

Assessment of Vadose-Zone Wells for Enhancing Groundwater Recharge in the
Mississippi Delta

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A thesis submitted to the faculty of The University of Mississippi in partial fulfillment of
the requirements of the Sally McDonnell Barksdale Honors College

Oxford
May 13, 2017

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Acknowledgements

I would like to thank my parents Chuck Jenkins and Sherry Williams-Jenkins for the countless patient hours they have given in raising me and helping me through my time in college. Their support, counsel, and love have been monumental and invaluable during my time at Ole Miss.

I would like to thank the Sally McDonnell Barksdale Honors College and the University of Mississippi Office of Research and Sponsored Programs. Their financial help made this thesis and the data collection and analysis possible.

I would like to thank the entire Department of Geology and Geological Engineering at the University of Mississippi for the instruction, guidance, and friendship they have shown me during my four years in the program. The education I have received is excellent and without it and their support, I would never have been able to finish this thesis.

Finally, I would like to thank my advisor, Dr. Andrew O'Reilly. Over the past two years he has helped me immensely, not only on my thesis but in my other classes and tribulations during life. I want to thank him for staying patient with me through all the unannounced office visits that lasted way too long, and the various complaints I have had over the past two years. Dr. O'Reilly's patience and kindness over this time period made the process enjoyable and feasible.

This thesis is dedicated to my late father Charles Michael "Chuck" Jenkins.

Abstract

Current pumping rates in the Mississippi River Valley Alluvial Aquifer in the Mississippi Delta are unsustainable as they are depleting the water level in the aquifer. The following thesis is a feasibility analysis on groundwater recharge to the Mississippi River Valley Alluvial Aquifer using vadose zone wells. Two days were spent in the field at Sky Lake near Belzoni, Mississippi, collecting core soil samples, bag soil samples, and constant head hydraulic conductivity values. Numerous days were spent in the lab conducting sieve analyses, Atterberg limits, and falling head hydraulic conductivity tests. The Atterberg limit test and sieve analyses were used to create a soil profile of the investigation area, and the hydraulic conductivities were used to find the flow rate of water from the vadose zone well into the soil. Using the data, it was found that an excessive number of wells would be required to offset the historical decline for the entire Delta. A smaller target area (37.5 square miles) was examined and it was determined that a remediation process of 283 wells at 8 wells per square mile would be a reasonable scenario in offsetting the decline in this analysis. In conclusion, vadose zone wells are not a feasible option for remediation of water-level decline over the entire Delta, but can be useful on a smaller scale.

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1.0 Introduction

The Mississippi Delta supports an agricultural industry that annually produces over \$1.5 billion in agricultural commodities (Delta Council, 2011). The region pulls its water supply from the Mississippi River Valley Alluvial Aquifer (MRVAA); however, water levels decreasing at a rate of 1 to 1.5 feet per year over the past three-and-a-half decades indicate that the rate of groundwater use is not sustainable (Brandon, 2015). The current and planned projects for water conservation and irrigation management practices are not projected to fully alleviate the long-term depletion of the aquifer (Pennington, 2015).

The MRVAA is recharged by multiple processes, such as lateral leakage from the Mississippi and other rivers in the Delta, lateral flow from the Bluff Hills on the eastern edge of the Delta, upward leakage from underlying aquifers, downward leakage from natural and man-made lakes, and the infiltration (vertical recharge) of rainwater and floodwater that is not used for evapotranspiration (the total amount of water removed from evaporation and transpiration, ET) (Arthur, 2001). Much of the Delta has layers of silt and clay at and near the surface, which overlay coarser material. These silts and clays make it difficult for vertical recharge to occur. Macropores provide preferential flow pathways and may increase the amount of recharge if they fully penetrate the fine grained upper soil layers.

Artificial recharge uses engineered systems where surface water is put on or in the ground for infiltration and ensuing movement to aquifers (Bouwer, 2002). There are multiple types of artificial recharge. This thesis will explore the feasibility of using

vadose zone wells to increase the vertical recharge of the MRVAA in the Delta using data collected at Sky Lake near Belzoni, Mississippi (Figure 1).

2.0 Background

The Mississippi Delta is a 6,250 square-mile area in northwest Mississippi. Originally mostly covered in hardwoods and marsh (Sumner, 1990), it was cultivated for over a century to be a prime agricultural area. It produced numerous cash crops and rose to financial prominence during the slavery and Jim-Crow eras, but has become very poor in recent decades. The area is made up of a Quaternary alluvium (Figure 2) deposited on an erosional surface that has a system of north-south valleys (Sumner, 1990). The alluvium fines upward with gravel and coarse sand at the bottom and clay at the top. The clay top is of varying thicknesses, but has an average thickness of about 20 feet. It can, however, have a thickness of up to 70 feet in some of the abandoned stream channels (Sumner, 1990). The average thickness of the entire alluvium unit is about 140 feet but ranges from 80 to 240 feet (Sumner, 1990). The area is a fluvial environment with sediments deposited from rivers, oxbow lakes, and Mississippi River floods.

2.1 Hydrogeology of the Delta

Quaternary-age sand and gravel deposits overlying an erosional Tertiary-age surface make up the MRVAA (Figures 2 and 3) (Fisk, 1944; Arthur, 2001). As stated before, there are multiple sources of recharge to the aquifer. Due to overlying clay and fine-grained material, the recharge due to infiltration is generally low. It discharges water to the Mississippi River, to streams within the Delta, to lakes, to aquifers underlying it, and to pumping (Arthur, 2001). The regional groundwater flow path used to be comprised of two flow components – flow from the north to the south and from the

east and west peripheries toward the center. The flow-paths generally followed topography, which slopes from north to south and has high points bounding it on the east and west (Barlow and Clark, 2011). Due to extensive development, the flow path is now intercepted by a large cone of depression in the central Delta (Figure 4) which has formed because of groundwater pumping for irrigation (Barlow and Clark, 2011). The aquifer varies between confined (overlying impermeable layer) and unconfined conditions (no impermeable overlying layer) (Sumner, 1990). The central portion is unconfined due to the relatively deep ground-water levels, however; on the eastern parts the aquifer is generally confined (Sumner, 1990).

2.2 Movement of Water in Soil

The vadose zone, the portion of the subsurface that is not fully saturated, and processes within are what control the movement of water that recharges aquifers. The exception is those places where the aquifer is in direct contact with surface water. Surface water from storms or other events will slowly percolate down through the soil until it reaches the water table. The rate at which this occurs is controlled by multiple factors, one of which is grain size. The rate that water moves through the soil is linearly directly proportional to the hydraulic gradient according to the hydraulic conductivity, which is generally directly proportional to the grain size of the soil. A finer grained soil, such as a clay or silt (.06 to <.002 mm diameter), will require much more time for the water to pass through it than a coarser grained material, (e.g. sand, .06 to 2 mm diameter). Thus, clays and silts have much lower hydraulic conductivities than sands under saturated conditions. Another factor in how quickly water moves through the soil is preferential flow pathways. These include soil cracks, root holes, boreholes, and other “macropores.”

These features allow water to bypass the bulk soil matrix, which may increase the rate of recharge to the aquifer. Macropores are common (Perkins et al., 2011) soil features in the Delta.

2.3 Artificial recharge methods

Artificial recharge intends to bypass the lower hydraulic conductivity soils in order to increase the amount of water recharged to an underlying aquifer. There are numerous sources of water for artificial recharge, which includes perennial or intermittent streams that might or might not be regulated with dams, storm runoff, aqueducts or other water-conveyance facilities, irrigation districts, drinking-water treatment plants, and sewage treatment plants (Bouwer, 2002). There are three types of artificial recharge: surface infiltration, vadose-zone infiltration (recharge trenches and vadose zone wells), and wells (Bouwer, 2002). Each type of recharge has its own benefits and drawbacks.

2.3.1 Surface Infiltration

Surface infiltration systems are divided into two types, in-channel and off-channel (Bouwer, 2002). In-channel systems consist of dams placed across ephemeral or perennial streams to back the water up and spread it out. This will increase the amount of wetted area. Off-channel systems are made of specially constructed infiltration basins, lagoons, old gravel pits, flood-irrigated fields, perforated pipes, or any other facility that facilitates the infiltration of water into the soil and underlying groundwater (Bouwer, 2002).

2.3.2 Recharge Trenches

Recharge trenches are typically less than about 3.28 ft wide and about 15.40 m deep. Backfilled with coarse sand or fine gravel, water is applied through a perforated pipeline on the surface of the back fill. The trench is covered to blend in with the surroundings. Since they are physically in the vadose zone, they cannot be pumped for backwashing to clean out clogs. Pre-treatment of the water is often required to extend the life of the trench. Where Pre-treatment is not required; however, implementing them is inexpensive, which makes creating new ones a viable option (Bouwer, 2002).

2.3.3 Vadose Zone Wells

Vadose zone wells (recharge shafts or dry wells) (Figure 5) are the focus of this thesis. They are up to 3.28 ft in diameter and 196.85 ft deep at their largest. The wells are filled with a coarse sand or fine gravel, and water is applied through a perforated or screened pipe in the center. To avoid air entrainment in the water, free-falling water in the pipe should be avoided. To avoid this, water is supplied through a smaller pipe inside the screened pipe that extends well below the water level in the pipe. This applies a sufficient head-loss to prevent free-falling water. The same clogging concerns and methods as recharge trenches apply to vadose zone wells (Bouwer, 2002).

2.3.4 Wells

Direct recharge or injection wells are the last type of artificial recharge methods. They are used when the other methods cannot be used. They are wells that are installed into the saturated zone and then treated water is pumped directly into the aquifer. In the United States, the water pumped in must meet drinking water standards. This is an expensive method with many chances for clogging (Bouwer, 2002).

3.0 Methods

3.1 Sample Collection

On June 22 and September 16 of 2016, trips to Sky Lake were conducted to perform field measurements of hydraulic conductivity, and to collect bag and core samples at two locations. The locations of the two boreholes can be seen in Figure 6 and a picture of each location can be seen in Figures 7 and 8. The boreholes were drilled using the borehole prep. kit (Figure 9) from SoilMoisture Equipment Corporation that had a 2.36-inch diameter hand auger. Both boreholes were designed to be drilled to 10 feet. In the first borehole, however, extra time and the occurrence of sand grains at 10 feet allowed for drilling and collecting a bag sample at 11 feet. Due to the lack of rain at the end of this summer, drilling at Site 2 was considerably more difficult. This difficulty caused time to run out and only nine feet drilled. These samples were used for the sieve analysis and Atterberg limit tests.

During the drilling, soil cores were also taken. Cores were taken at 1, 2, 3, 4, 6, 8, and 10 feet at the first location. At the second location, cores were taken at 1, 2, 3, 4, 6, and 8 feet. The cores were taken using a soil core sampler (Figure 10) from SoilMoisture Company. The core sampler allows for the collection of a relatively intact and undisturbed soil sample. The samples were 2.25 inches in diameter and 1.2 inches in length and held in a brass ring. The samples were then capped, taped with electrical tape, and labeled (Figure 11). These samples were then used in the lab for a falling head permeameter test. An attempt to take a core at nine feet at the second location was performed, but the soil was too dry to stick together and was unable to be recovered.

3.2 Soil Texture

3.2.1 Sieve Analysis

In the lab, a portion of each bag sample was dried at 110° C for 24 hours to perform sieve analysis and Atterberg limit tests. For sample preparation, a combination of ASTM D421 and D422, as well as AASHTO T87 was followed (Bowles, 1970). For the test procedure, AASHTO T88 was followed. The samples were first weighed before they were baked to ensure enough sample was present. After the samples were baked, a mortar and pestle were used to break up the clumps of soil. Once all the samples were finished being crushed, the sieve analysis was run. For each sample a maximum of 500 grams was used; some samples did not have 500 grams of sample so the entirety of the bag was used. They were placed in a sieve stack containing a 10, 18, 40, 60, 100, 120, 200, and 230 size sieves with a pan on the bottom. All sieves with numbers 200 and smaller catch all sand and larger size particles with very fine sand being caught on the number 200 and small pebbles being caught on the number 10 sieve. The number 230 sieve and pan collect all fine-grained material (clays and silts). Each sieve was weighed prior to the addition of the soil. The stack was then placed in a Rotosift (Figure 12) and sieved for 15 minutes. After the sieving was complete, each sieve was individually weighed and recorded. Using the total weight of soil after sieving, a value for the percent retained and percent passing each sieve was calculated. These percentages were then used to determine the amount of gravel, sand, and fines for each sample.

3.2.2 Atterberg Limits

For the Atterberg limits procedures, ASTM D 4318 was followed for the liquid limit and plastic limit tests. The test used is the one point liquid limit test. First, a

maximum of 200 grams of sample was collected in an evaporating dish. The sample was then saturated and allowed to sit, covered, for one hour to become fully saturated. Then, it was divided in half for the completion of the liquid limit and plastic limit tests.

For the liquid limit test, the soil was put in an automatic liquid limit machine (Figure 13). The soil is spread evenly in the front half of the brass cup, then, a groove is cut out of the middle of it using the grooving tool (Figure 14). The apparatus is then turned on; it drops the cup 1 cm repeatedly and records how many blows occurred. If the soil in the cup creates a closure about 1.5 cm long within a range of 20 to 30 blows it is at a good moisture content. The closure and soil within it is removed and put in a previously weighed tin cup and covered. A second test with the same soil is performed in order to ensure accuracy. To do this, the second test must achieve closure within two blows on either side of the original count. If closure is not completed within the range, either on the first or second test, the sample is removed from the brass cup. If it required more than 30 blows then more water was added and mixed in, but if less than 20 blows was required then the soil is worked with spatulas in order to remove water. Once the two required samples are obtained, they are then weighed and baked at 110° C for 24 hours. The dry weight is then recorded to obtain the water content. Since the one point liquid limit method was used, the conversion factor specified for each number of blows specified in the ASTM standard was then multiplied by the water content to obtain the liquid limit.

For the plastic limit, the other half of the saturated soil is used. Using one's hands, the soil is rolled out in thin pieces (Figure 15) at a constant rate. Once the soil reaches the size of a number 2 pencil, it is closely observed while rolling. If cracks

appear along the sample, it is then placed in a previously weighed tin cup and capped. This procedure is continued until two tins each have 10 grams of sample in it. The tins are then weighed and baked for 24 hours at 110° C. After drying, the sample is weighed and the water content calculated, then the plastic limit is calculated by computing the average of the water contents.

3.3 Hydraulic Conductivity

3.3.1 Field Measurement

The hydraulic conductivity of the soil was taken using two separate test procedures, one in the field and one in the lab. Due to time constraints, a test in the field could not be run at each foot of depth. To obtain consistency, a field measure of hydraulic conductivity was performed at 1, 2, 3, 4, 6, 8, and 10 feet at site one, and at 1, 2, 3, 4, 6, and 8 feet at site two. Soil cores were taken at these same depths. The field measurements were collected using an Aardvark permeameter kit (APK) (Figure 16) from SoilMoisture Equipment Company. The APK is a constant head permeameter, meaning it keeps a constant water level throughout the experiment. A digital scale is connected to a computer that runs the devices software and records changes in water volume through change in weight. It then calculates the hydraulic conductivity of the soil. An 11,600 ml jug of water is placed on top of the scale. Once the hole has been augered to the desired depth and a soil core taken, the bottom of the hole is flattened and squared off. The permeameter module is then connected to the jug using a flexible plastic tubing, where it maintains a constant depth of water in the bottom of the hole. The module is connected to a modified tape measure that gives an accurate reading of how deep the hole is, which must be used in the software. Once everything is connected,

the module is lowered into the hole. The water is then turned on using the spigot on the jug, and measurements are recorded. The measurements continue until the software calculates three consecutive readings within 10% of each other. On some samples, the software never reached the 10% change threshold and the operators stopped the program when it was within 20 to 30% change due to time constraints. Due to a software error, test data at 8 feet for site one, and 4 feet for site two were lost.

3.3.2 Laboratory Measurement

In the lab, a falling head permeameter test was conducted on a soil core from each of the depths. The test was conducted using a 2816G1 Chameleon Station from Soilmoisture Equipment Corporation (Figure 17). A 100-cm³ cylinder is filled with water and attached to an elevated base, which holds the soil core, by a flexible plastic tube connecting to two ports. The port on the cylinder has a nozzle that dictates in what directions the water can flow. The soil core is capped by a piece that allows for water flowing upward from the base to exit the top of the core and overflow through an open tube. Also attached to the port on the cylinder is a pressure transducer that uses fluctuations in pressure to determine the head difference across the core and transmits that data to the attached computer. The computer runs a software to compute the hydraulic conductivity. The cylinder is closed on top by a rubber stopper. The stopper is penetrated by a plastic tube and port that can be opened or closed to the atmosphere. The plastic tube is positioned in the cylinder such that its bottom is level with the base of the soil core. To prepare the core for testing, a port on the bottom of the cylinder is opened such that water will only flow to the core. This port is left open until water is flowing out of the open tube above the soil core. To achieve saturation, the port on the rubber stopper

is closed, allowing no air to enter the cylinder, thereby maintaining a constant water level. Once saturation is achieved, the port is turned to allow flow to both the transducer and the soil core. It will continue to take readings until the water level drops below a certain height (5 cm for these tests) or the user determines enough time has passed and data points collected.

3.4 Vadose Zone Well Flow Rate Calculations

To calculate the flow rate of the vadose zone wells an equation presented by Bouwer (2002) as simplified from Bouwer (1978) was used:

$$Q = \frac{2\pi K L_w^2}{\ln\left(\frac{2L_w}{r_w}\right) - 1} \quad (1)$$

where Q is the flow rate (L³/T), K is the hydraulic conductivity of the soil (L/T), L_w is the water depth in the well (L), and r_w is the radius of the well (L) as depicted in figure 5b. This equation assumes that the soil material is uniform. The average depth to the water table in the area is about 30 feet. For equation 1 to be valid, the distance to the water table must be at least twice the length of L_w. For the soil conditions encountered, L_w needs to start at a depth of 10 feet. Therefore, L_w can be no larger than 6 feet, which leaves 14 feet from the bottom of L_w to the water table and satisfies the requirements. In the equation, r_w must also be one-tenth the size of L_w. This gives a value of 0.6 feet for r_w.

Equation (1) yielded a rather small Q for the required design parameters, so a second equation presented by Bouwer (1978) as simplified from Zangar (1953) was used:

$$Q = \frac{K(\pi L_w(3L_w + 2S_i))}{3\ln\left(\frac{L_w}{r_w}\right)} \quad (2)$$

where K , L_w , and r_w are all the same variables as Equation 1. S_i is the depth from the base of the well to the water table (L), as depicted in figure 5b. The difference in this equation is that it does not require for S_i to be twice as long as L_w . Using this equation, L_w is 10 feet, S_i is 10 feet, and r_w is 1 foot. The radius must again be one tenth the size of L_w . This equation yielded much better results.

4.0 Results

4.1 Soil Profile

Using the data from the sieve analysis (Appendix A) and Atterberg limit tests (Appendix B), a soil profile was made for both boreholes. The first borehole is consistent with the typical geology of the Delta, as it coarsens with depth. It is primarily clay at the top, coarsens to silt in the middle, and coarsens to a sand after ten feet. This is also consistent with a natural gamma log (Figure 18) from a well that was drilled about 50 feet to the east in the same field. On a gamma log, a higher value indicates a more fine grained soil containing clay, while a lower value indicates a lesser abundance of clay often indicative of a more coarse-grained material. Figure 18 shows a gamma log that starts out high and diminishes with depth. It reaches a reading that correlates with sand around 10 feet in depth, which is consistent with our findings.

The second borehole, however, has a different lithologic profile than the first. Whereas in the first borehole the soil coarsens with depth, soil at the second borehole is fine grained all the way through. It is similar that the top is primarily clay and coarsens to silt in the middle, but then it moves back into a fine-grained sediment. This, however, does agree with a geophysical log (Figure 19) from a well about 1,300 feet away in a

similar environment. The well log shows a gamma signature that stays relatively high for the first 20 feet, which agrees with the soil analysis made from the sieving.

4.2 Hydraulic Conductivity

Table 1 contains the hydraulic conductivity values for both field and lab measurements. Appendix C contains the individual data sheets for each sample depth tested. For the first borehole, the average hydraulic conductivity in the field is 2.25×10^1 ft/d. The maximum value is 6.34×10^1 ft/d and a minimum value of 1.34×10^1 ft/d. For the second borehole, the average hydraulic conductivity in the field is 1.1×10^0 ft/d. The maximum value is 3.78×10^0 ft/d and the minimum value is 1.40×10^{-1} ft/d.

The lab hydraulic conductivities, however, were slightly different. The average hydraulic conductivity for the first borehole from the lab is 1.68×10^0 ft/d, which is smaller than what was in the field. The maximum value from the lab is 7.5710^0 ft/d and the minimum value is 5.68×10^{-3} ft/d. The average value at the second borehole is 6.72×10^{-3} ft/d. The maximum value is 1.58×10^{-2} ft/d and the minimum is 2.88×10^{-3} ft/d. All of these are smaller than their corresponding field measurements.

4.3 Vadose Zone Well Flow Rate

The flow rate for the vadose zone wells was calculated using both equations 1 and 2 and expressed in ft^3/d and ft^3/yr . Table 1 contains the values for each. Graphs showing the relation between the hydraulic conductivity and flow rate can be found in Figures 20 and 21 for ft^3/d and ft^3/yr respectively. The values were calculated using Eq. (1) with an L_w value of 6 feet and an r_w value of .6 feet, and the K value is based off the lab measurements. Deciding to use the lab values was determined for a number of reasons. They were not subjected to the same time constraints as the field measurements and were

in a more controlled environment. They were also chosen because a falling head permeameter gives a much more accurate reading of fine-grained soils than a constant head permeameter does. They also give conservative values when calculating the flow rates. The average flow rate for the first borehole is 190 ft³/d and 69,630 ft³/yr. The maximum flow rate is 860 ft³/d and 313,100 ft³/yr. The minimum value is .7 ft³/d and 240 ft³/yr. The second borehole followed the same parameters. It has an average flow rate of .8 ft³/d and 280 ft³/yr. The maximum value is 1.8 ft³/d and 650 ft³/yr. The minimum value is .3 ft³/d and 120 ft³/yr.

For Equation (2), flow rates in the same units were calculated. In this case, however, L_w has a value of 10 feet, r_w has a value of 1 foot, and S_i had a value of 10 feet. The K value is still from the lab measurements. The average flow rate for the first borehole is 383 ft³/d and 139,690 ft³/yr. The maximum flow rate is 1720 ft³/d and 628,180 ft³/yr. The minimum value is 1.3 ft³/d and 486 ft³/yr. The second borehole followed the same parameters. It has an average flow rate of 1.5 ft³/d and 560 ft³/yr. The maximum value is 3.6 ft³/d and 1310 ft³/yr. The minimum value is .7 ft³/d and 240 ft³/yr.

5.0 Discussion

The main problem with the MRVAA is the rate at which water is being depleted. Using the delineated cone of depression from Figure 4 to find the area (1870 mi²) and a cumulative storage loss of 3,316,500 acre-ft from 1987 to 2009 (Barlow and Clark, 2011) the total depth lost is calculated to be 2.77 feet. The average rate of decline for the aquifer for that time span is .12 ft/yr. According to Figure 22, however, the rate of decline has increased in the new millennium resulting in a cumulative storage loss of 1,016,500 acre-feet. Using the same method, the cumulative depth lost from 2005 to

2009 is .85 feet at a rate of .17 ft/yr. Calculations (Table 2) were made to determine the number of vadose zone wells required to offset the long-term (1987-2009) decline due to pumping. For aquifer water-level decline remediation calculations, the highest calculated vadose zone well flow rate was used. It occurs when the lithology changes to sand, and knowing the general lithology of the region coarsens with depth it makes sense to use the highest value beneath the fine-grained sediments. The maximum flow rate from Equation (2) was used because it is over twice as large as the maximum flow rate from Equation (1). At a flow rate of 630,000 ft³/yr, each well would replenish about 1.21×10^{-5} ft/yr for the entire cone of depression. Using this rate, it would take 14,098 wells at eight wells per square mile to offset the current rate of decline per year in the aquifer.

Just over 14,000 wells is too large a number to be a reasonable method to use for recharge; however, vadose zone wells could be a realistic alternative for a smaller area. Figure 23 shows a more localized region around Sky Lake and Belzoni, Mississippi. It has an area of 37.5 square miles. Using the maximum flow rate, each well could replenish 6×10^{-4} ft³/yr. This would require only 283 wells at eight wells per square mile. This is a much more feasible number. At a diameter of 2 feet, they are small enough to not have a large ecological impact and could be accommodated around farming operations.

There are a couple of problems with this method, however, and one has to do with rainfall. The numbers above assume that there is as much water going in to the wells as there is being pumped out. The normalized average rainfall in the Delta is between 50 and 60 inches per year (Arthur, 2001). In 2000, the total pumpage from the MRVAA in the Delta was about 1300 million gallons per day (USGS, 2000). This is about 3.9 inches

per year over the entire Delta area. ET removes about 33-inches of rainfall (Sumner, 1990), which leaves about 20 inches leftover. Since the surface soil is mostly fine-grained material, not much of the rainfall will percolate far into the soil. The vast majority of it will runoff to streams and human structures such as storm drains. The purpose of the vadose zone wells is to facilitate the enhanced recovery of the remaining runoff and assist it in reaching the MRVAA.

It also assumes that the lithology is uniform throughout the Delta. This is not the case, obviously. The lithology does coarsen with depth but in some areas, it takes more than 20 feet to get to coarser material (Sumner, 1990). In addition, some areas may have more coarse material near the surface that will allow for an even greater rate of hydraulic conductivity and shallower wells.

Both of these concerns are why it is important to remember this is a feasibility analysis. Under the normalized and ideal conditions, the wells are not a viable option for the entire area. While generally inexpensive, the implementation of 14,000 wells will be very expensive due to the sheer number of them. This is a major factor because the Delta is one of the poorest regions in the country (ACS, 2017, Geography of poverty, retrieved March 16, 2017, available at: <https://www.ers.usda.gov/topics/rural-economy-population/rural-poverty-well-being/geography-of-poverty.aspx>). Vadose zone wells, however, can be viable in a smaller area: 283 wells at eight wells per square mile is not unrealistic. It would not solve all the problems, but it would serve as part of a holistic water management plan for the farmers of the region. As can be seen in Figure 24, there are already many permitted wells in the area. There are already numerous monitoring

wells in the permitted area. Adding eight more wells per square mile is less than the current density in the area.

6.0 Summary

The Mississippi River Valley Alluvial Aquifer is the most important water source for agriculture in the Mississippi Delta. The current irrigation practices, however, are depleting its water supply faster than it can naturally recharge due to low hydraulic conductivity, fine-grained units at the surface. Vadose zone wells are an artificial recharge system that bypasses fine-grained units and facilitates the release of water into the coarser grained portion of the unsaturated zone, where this additional water can continue to move downward and recharge the aquifer. The wells are not a feasible option for remediation of water-level decline over the entire Delta, but can be useful on a smaller scale.

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Tables

ID	Ksat (ft/d) field	Ksat (ft/d) lab	Q (ft ³ /d)	Q (ft ³ /yr)	Q2 (ft ³ /d)	Q2 (ft ³ /yr)
SL-1-1	0.19	0.0059	0.7	242.3	1.3	486.2
SL-1-2	1.6	0.11	12.5	4573.4	25.1	9175.7
SL-1-3	36.4	2.4	267.7	97721.6	537.2	196062.0
SL-1-4	0.1	0.2	26.6	9713.5	53.4	19488.6
SL-1-6	33.5	1.4	160.6	58632.9	322.3	117637.2
SL-1-8	N/A	0.08	9.3	3400.7	18.7	6823.0
SL-1-10	63.4	7.6	857.8	313099.9	1721.0	628182.5
SL-2-1	0.1	0.0032	0.4	131.3	0.7	263.5
SL-2-2	0.5	0.0076	0.9	313.6	1.7	629.1
SL-2-3	0.6	0.0066	0.7	271.7	1.5	545.1
SL-2-4	N/A	0.0029	0.3	119.2	0.7	239.2
SL-2-6	3.8	0.0043	0.5	179.2	1.0	359.6
SL-2-8	0.5	0.016	1.8	652.8	3.6	1309.7

Table 1: Values of lab and field hydraulic conductivity and flow rate for ft³/d and ft³/yr. Q is using Equation 1 and Q2 is using Equation 2.

Total Volume Change (acre-ft)		year	volume	
	3316500	2005	2300000 acre-ft	
	1.44467E+11 ft ³	2009	3316500 acre-ft	
	2.772993311 ft	total	1016500 acre-ft	
	0.120564927 ft/yr 1987-2009		44278740000 ft ³	
			0.849916388 ft	0.169983 ft/yr 2005-2009
Target area	Best Case	SL-1-10		Entire Area
		37.5 mi ²		1868.75 mi ²
		1045440000 ft ²		52097760000 ft ²
	Q	628182.5082 ft ³ /yr		Q
	Change in volume	0.000600879 ft/yr		Change in Volume
			282.8912 wells	14097.41 wells
			7.543766 wells/mile	7.543766 wells/mile
	Not Best Case	SL-1-3		
		37.5 mi ²		
		1045440000 ft ²		
	Q	196061.96 ft ³ /yr		
	Change in volume	0.00018754 ft/yr		
			906.3835 wells	
			24.17023 wells/mile	
	Not Best Case	SL-1-6		
		37.5 mi ²		
		1045440000 ft ²		
	Q	117637.17 ft ³ /yr		
	Change in volume	0.000112524 ft/yr		
			1510.639 wells	
			40.28371 wells/mile	

Table 2: Calculations and data of number of wells required to offset pumping

Figures

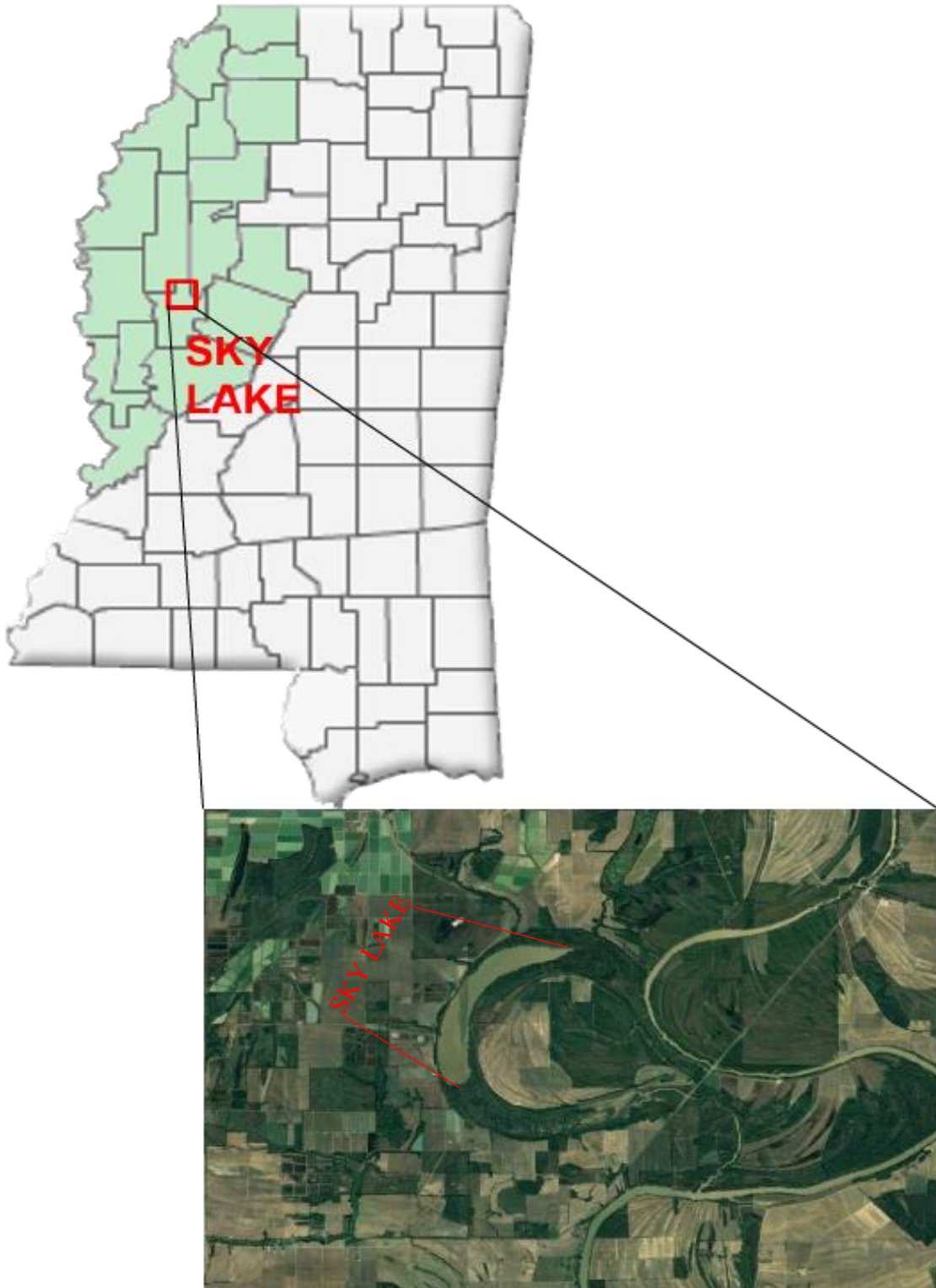


Figure 1: Location map of the Delta and Sky Lake

ERATHEM	SYSTEM	EPOCH	GROUP	LOUISIANA		ARKANSAS		MISSOURI	KENTUCKY	TENNESSEE	MISSISSIPPI	ALABAMA	Hydrogeologic units	Model layer number			
				Southern	Northeastern												
CENOZOIC	TERTIARY	UPPER PALEOCENE	Wilcox	Doleet Hills Formation	Undifferentiated Naboroth Formation	Undifferentiated	Flour Island Formation	Fort Pillow Sand	No Wilcox deposits identified as being of Paleocene age	Flour Island Formation	Fort Pillow Sand	Old Breastworks Formation	Hatchegbae Formation	Middle Wilcox aquifer	11		
																Middle Wilcox aquifer	Lower Wilcox aquifer ¹
		Carrizo Sand	Cane River Formation	Memphis Sand	Sparta Sand	Tallahatta Formation	Cockfield Formation	Gosport Sand	Upper Claiborne aquifer	Middle Claiborne confining unit	Cock Mountain Formation	Sparta Sand	Tallahatta Formation	Memphis Sand	Sparta Sand		
																Vicksburg Formation	Alluvium and terrace deposits
		Vicksburg	Vicksburg Formation	Not present in study area	Alluvium and loess deposits	Alluvium and loess deposits	Vicksburg Formation	Alluvium and terrace deposits	Mississippi River Valley alluvial aquifer	2							
											Jackson	Jackson Formation	Cockfield Formation	Gosport Sand	Upper Claiborne aquifer	Middle Claiborne confining unit	Lower Claiborne aquifer ¹
		Mid-Wilcox	Midway Group	Midway confining unit	Base of model												

Modified from Haseman and Weis, 1991

¹ Lower Claiborne aquifer includes the upper Wilcox aquifer in some parts of Mississippi.
² Winona-Tallahatta Formation is included with lower Claiborne confining unit in Hart and others (2008).
³ Old Breastworks confining unit is included with middle Wilcox aquifer in Hart and others (2008).
⁴ El Dorado confining unit and El Dorado Sand are included with middle Claiborne aquifer.

Figure 2: Stratigraphic column for the Mississippi Embayment showing the units composing as well as underlying the Mississippi River Valley Alluvial Aquifer (Clark and Hart, 2009)

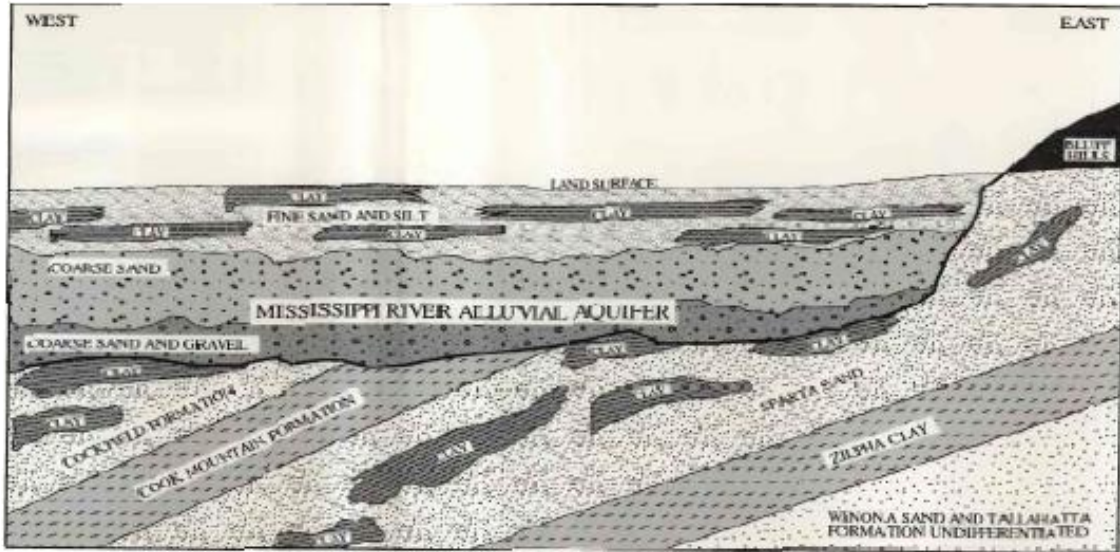


Figure 3: General hydrogeology of the Delta (Arthur, 1994)

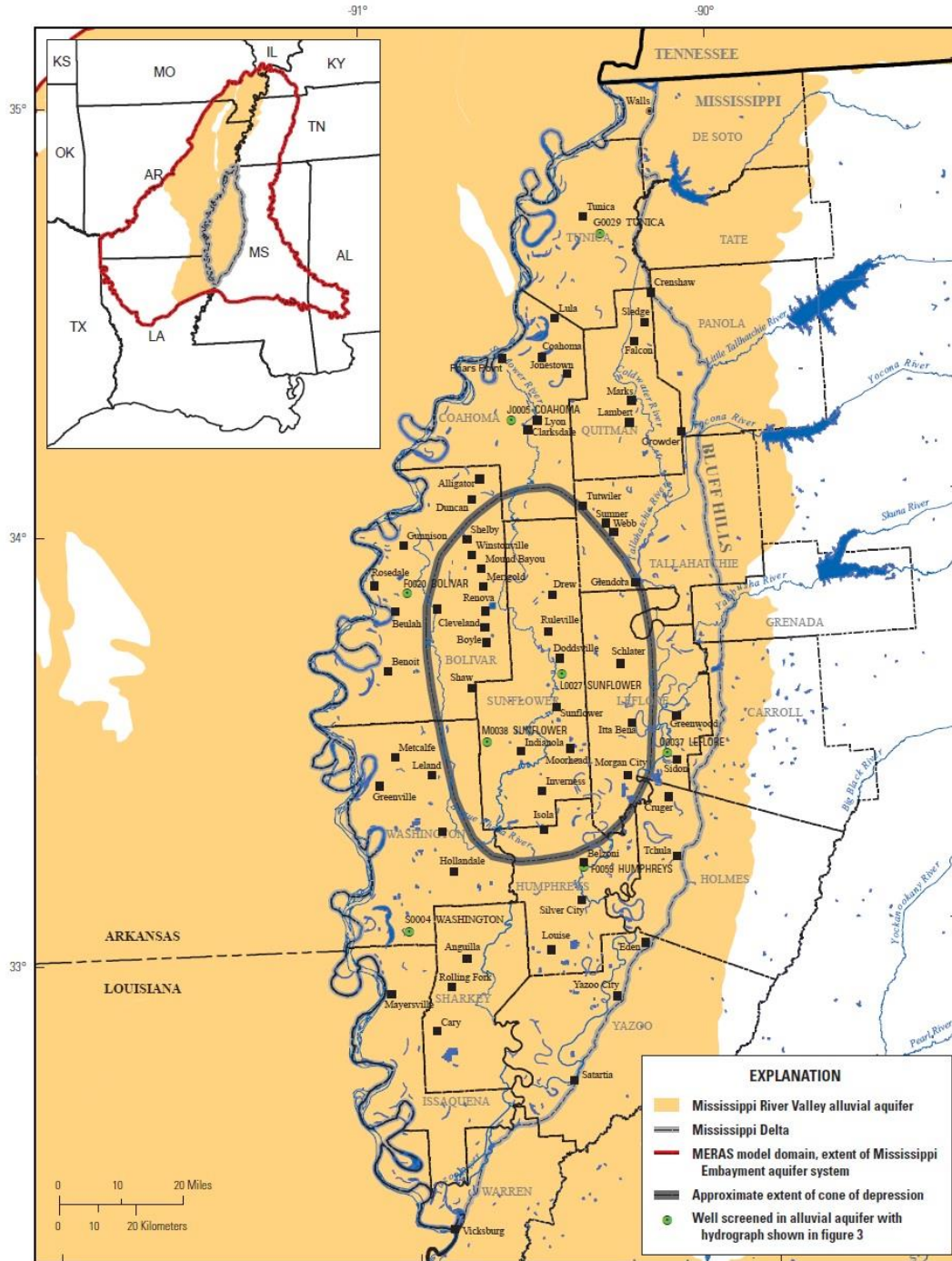


Figure 4: Delineated cone of depression for the Mississippi Delta (Barlow and Clark, 2011)

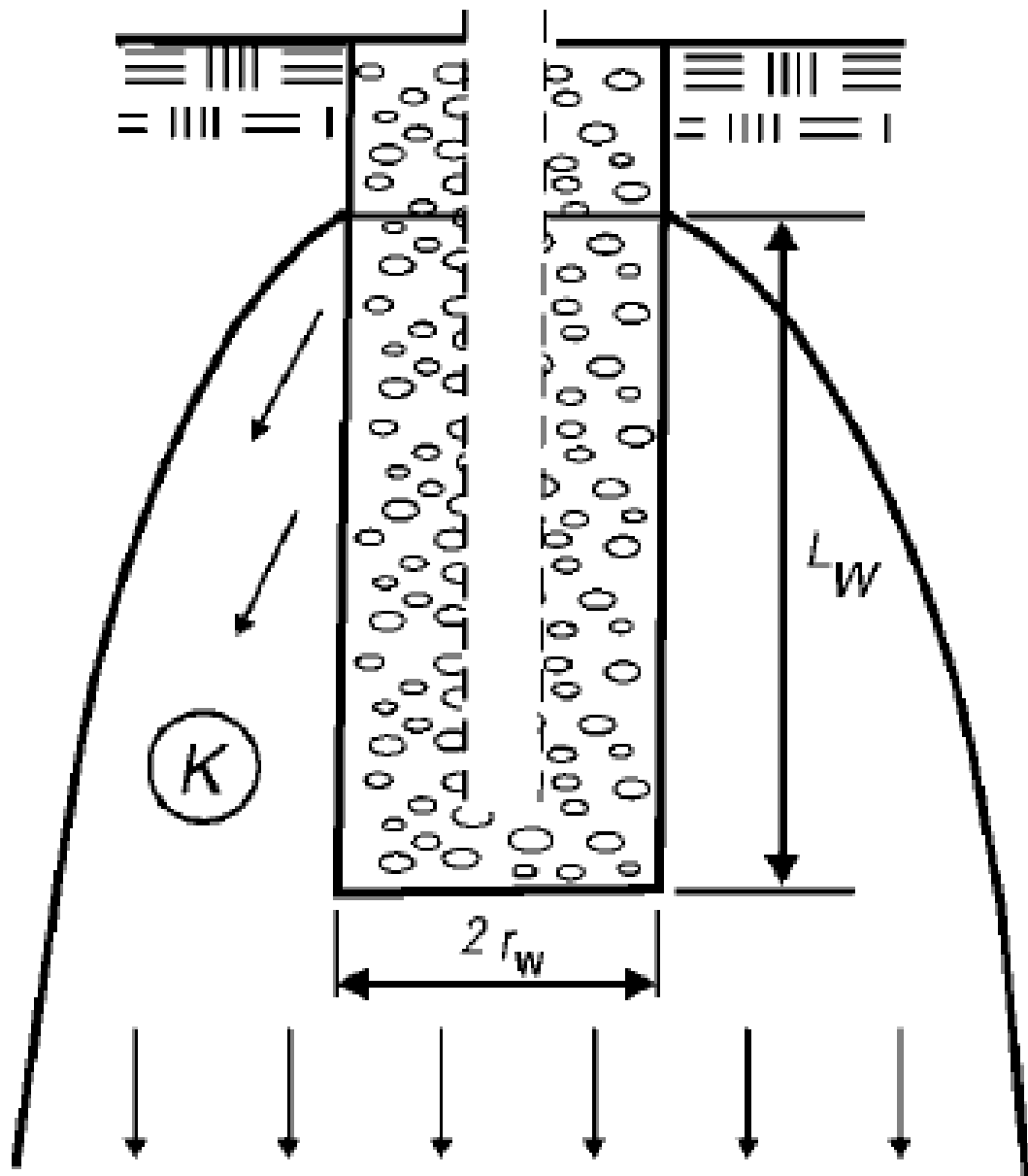


Figure 5: Typical depiction of a vadose zone well (Bouwer, 2002)

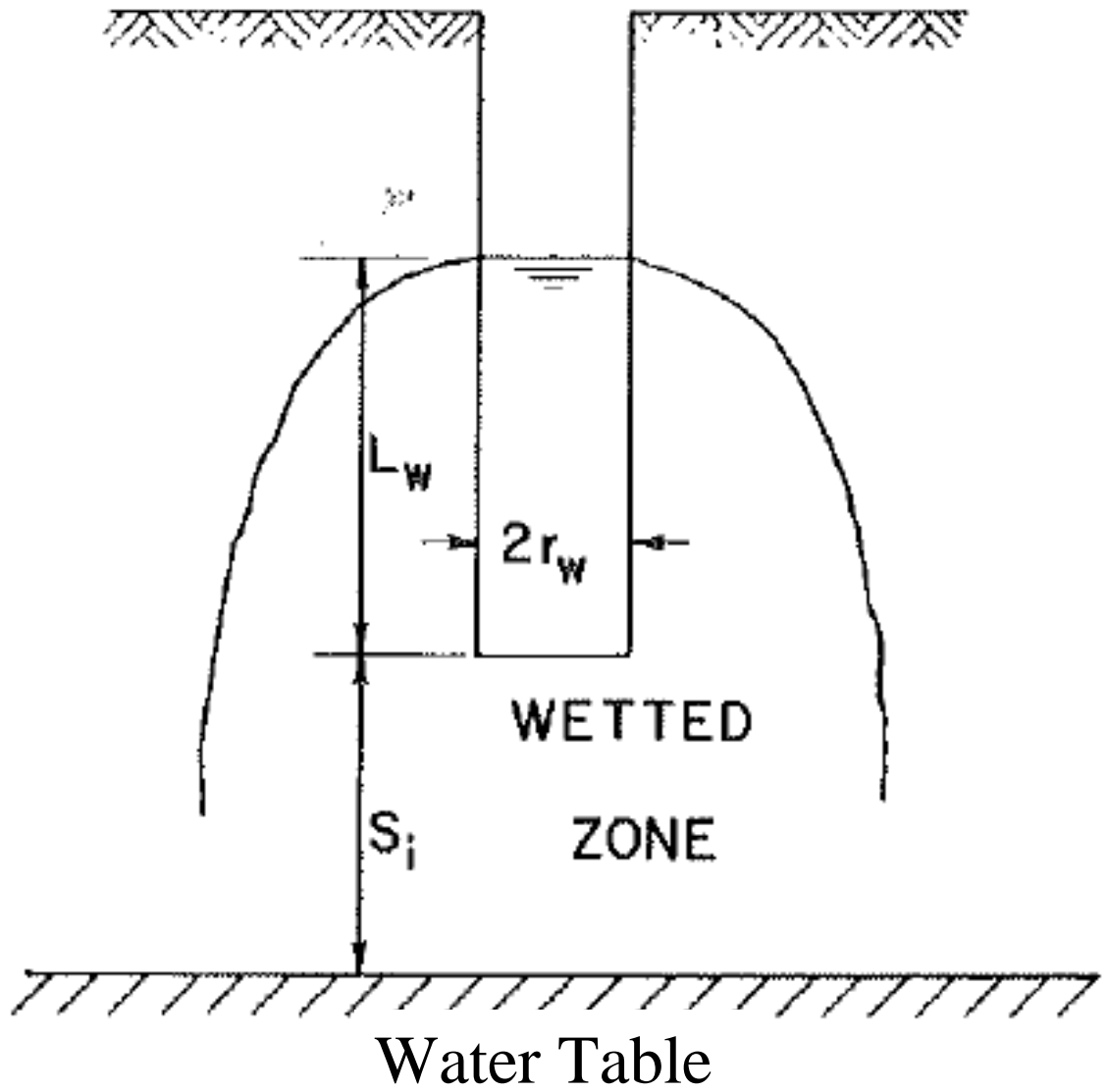


Figure 5b: Conceptual diagram of a vadoze zone well showing variables used to calculate flow rate from well (modified from Bouwer, 1978)



Figure 6: Locations of research sites

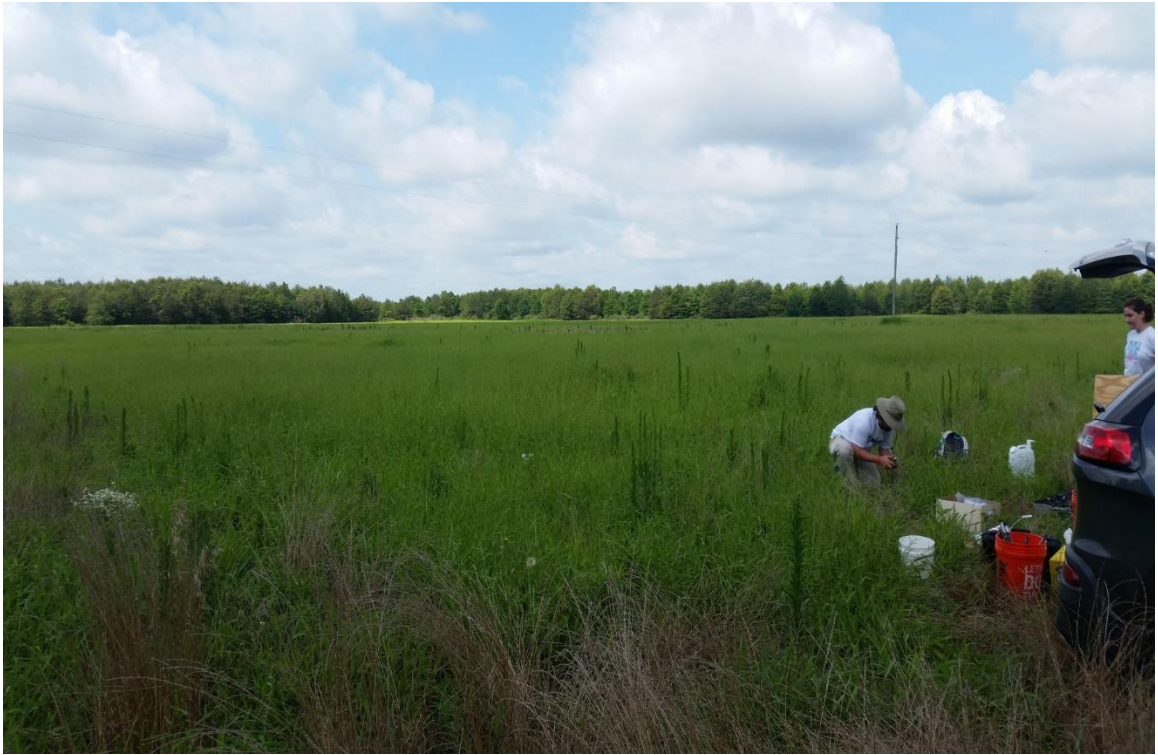


Figure 7: Location of the first borehole



Figure 8: Location of the second borehole



Figure 9: Borehole prep kit



Figure 10: Soil core sampler

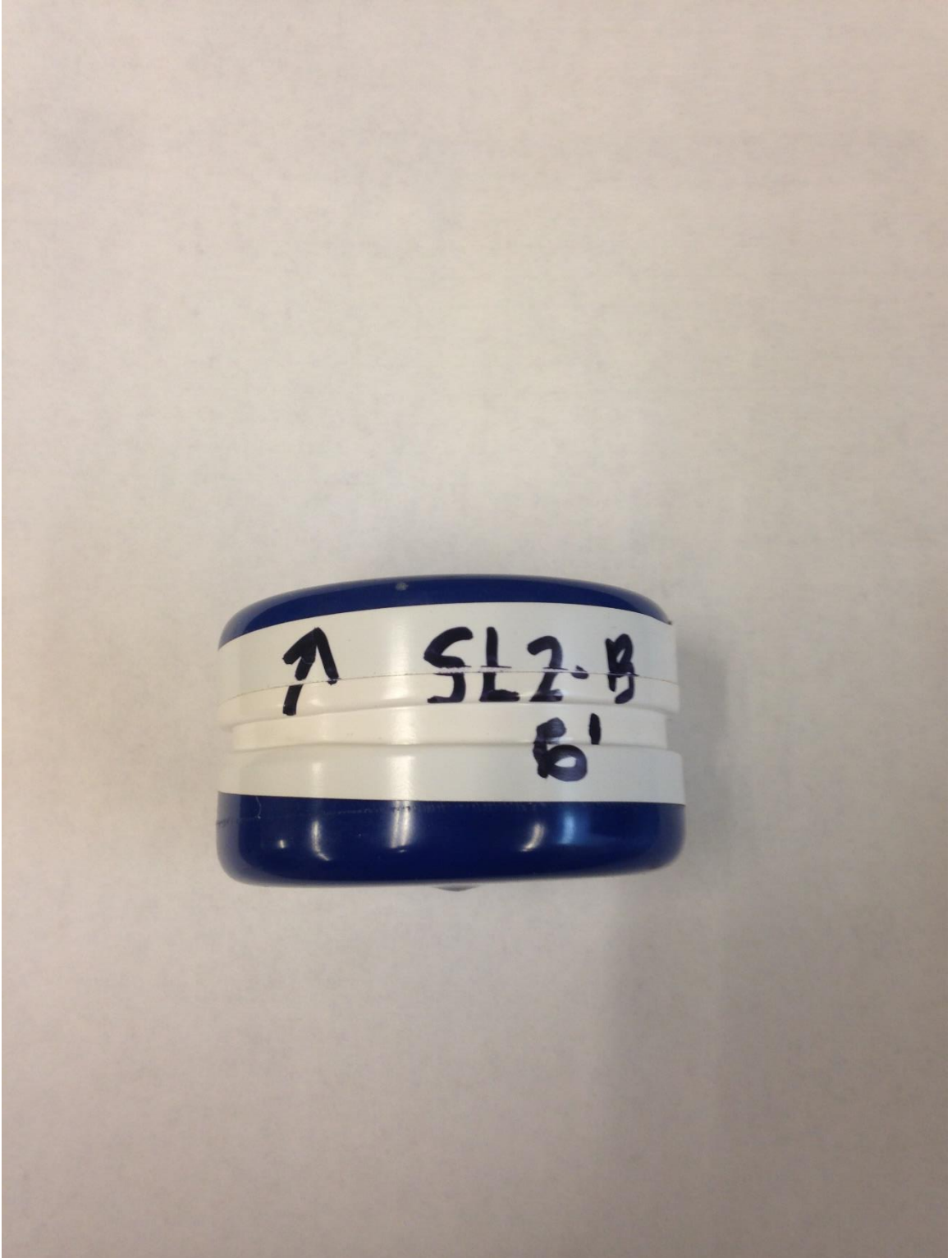


Figure 11: Wrapped soil core



Figure 12: Rotosift used for sieving

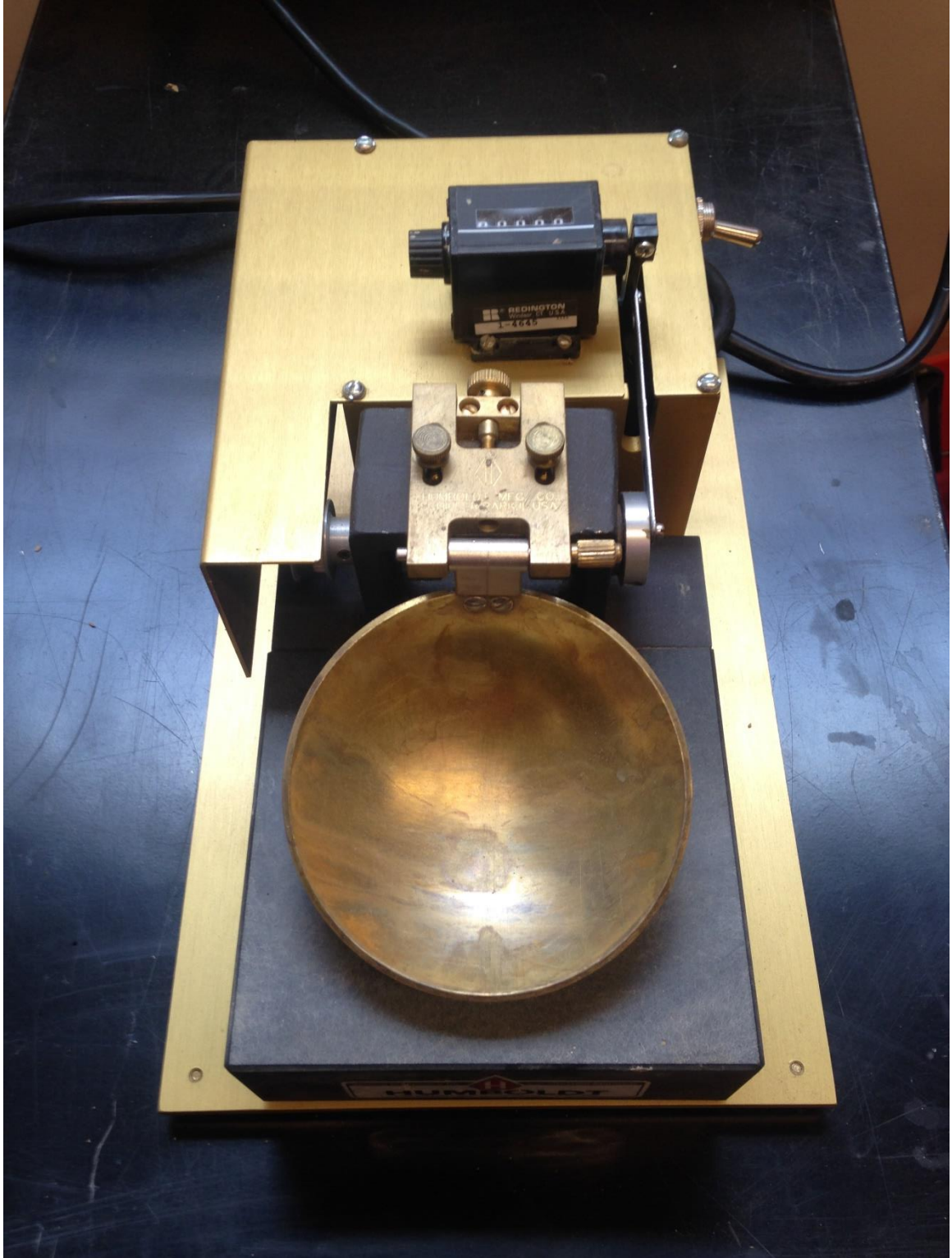


Figure 13: Automatic liquid limit machine

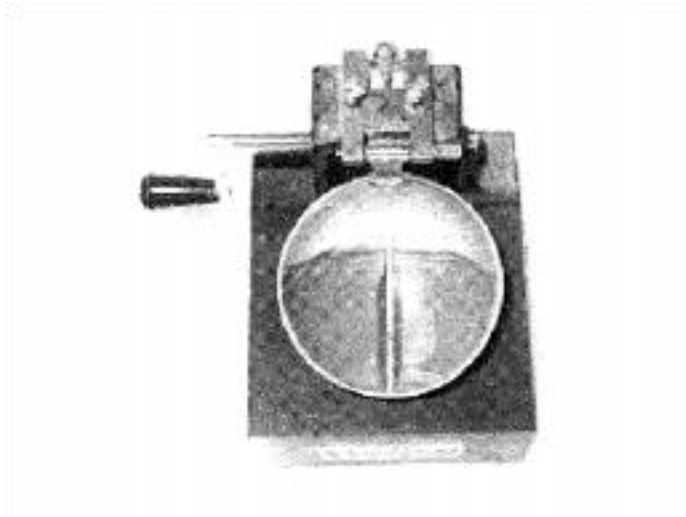


Figure 14: Brass dish with soil and groove



Figure 15: Example of plastic limit roll



Figure 16: Aardvark Permeameter Kit

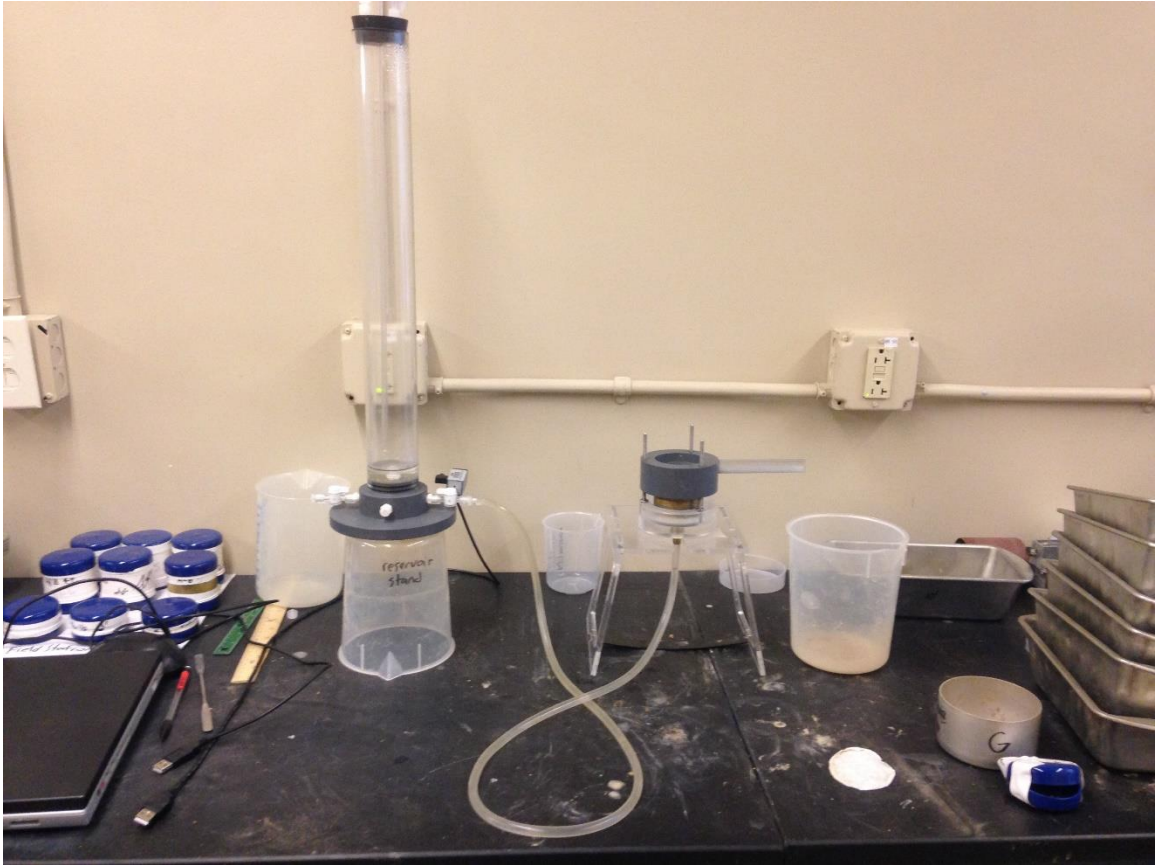


Figure 17: 2816G1 Chameleon Station

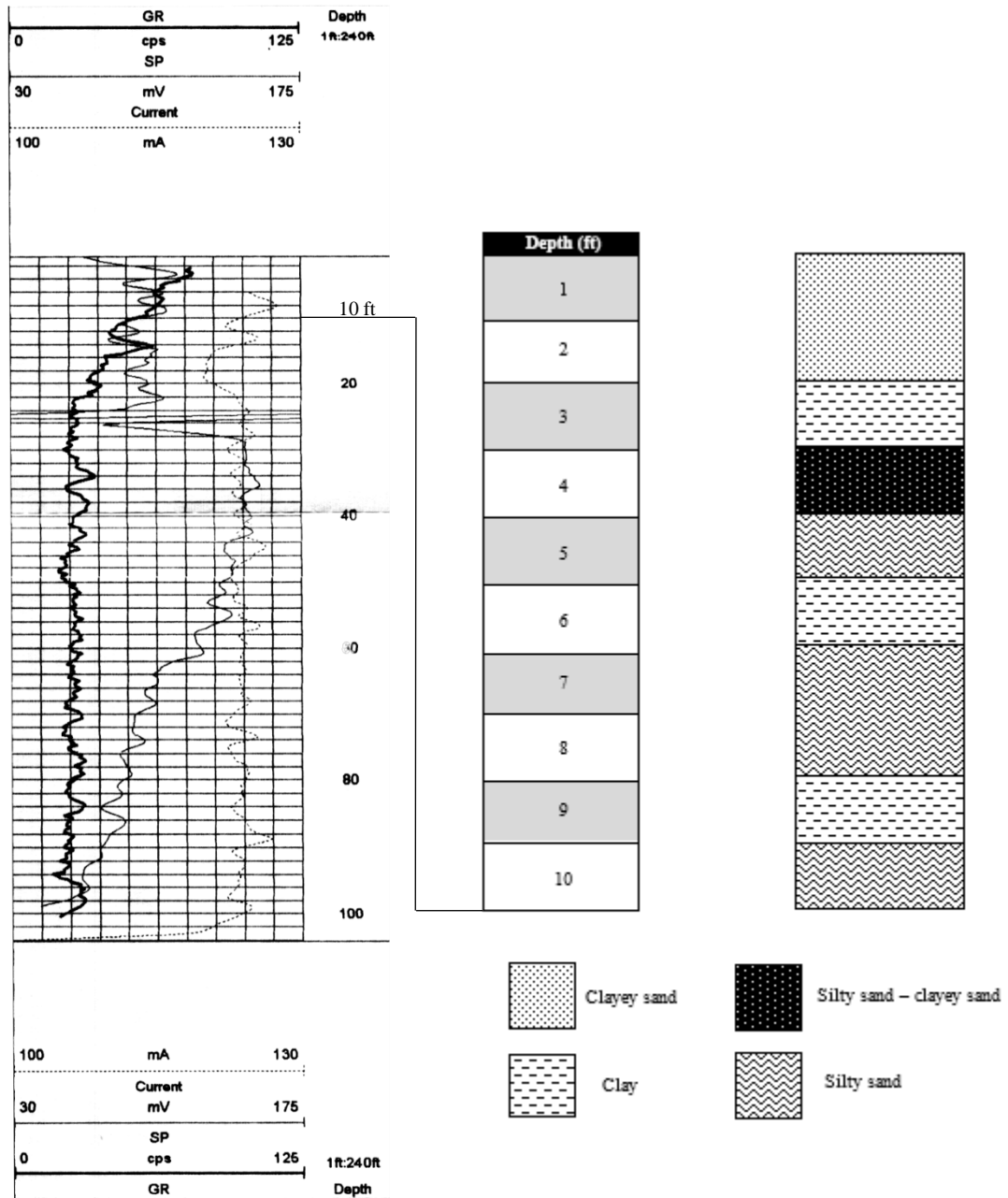


Figure 18: Geophysical log for site one with soil profile (Moore, 2017) correlation

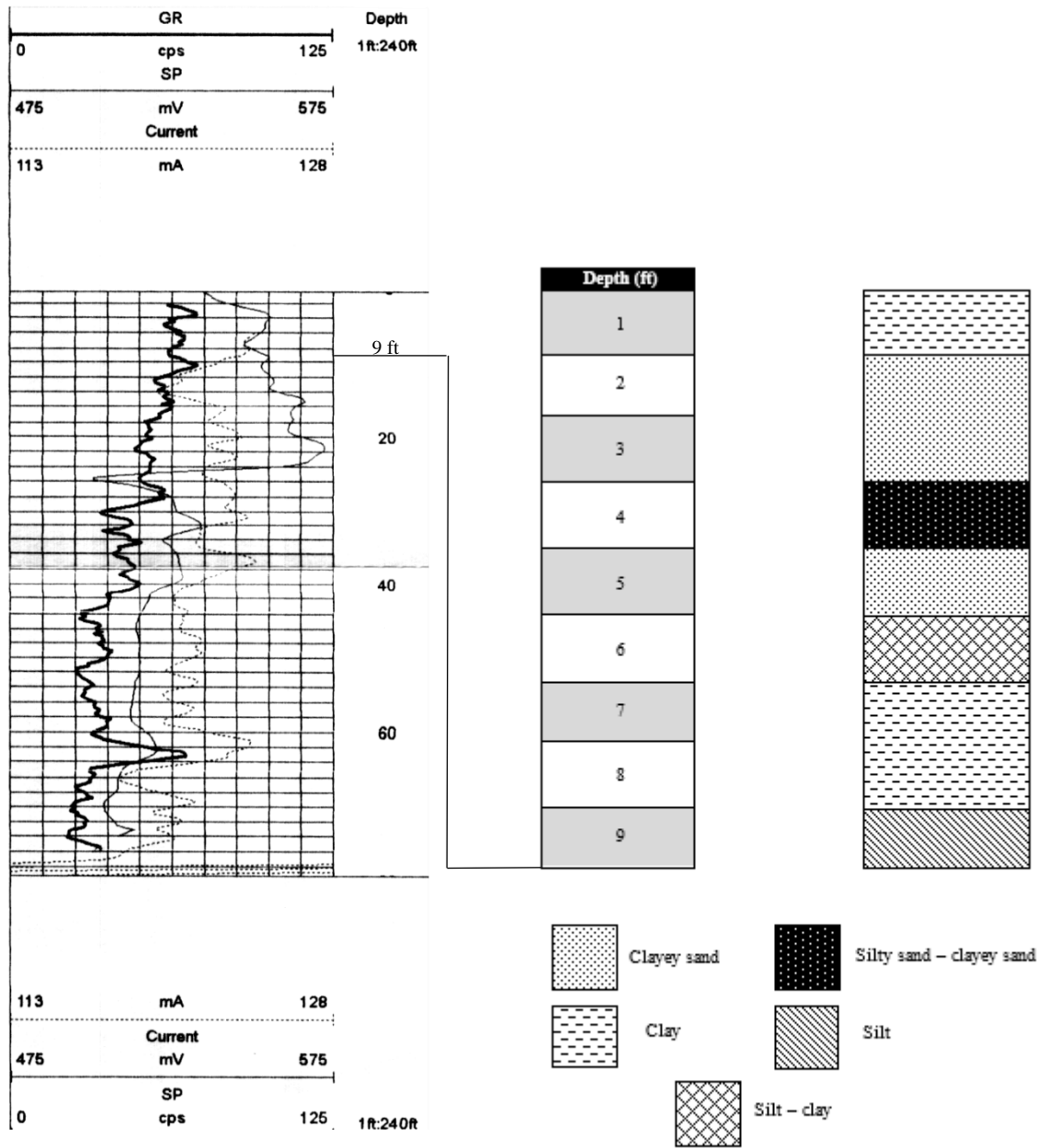


Figure 19: Geophysical log for site two with soil profile (Moore, 2017) correlation

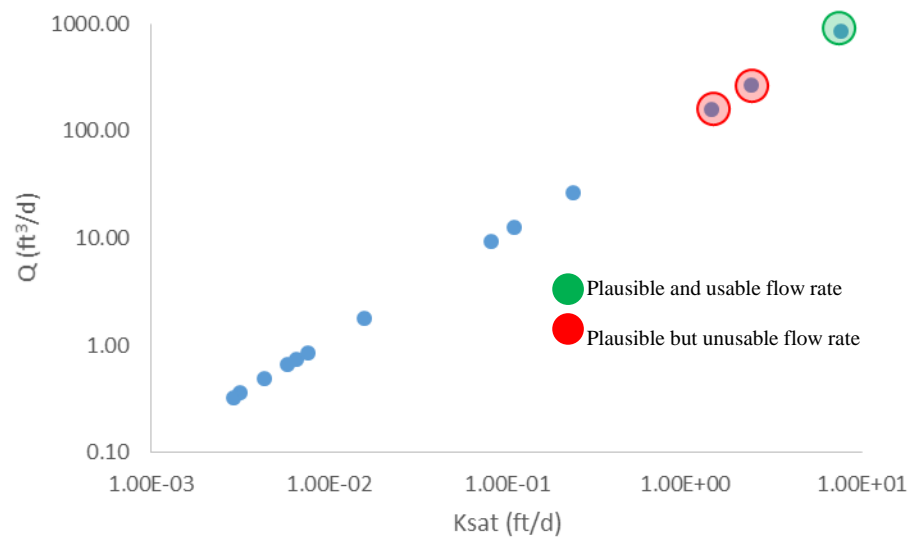


Figure 20: Graph of Q and K_{sat} relationship for ft³/day

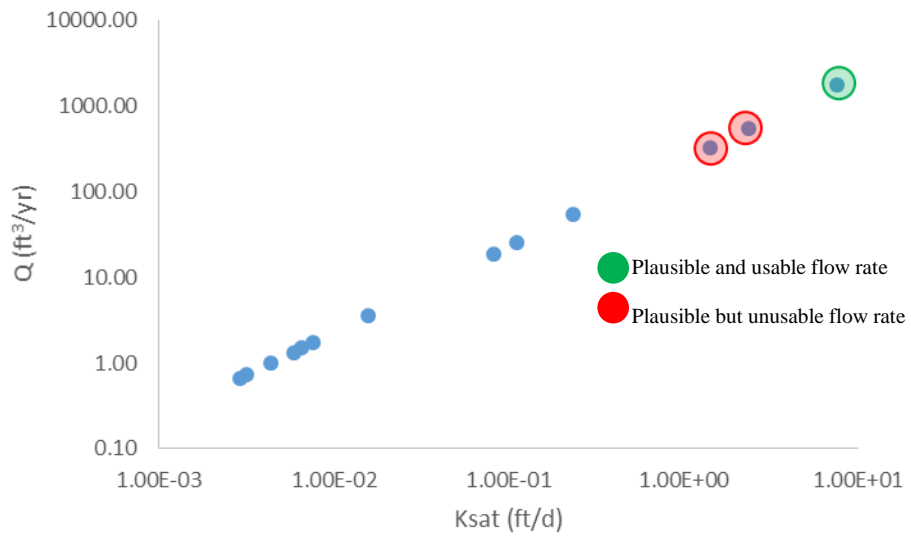


Figure 21: Graph of Q and K_{sat} relationship for ft^3/yr

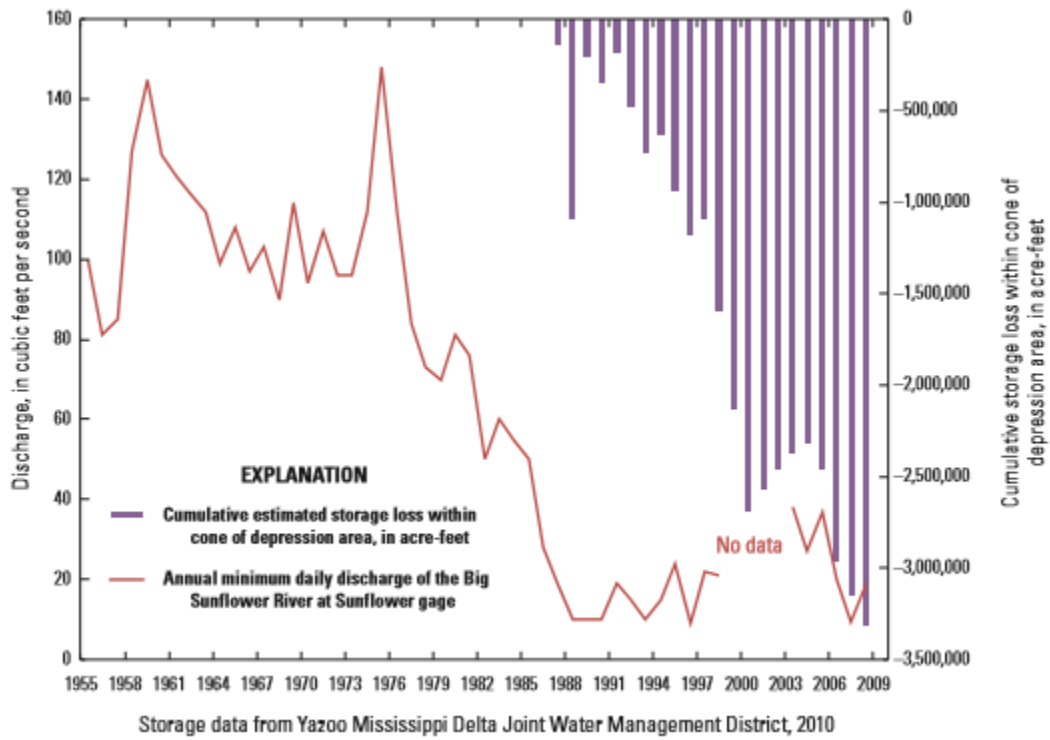


Figure 22: Graph showing storage data in MRVAA (Barlow and Clark, 2011)

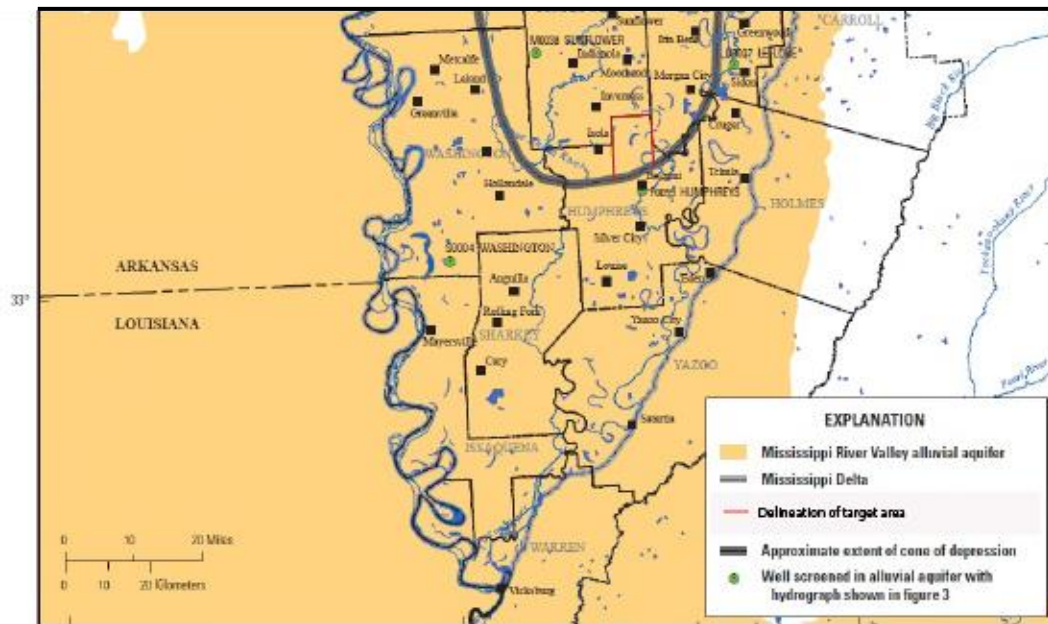


Figure 23: Target area, delineated in red at southern end of cone of depression (Modified from Barlow and Clark, 2011)

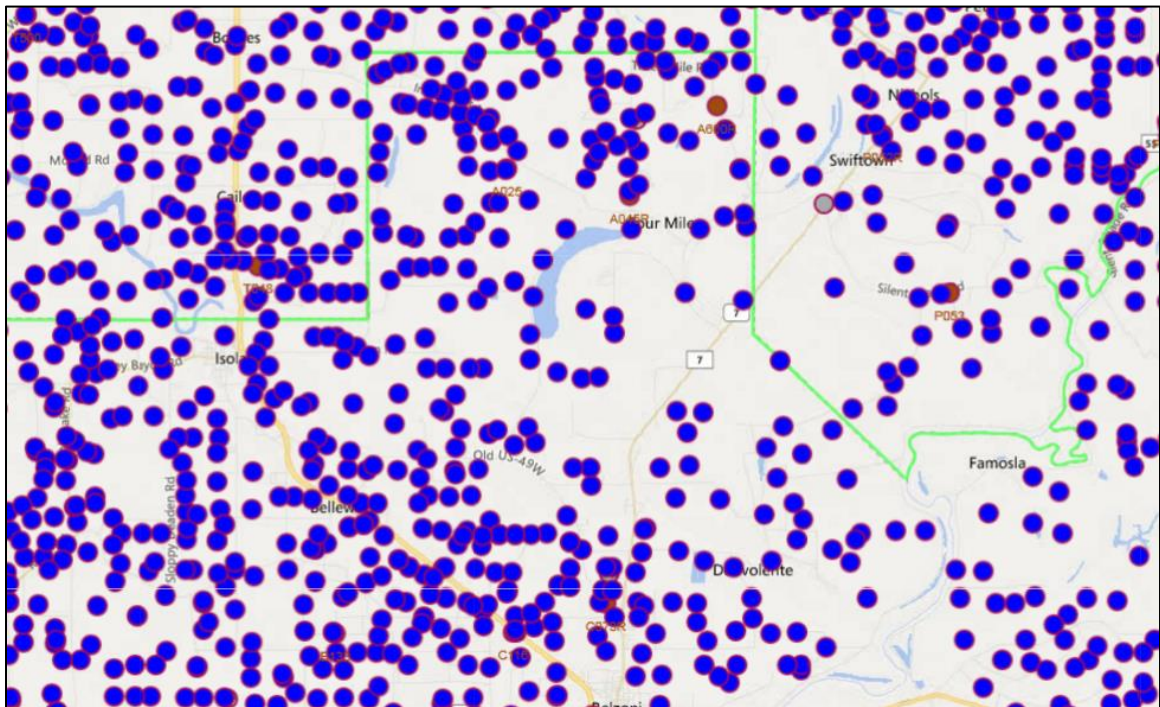


Figure 24: Map showing all permitted wells in target area and surrounding area. Blue wells indicate permitted wells, and red wells indicate monitoring wells. (YMD, 2017, YMD mapping: accessed March 17, 2017, available at <https://ymd.map.beehere.net/>)

Appendix A

The following data is listed electronically:

1. Sieve data lab sheets
2. Sieve data excel sheets

Appendix B

The following data is listed electronically:

1. Atterberg limits data lab sheets
2. Atterberg limits data excel sheets

Appendix C

The following data is listed electronically:

1. Field constant-head permeameter data sheets
2. Lab falling-head permeameter data sheets