- 5) As previously mentioned, complete water level recovery between 20 minutes and 65 minutes is not seen on the boundary analysis plots for P50 and P100 even though it is quite obvious for all four wells on Figure 6 in the Clearwater Report. The fact that the smooth and continuously increasing drawdown trends shown on the P50 and P100 data plots prepared by RMEA do not agree with the information presented in the Clearwater Report is a concern to me because it appears that the P50 and P100 drawdown data used in the RMEA boundary analysis are from a different, undocumented aquifer test. Additional evidence supporting the hypothesis that RMEA used data from a different aquifer test includes the following:
 - a) Whereas the aquifer test documented in the Clearwater Report involved 7 days of pumping, the maximum elapsed time on the boundary analysis plots for P50 and P100 is only 800 minutes, which is less than one day.
 - b) As would be expected, Figures 5 and 6 in the Clearwater report indicate that the total drawdown generally increases as a function of elapsed time and decreases as a function of distance from the pumping well. Comparison of the boundary analysis plots for P20 and P50, however, indicates greater drawdown in P50 after only 800 minutes of pumping (2.0 feet) than measured in P20 after seven days of pumping (1.2 feet).
 - c) The boundary analysis plot for P100 includes a notation detailing the removal and replacement of a transducer at an elapsed pumping time between 20 and 30 minutes. Although the second paragraph of the Clearwater Report details the impact of transducer removal and reinstallation in the pumping well at an elapsed time of 9,881 minutes, there is no mention of transducer removal and replacement in P100.
- 6) 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 in slope 700 minutes after pumping began (see the title of Figure 6 in the Clearwater Report) and 1,400 minutes after recovery began (see the title of Figure 7 in the Clearwater Report). Inspection of Figure 5 in the Clearwater Report reveals that the Clearwater analyst used all but the “near steady state” recovery data to determine aquifer properties using the Cooper and Jacob (1946) method. In other words, it appears likely that the Clearwater analyst utilized the P100 recovery data trend out to 1,400 minutes for determination of hydraulic properties using the Jacob and Cooper (1946) method, whereas RMEA interpreted the P100 drawdown data after approximately 15 minutes as having already been affected by a recharge boundary. RMEA’s interpretation of an early boundary on the P100 data plot is potentially significant in the context of hydraulic property estimation because “*The early recovery data that define the first straight-line segment,*

should reflect the aquifer transmissivity” and the use of data already affected by a boundary “would yield an erroneous value of transmissivity” (Hargis, 1979, p. 82). Despite what appears to be significantly different interpretations of aquifer test data from the same piezometer, the Clearwater analyst summarizes his review of the RMEA boundary analysis in the fifth paragraph by concluding “I reviewed your calculations and found them to be accurate.”

In conclusion, it appears: 1) RMEA’s hydraulic boundary analysis relied, at least in part, on data from a different aquifer test than the one that is described in the technical submittals and, 2) despite the Clearwater analyst’s endorsement of the boundary analysis, Clearwater’s use of later-time data from P100 to estimate transmissivity appears to be in conflict with RMEA’s interpretation of an almost immediate hydraulic connection with the slough based on analysis of data from the same piezometer. The submittal of an analysis that appears to involve undocumented test data is confusing and, in my opinion, detracts from the technical merit of the submittals accompanying the transfer application, as does Clearwater’s failure to recognize, acknowledge, and attempt to explain the different interpretations. The existence of significant data and interpretation discrepancies casts doubt on both the RMEA hydraulic boundary analysis and the Clearwater aquifer test data analysis.

Application of the Straight Line Method

The RMEA boundary analysis, as well as the Clearwater analysis that was used to develop the high transmissivity estimate, involved fitting straight lines through semi-logarithmic plots of water level versus time. The straight line method (a.k.a., the modified Theis non-equilibrium method) is based on the truncated infinite series approximation of the Theis (1935) non-equilibrium well equation. The truncated infinite series approximation is only valid after sufficient time has elapsed for the so-called “*u-criterion*” to be met (see, for example, Driscoll, 1986, p. 220; USBR, 1985, p. 112). In addition to elapsed time, the parameter *u* is a function of the hydraulic properties (transmissivity and storage) and the radial distance from the pumped well. For strict application of the straight line method, *u* must be less than 0.01 (see, for example, USBR, 1985, p. 112) but a less-stringent value of 0.05 can also yield acceptable results (Driscoll, 1986, p. 220). From the documentation, it’s unclear whether the Clearwater analyst or the RMEA analyst evaluated when the truncated infinite series approximation of the Theis (1935) equation became valid.

Using the hydraulic properties that were used by RMEA as input for the Jenkins (1970) stream depletion analysis (see page 2 of the Technical Evaluation), the less stringent *u-criterion* ($u < 0.05$) was not satisfied for any of the data falling on the early line of best fit on the P100 boundary analysis plot and also not satisfied for most, if not all, of the data falling on the early line of best fit on the P50 boundary analysis plot (Calculation 8, Attachment C). This undermines the credibility of the RMEA hydraulic boundary analysis because the truncated infinite series approximation is “*not applicable*” when the value of *u* is larger than 0.05 (Driscoll, 1986, p. 220).

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ATTACHMENT A

MAPS

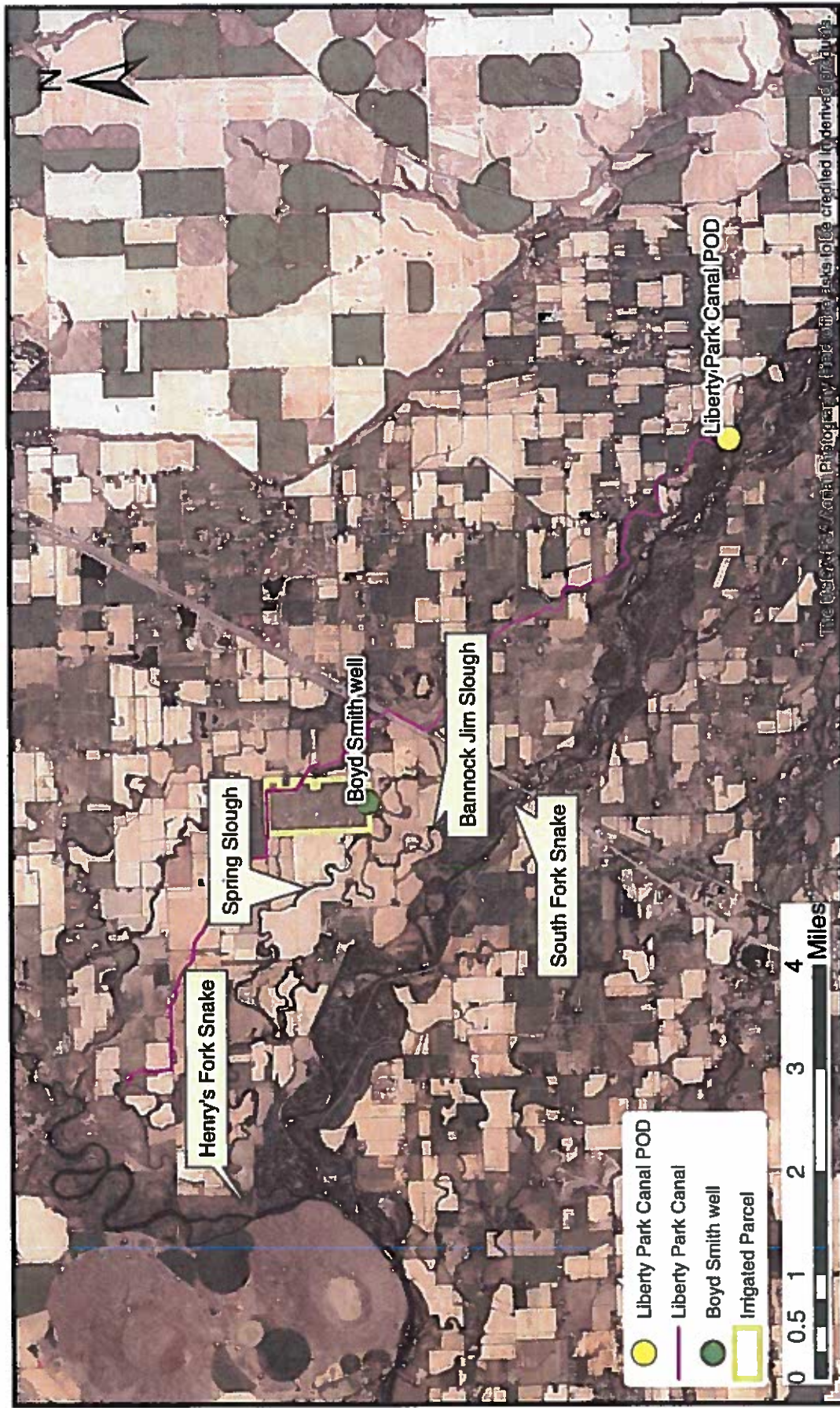


Figure 1 - Location Map (2013 Aerial Photography)

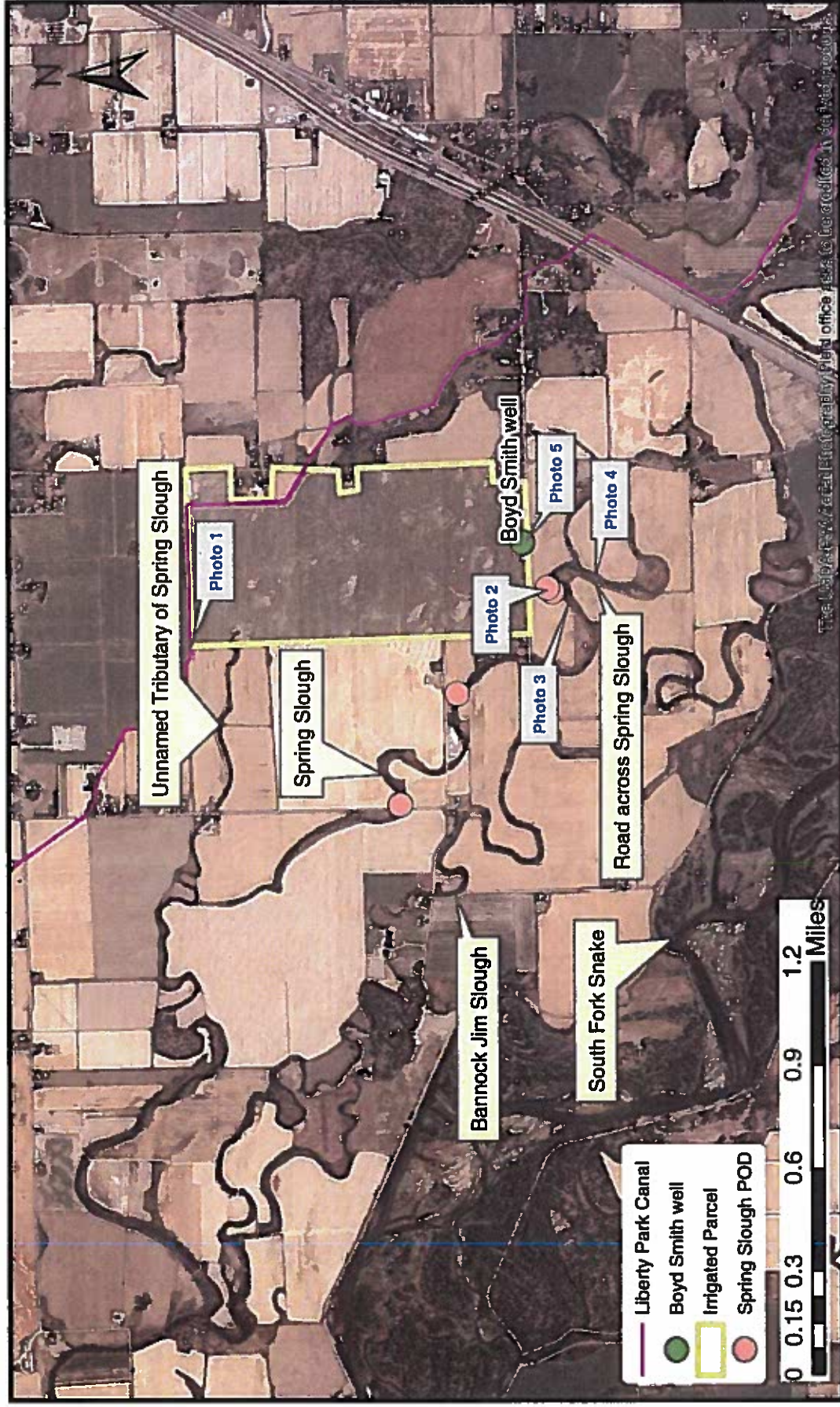


Figure 2 - Spring Slough PODs in Vicinity of Boyd Smith Well

ATTACHMENT B

PHOTOGRAPHS FROM 12/9/2015 SITE VISIT



Photo 1. View looking south of ditch connecting Liberty Park Canal to unnamed tributary.



Photo 2. Points of diversion on Spring Slough near the Boyd Smith well.

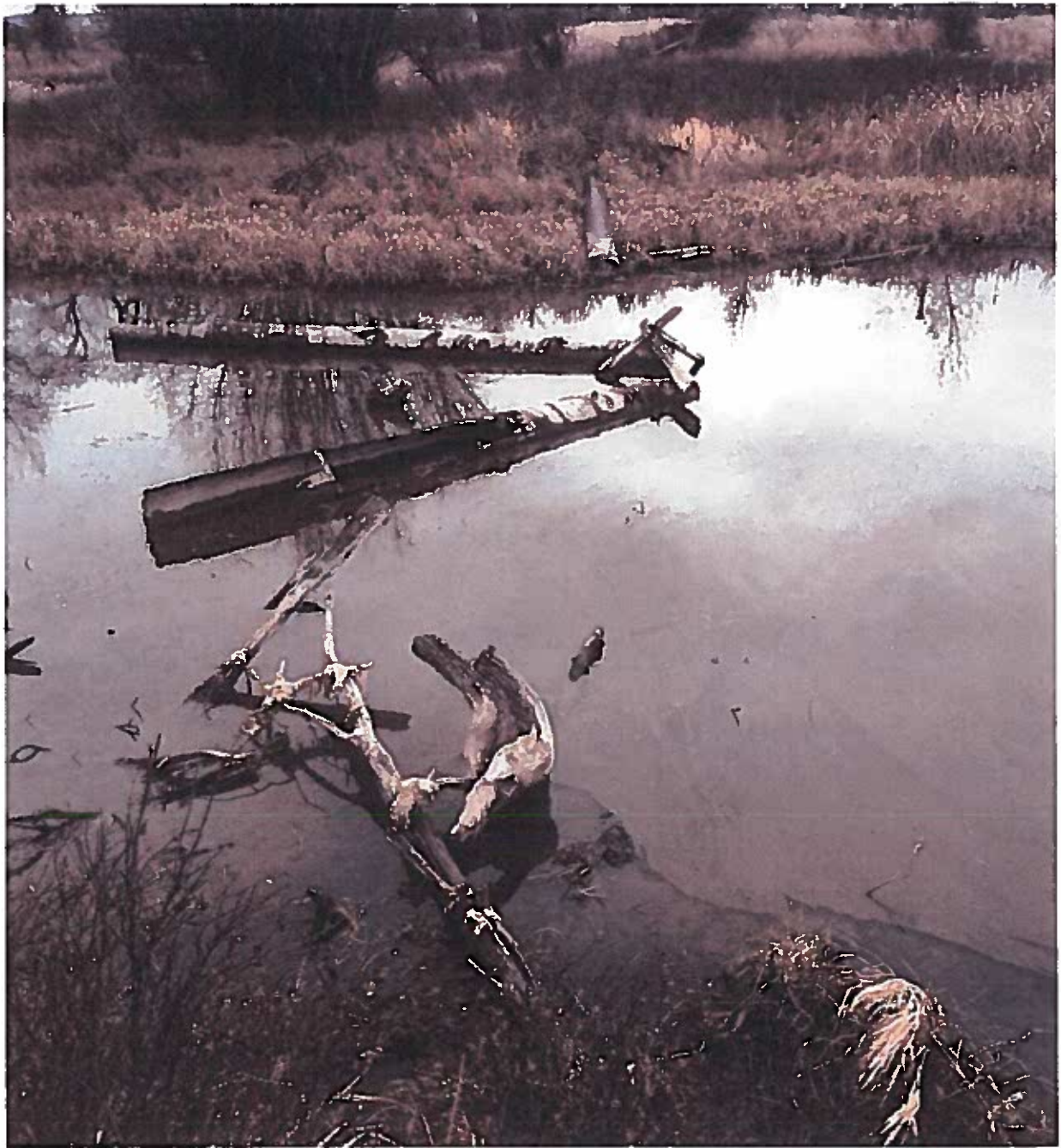


Photo 3. Spring Slough near the Boyd Smith well.

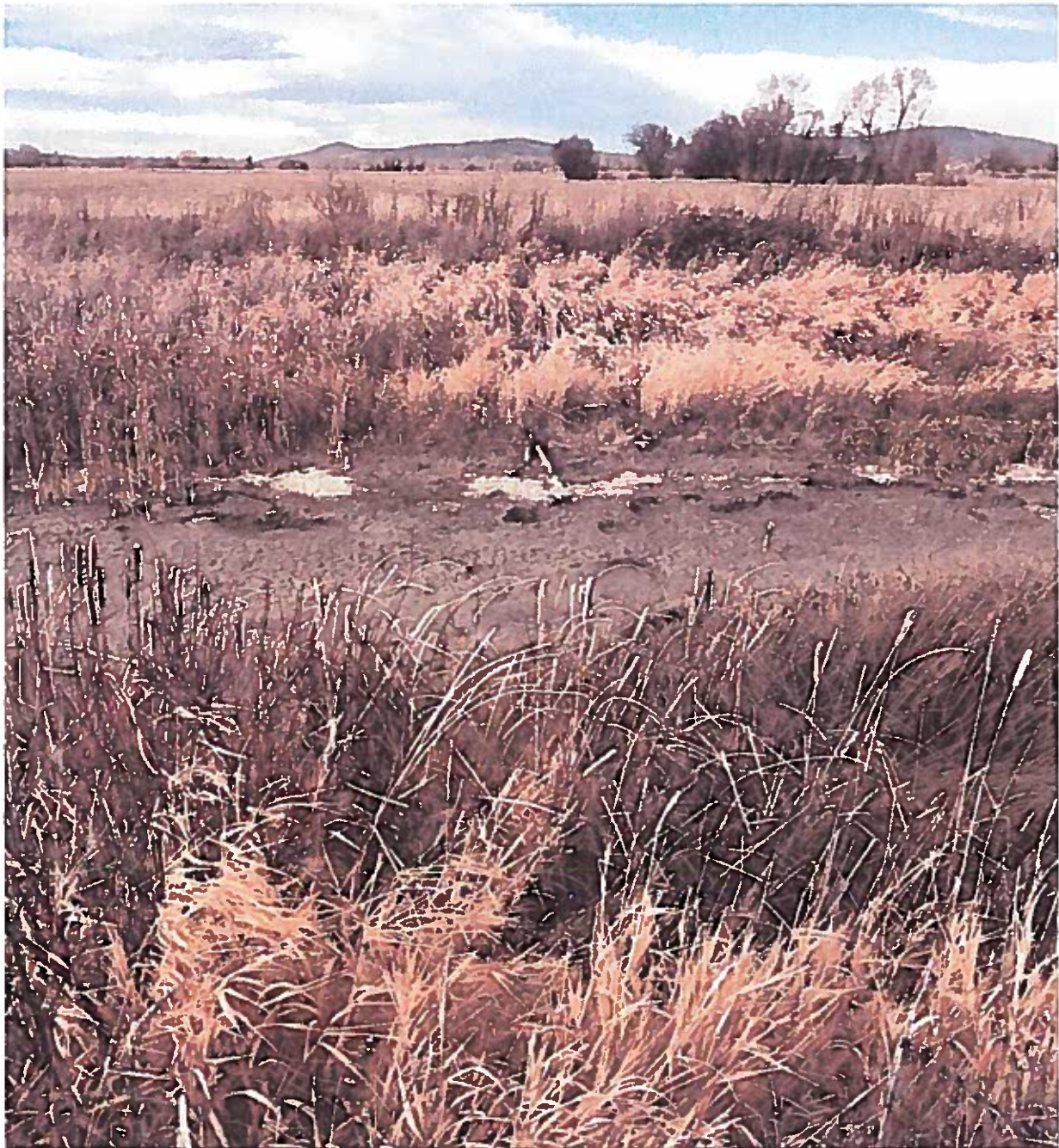


Photo 4. Dry channel on the upstream side of road across Spring Slough.



Photo 5. Electric motor on Boyd Smith well.

ATTACHMENT C

CALCULATIONS

Given:

$$r_{100} := 98.79 \cdot ft$$

...radial distance to P100 from test well

$$t_{over_r2} := 10^1 \cdot \frac{min}{ft^2}$$

...t/r2 value at end of recovery period on Figure 5 in Clearwater Report

Solve for time (t):

$$t := t_{over_r2} \cdot r_{100}^2$$

$$t = 97594.6 \text{ min}$$

$$t = 67.8 \text{ day}$$

...time corresponding to the last t/r2 value for P100

Given:

$$gpm := \frac{gal}{min}$$

$$Q_{low} := 1860 \cdot gpm \quad \dots \text{low pumping rate estimate from Clearwater Report}$$

$$Q_{high} := 2436 \cdot gpm \quad \dots \text{high pumping rate estimate from Clearwater Report}$$

$$\frac{Q_{high}}{Q_{low}} = 1.31 \quad \dots \text{ratio of high to low pumping rate estimate from Clearwater Report}$$

$$T_{high} := 2.225 \cdot 10^5 \cdot \frac{ft^2}{day} \quad \dots \text{high transmissivity estimate from Figure 5 of Clearwater Report}$$

Solve for slope:

$$T = \frac{2.3 \cdot Q}{4 \cdot \pi \cdot \Delta s} \quad \dots \text{Cooper and Jacob (1946) equation for transmissivity}$$

$$\Delta s := \frac{2.3 \cdot Q_{high}}{4 \cdot \pi \cdot T_{high}} \quad \dots \text{rearrange equation to solve for slope}$$

$$\Delta s = 0.3857 \text{ ft} \quad \dots \text{slope of line of best fit on Figure 5 in Clearwater Report}$$

Solve for low transmissivity estimate:

$$T_{low} := \frac{2.3 \cdot Q_{low}}{4 \cdot \pi \cdot \Delta s} \quad \dots \text{Cooper and Jacob (1946) equation for transmissivity}$$

$$T_{low} = (1.699 \cdot 10^5) \frac{ft^2}{day} \quad \dots \text{transmissivity estimate for low pumping rate estimate to 4 significant digits}$$

$$T_{low} = (1.7 \cdot 10^5) \frac{ft^2}{day} \quad \dots \text{transmissivity estimate for low pumping rate estimate to 2 significant digits}$$

Verity ratios:

$$\frac{Q_{high}}{Q_{low}} = 1.31 \quad \frac{T_{high}}{T_{low}} = 1.31 \quad \frac{2.22}{1.7} = 1.31$$

Given:

$a := 895 \cdot ft$...perpendicular distance from well to stream

$S := 0.1$...high end of Clearwater specific yield range

$T := 1.2 \cdot 10^5 \cdot \frac{ft^2}{day}$...arithmetic average transmissivity from Clearwater Report

$b := 18 \cdot ft$...assume aquifer thickness is equal to landowner reported pumping well depth

$K := \frac{T}{b}$ $K = 2.35 \frac{cm}{s}$...hydraulic conductivity for assumed aquifer thickness (representative of a gravel aquifer - see Freeze and Cherry, p. 29)

Apply Jenkins (1970) method to compute stream depletion ratio after 24 hours of pumping:

$sdf := \frac{a^2 \cdot S}{T}$...stream depletion factor

$i := 1 \dots 24$...hourly index variable

$tsdf(i) := \frac{i \cdot 1 \cdot hr}{sdf}$...t/sdf equation

$tsdf(i) =$ [0.06242]
[0.12484]
[0.18726]
[0.24968]
[0.3121]
[0.37452]
[0.43694]
[0.49936]
[0.56178]
[0.6242]
[0.68662]
[0.74904]
[0.81146]
[0.87388]
[0.9363]
[0.99872]
[1.06114]
[1.12356]
[1.18598]
[1.2484]
[1.31082]
[1.37324]
[1.43566]
[1.49808]

...hourly t/sdf values

t_{over_sdf}	q_{over_Q}
0.07	0.006
0.1	0.025
0.15	0.068
0.2	0.114
0.25	0.157
0.3	0.197
0.35	0.232
0.4	0.264
0.45	0.292
0.5	0.317
0.55	0.34
0.6	0.361
0.65	0.38
0.7	0.398
0.75	0.414
0.8	0.429
0.85	0.443
0.9	0.456
0.95	0.468
1	0.48
1.1	0.5
1.2	0.519
1.3	0.535
1.4	0.55
1.5	0.564
1.6	0.576
1.7	0.586

...table of t/sdf and q/Q values from Table 1 in Jenkins (1970)

$frac(i) := \text{interp}(t_over_sdf, q_over_Q, tsdf(i))$

$frac(i) =$

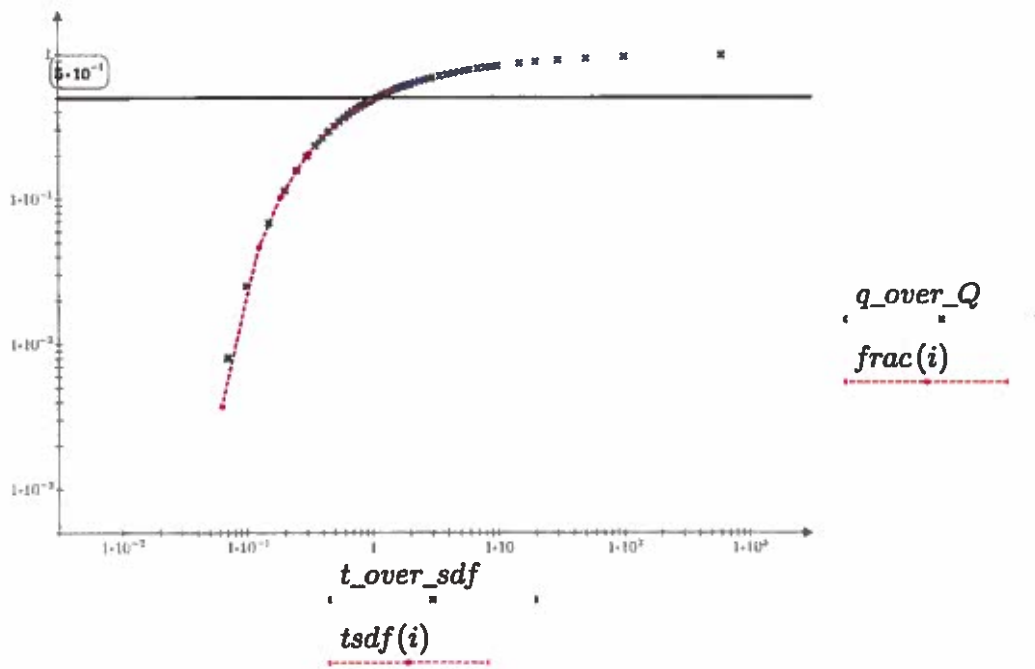
0.0037
0.04636
0.10228
0.15672
0.20547
0.24769
0.28469
0.31668
0.34495
0.3702
0.39318
0.41369
0.43221
0.44921
0.46471
0.47969
0.49223
0.50448
0.51634
0.52674
0.53662
0.54599
0.55499
0.56373

...ratio of stream depletion
rate to pumping rate

1.8	0.598
1.9	0.606
2	0.617
2.3	0.634
2.4	0.648
2.6	0.661
2.8	0.673
3	0.683
3.5	0.705
4	0.724
4.5	0.739
5	0.752
5.5	0.763
6	0.773
7	0.789
8	0.803
9	0.814
10	0.823
15	0.855
20	0.874
30	0.897
50	0.92
100	0.944
500	0.977

$frac(24) = 56.37\%$

...ratio of stream depletion rate
to pumping rate at 24 hours



Given:

$a := 895 \cdot ft$...perpendicular distance from well to stream

$S := 0.3$...specific yield based on analysis of aquifer test data

$T := 1.2 \cdot 10^5 \cdot \frac{ft^2}{day}$...arithmetic average transmissivity from Clearwater Report

$b := 18 \cdot ft$...assume aquifer thickness is equal to landowner reported pumping well depth

$K := \frac{T}{b}$ $K = 2.35 \frac{cm}{s}$...hydraulic conductivity for assumed aquifer thickness (representative of a gravel aquifer - see Freeze and Cherry, p. 29)

Apply Jenkins (1970) method to compute stream depletion ratio after 24 hours of pumping:

$sdf := \frac{a^2 \cdot S}{T}$...stream depletion factor

$i := 1 \dots 24$...hourly index variable

$tsdf(i) := \frac{i \cdot 1 \cdot hr}{sdf}$...t/sdf equation

$tsdf(i) =$

0.02081
0.04161
0.06242
0.08323
0.10403
0.12484
0.14565
0.16645
0.18726
0.20807
0.22887
0.24968
0.27049
0.29129
0.3121
0.33291
0.35371
0.37452
0.39533
0.41613
0.43694
0.45775
0.47855
0.49936

 ...hourly t/sdf values

t_over_sdf	q_over_Q
0.07	0.008
0.1	0.023
0.15	0.068
0.2	0.114
0.25	0.157
0.3	0.197
0.35	0.232
0.4	0.264
0.45	0.292
0.5	0.317
0.55	0.34
0.6	0.361
0.65	0.38
0.7	0.398
0.75	0.414
0.8	0.429
0.85	0.443
0.9	0.456
0.95	0.468
1	0.48
1.1	0.5
1.2	0.519
1.3	0.533
1.4	0.55
1.5	0.564
1.6	0.578
1.7	0.588

...table of t/sdf and q/Q values from Table 1 in Jenkins (1970)

$frac(i) := \text{linterp}(t_over_sdf, q_over_Q, tsdf(i))$

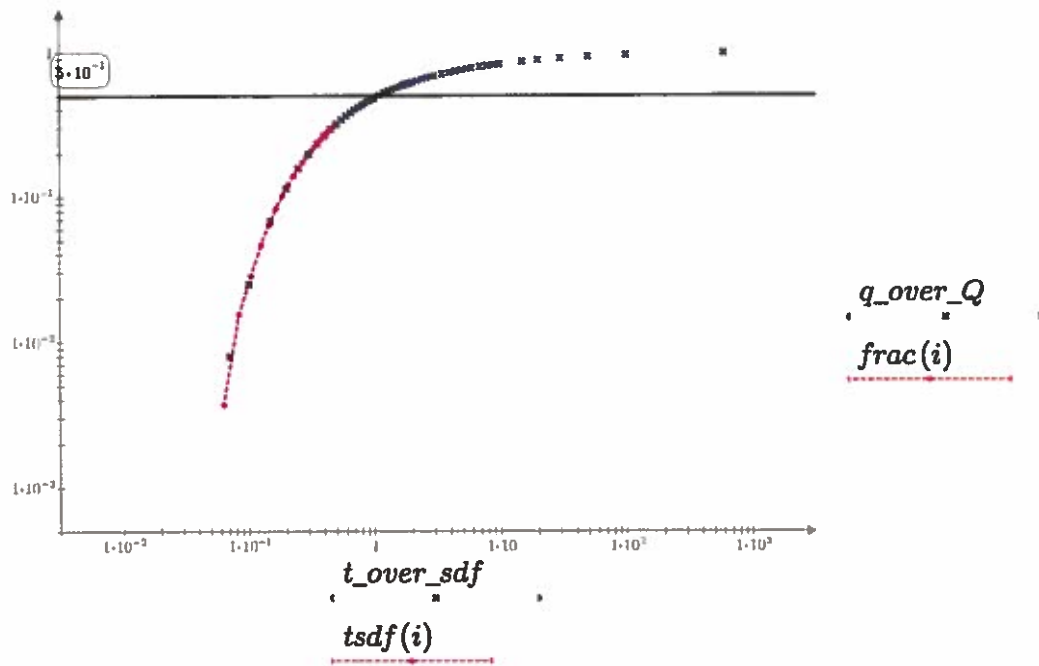
$frac(i) =$

-0.01988
-0.00809
0.0037
0.0155
0.02847
0.04636
0.06426
0.08314
0.10228
0.12094
0.13883
0.15672
0.17339
0.19003
0.20547
0.22003
0.23438
0.24769
0.26101
0.27303
0.28469
0.29587
0.30628
0.31668

...ratio of stream depletion
rate to pumping rate

1.5	0.895
1.9	0.908
2	0.917
2.2	0.934
2.4	0.948
2.6	0.961
2.8	0.973
3	0.983
3.5	0.995
4	0.999
4.5	0.999
5	0.999
5.5	0.999
6	0.999
7	0.999
8	0.999
9	0.999
10	0.999
15	0.999
20	0.999
30	0.999
50	0.999
100	0.999
600	0.999

$frac(24) = 31.7\%$...ratio of stream depletion
rate to pumping rate at 24 hours



Given:

$a := 895 \cdot ft$...perpendicular distance from well to stream

$S := 0.1$...specific yield

$T := 2 \cdot 10^4 \cdot \frac{ft^2}{day}$...low end of transmissivity estimate
range in Clearwater Report

$b := 18 \cdot ft$...assume aquifer thickness is equal to
landowner reported pumping well depth

$K := \frac{T}{b}$ $K = 0.39 \frac{cm}{s}$...hydraulic conductivity for assumed
aquifer thickness (representative of a gravel
aquifer - see Freeze and Cherry, p. 29)

Apply Jenkins (1970) method to compute stream depletion ratio after 24 hours of pumping:

$sdf := \frac{a^2 \cdot S}{T}$...stream depletion factor

$i := 1 \dots 24$...hourly index variable

$tsdf(i) := \frac{i \cdot 1 \cdot hr}{sdf}$...t/sdf equation

$tsdf(i) =$

0.0104
0.02081
0.03121
0.04161
0.05202
0.06242
0.07282
0.08323
0.09363
0.10403
0.11444
0.12484
0.13524
0.14565
0.15605
0.16645
0.17686
0.18726
0.19766
0.20807
0.21847
0.22887
0.23928
0.24968

 ...hourly t/sdf values

t_{over_sdf}	q_{over_Q}
0.07	0.008
0.1	0.025
0.15	0.068
0.2	0.114
0.25	0.157
0.3	0.197
0.35	0.232
0.4	0.264
0.45	0.292
0.5	0.317
0.55	0.34
0.6	0.361
0.65	0.38
0.7	0.398
0.75	0.414
0.8	0.429
0.85	0.443
0.9	0.456
0.95	0.468
1	0.48
1.1	0.5
1.2	0.519
1.3	0.535
1.4	0.55
1.5	0.564
1.6	0.576
1.7	0.588

...table of t/sdf
and q/Q values
from Table 1 in
Jenkins (1970)

$frac(i) := \text{interp}(t_over_sdf, q_over_Q, tsdf(i))$

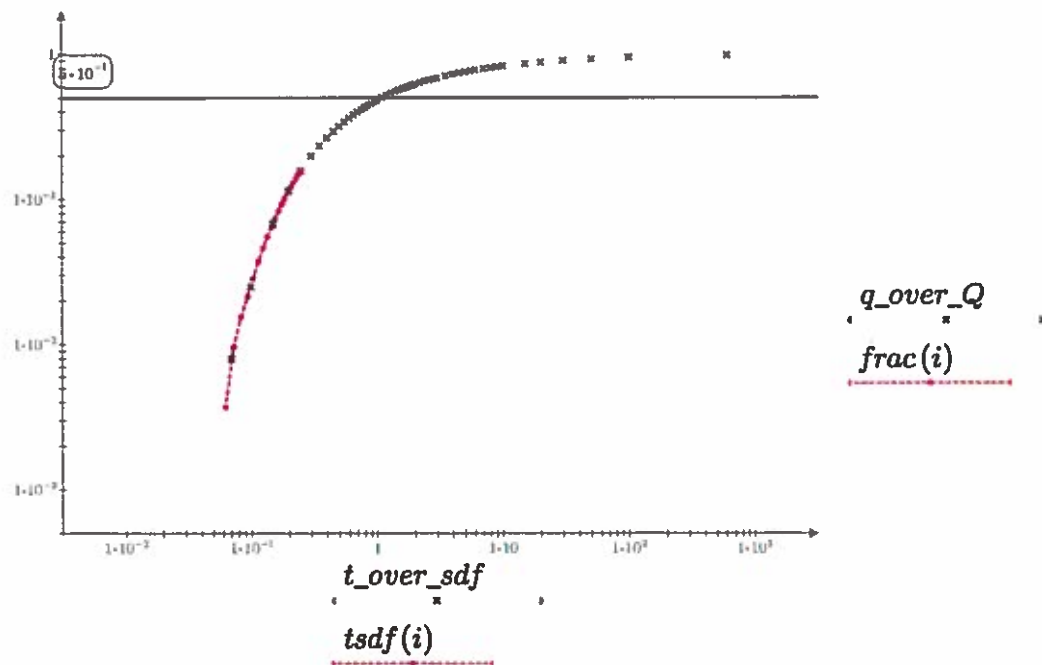
$frac(i) =$

-0.02577
-0.01988
-0.01398
-0.00809
-0.00219
0.0037
0.0096
0.0155
0.02139
0.02847
0.03742
0.04636
0.05531
0.06426
0.07357
0.08314
0.09271
0.10228
0.11185
0.12094
0.12988
0.13883
0.14778
0.15672

...ratio of stream depletion
rate to pumping rate

1.8	0.898
1.9	0.608
2	0.617
2.2	0.634
2.4	0.648
2.6	0.661
2.8	0.673
3	0.683
3.5	0.705
4	0.724
4.5	0.739
5	0.752
5.5	0.763
6	0.773
7	0.789
8	0.803
9	0.814
10	0.823
15	0.855
20	0.874
30	0.897
50	0.92
100	0.944
500	0.977

$frac(24) = 15.7\%$...ratio of stream depletion
rate to pumping rate at 24 hours



Given:

$$T_{low} := 2.0 \cdot 10^4 \cdot \frac{ft^2}{day} \quad T_{high_highQ} := 2.2 \cdot 10^5 \cdot \frac{ft^2}{day} \quad \dots \text{low and high transmissivity estimates in Clearwater Report}$$

$$T_{Report} := \text{mean}(T_{low}, T_{high_highQ}) \quad T_{Report} = (1.2 \cdot 10^5) \frac{ft^2}{day} \quad \dots \text{arithmetic mean of high and low estimates}$$

$$T_{high_lowQ} := 1.5 \cdot 10^5 \cdot \frac{ft^2}{day} \quad \dots \text{high transmissivity estimate for low pumping rate in Clearwater Report}$$

$$Q_{low} := 1860 \cdot gpm \quad Q_{high} := 2436 \cdot gpm \quad \dots \text{low and high pumping rate estimates from Clearwater Report}$$

$$Q_{avg} := \text{mean}(Q_{high}, Q_{low}) = 2148 \cdot gpm \quad \dots \text{average pumping rate}$$

Evaluate impact on arithmetic mean of using high and low pumping rate estimates:

$$Q_{actual_ratio} := \frac{Q_{high}}{Q_{low}} = 1.31 \quad \dots \text{ratio of high and low pumping rate estimates}$$

$$Q_{ratio} := \frac{T_{high_highQ}}{T_{high_lowQ}} = 1.467 \quad \dots \text{ratio of high to low pumping rate estimates based on reported Cooper-Jacob transmissivity estimates}$$

$$Q_{high} := Q_{avg} \cdot \frac{T_{high_highQ}}{\text{mean}(T_{high_lowQ}, T_{high_highQ})} = 2554.4 \cdot gpm \quad \dots \text{effective low and high pumping rates back-calculated using reported transmissivity range}$$

$$Q_{low} := Q_{avg} \cdot \frac{T_{high_lowQ}}{\text{mean}(T_{high_lowQ}, T_{high_highQ})} = 1741.6 \cdot gpm$$

$$\frac{Q_{high}}{Q_{low}} = 1.467 \quad \dots \text{verify ratio of back-calculated pumping rates = ratio of trans. estimates}$$

$$T_{low_avgQ} := T_{low} \cdot \frac{Q_{avg}}{Q_{low}} = (2.467 \cdot 10^4) \frac{ft^2}{day} \quad \dots \text{low end of transmissivity range for average pumping rate}$$

$$T_{high_avgQ} := T_{high_highQ} \cdot \frac{Q_{avg}}{Q_{high}} = (1.85 \cdot 10^5) \frac{ft^2}{day} \quad \dots \text{high end of transmissivity range for average pumping rate}$$

$$T2 := \text{mean}(T_{low_avgQ}, T_{high_avgQ}) = (1.048 \cdot 10^5) \frac{ft^2}{day} \quad \dots \text{arithmetic mean for average pumping rates}$$

$$\frac{T_{Report} - T2}{T2} = 14.47\% \quad \dots \% \text{ impact on arithmetic mean of using high pumping rate estimate for high transmissivity estimate and low pumping rate for low transmissivity estimate}$$

Given:

$a := 895 \cdot ft$...perpendicular distance from well to stream

$S := 0.1$...specific yield

$T_{avg_Q} := 1.048 \cdot 10^5 \cdot \frac{ft^2}{day}$...arithmetic average transmissivity for average of low and high effective pumping rate estimates based on reported transmissivity estimates

$Jenkins24_{RMEA} := 56.3453\%$...Reported Jenkins stream depletion ratio after 24 hours of pumping

Apply Jenkins (1970) method to compute stream depletion ratio after 24 hours of pumping:

$sdf := \frac{a^2 \cdot S}{T_{avg_Q}}$...stream depletion factor

$i := 1..24$...hourly index variable

$tsdf(i) := \frac{i \cdot 1 \cdot hr}{sdf}$...t/sdf equation

$tsdf(i) =$...hourly t/sdf values

0.05451
0.10903
0.16354
0.21805
0.27257
0.32708
0.38159
0.43611
0.49062
0.54513
0.59965
0.65416
0.70868
0.76319
0.8177
0.87222
0.92673
0.98124
1.03576
1.09027
1.14478
1.1993
1.25381
1.30832

t_over_sdf	q_over_Q
0.07	0.008
0.1	0.025
0.15	0.068
0.2	0.114
0.25	0.157
0.3	0.197
0.35	0.232
0.4	0.264
0.45	0.292
0.5	0.317
0.55	0.34
0.6	0.361
0.65	0.38
0.7	0.398
0.75	0.414
0.8	0.429
0.85	0.443
0.9	0.456
0.95	0.468
1	0.48
1.1	0.5
1.2	0.519
1.3	0.535
1.4	0.55
1.5	0.564
1.6	0.576
1.7	0.588

...table of t/sdf and q/Q values from Table 1 in Jenkins (1970)

$frac(i) := \text{interp}(t_over_sdf, q_over_Q, tsdf(i))$

$frac(i) =$

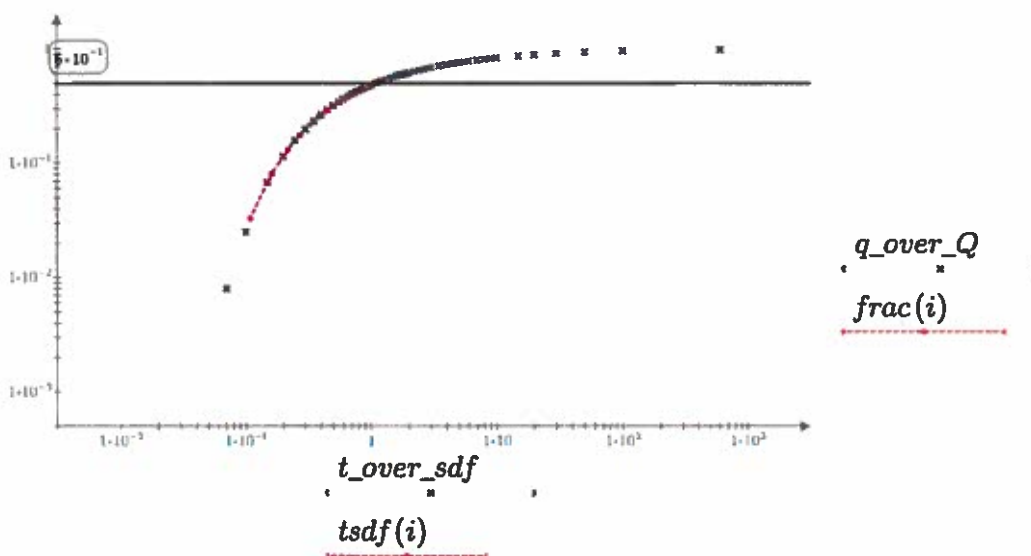
-0.00078
0.03276
0.08046
0.12953
0.17505
0.21596
0.25222
0.28422
0.31231
0.33776
0.36085
0.3815
0.40078
0.41796
0.43396
0.44878
0.46242
0.4755
0.48715
0.49805
0.50851
0.51887
0.52761
0.53625

...ratio of stream depletion
rate to pumping rate

1.8	0.598
1.9	0.608
2	0.617
2.2	0.634
2.4	0.648
2.6	0.661
2.8	0.673
3	0.683
3.5	0.705
4	0.724
4.5	0.739
5	0.752
5.5	0.763
6	0.773
7	0.789
8	0.803
9	0.814
10	0.823
15	0.855
20	0.874
30	0.897
50	0.92
100	0.944
600	0.977

$frac(24) = 53.6249\%$...ratio of stream depletion rate to pumping rate at 24 hrs

$Jenkins24_{RMEA} - frac(24) = 2.7204\%$...change in stream depletion rate
caused by uncertainty in pumping rate



Given:

$r_{50} := 48.83 \cdot ft$...radial distance to P50 from test well (Clearwater Report, 1st paragraph)

$r_{100} := 98.79 \cdot ft$...radial distance to P100 from test well (Clearwater Report, 1st paragraph)

$T := 1.2 \cdot 10^5 \cdot \frac{ft^2}{day}$...transmissivity estimate assumed in Technical Evaluation

$S := 0.1$...storage estimate assumed in Technical Evaluation

$u = \frac{r^2 \cdot S}{4 \cdot T \cdot t}$...u equation (Freeze and Cherry, 1979, p.317)

$u := 0.05$...maximum u for straight line approximation (Driscoll, 1986, p. 220)

For each piezometer, determine the elapsed time at which the straight line approximation of the Theis (1935) equation becomes valid:

$t = \frac{r^2 \cdot S}{4 \cdot T \cdot u}$...u equation rearranged to solve for time

$t_{critical_{50}} := \frac{r_{50}^2 \cdot S}{4 \cdot T \cdot u} = 14.3 \text{ min}$...time at which straight line approximation becomes valid in P50

$t_{critical_{100}} := \frac{r_{100}^2 \cdot S}{4 \cdot T \cdot u} = 58.6 \text{ min}$...time at which straight line approximation becomes valid in P100

ATTACHMENT D

DIAGRAM OF NEUMAN AND WITHERSPOON (1969) BOUNDARY VALUE PROBLEM

