ANDREWS ENGINEERING
3300 Ginger Creek Drive | 217.787.2334
Springfield, IL 62711

Ex. D15

February 1, 2022

Hand-Delivered

Illinois Environmental Protection Agency
DWPC – Permits MC #15
Attn: Part 845 Coal Combustion Residual Rule Submittal
1021 North Grand Avenue East
Springfield, IL 62794-9276

Re: City Water, Light and Power
Closure Construction Permit Application

Dear Correspondent:

On behalf of City Water, Light and Power, submitted herein is a Closure Construction application for the Lakeside Ash Pond and Dallman Ash Pond pursuant to Ill. Adm. Code 845.750. The CCR 1, CCR 2CC, and CCR 2CA forms are provided in Attachment A of the application. A digital copy of the application was uploaded to the following address: https://filet.illinois.gov/filet/PIMupload.asp.

If you have any questions or require further information, please contact P.J. Becker at (217) 757-8610, extension 1110. Thank you.

Sincerely,

Brad J. Hunsberger

Brad Hunsberger, LPG
Vice President

w/attachments

cc: P.J. Becker – CWLP (hard copy and electronic file)
Deborah Williams – CWLP (electronic file)
Eric Staley – CWLP (electronic file)
Tabitha Bernhardt – Illinois EPA (electronic file)
Paul Van Metre - Andrews Engineering (cover letter only)

ATTACHMENT 2

CONTAMINANT TRANSPORT MODEL

City Water, Light & Power

CCR Impoundments – Lakeside and Dallman Ash Ponds

Springfield, Sangamon County, Illinois

Closure Alternatives Assessment - Contaminant Transport Model

October 2021

Prepared for:
City Water, Light & Power
3100 Stevenson Drive
Springfield, Illinois 62703

ANDREWS ENGINEERING

3300 Ginger Creek Drive, Springfield, IL 62711 | 217.787.2334

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TABLE OF CONTENTS

EXECUTIVE SUMMARY ..... **I**
1. INTRODUCTION **1**
2. SURFACE IMPOUNDMENT CONSTRUCTION **3**
3. SUBSURFACE CONDITIONS AT THE CCR SURFACE IMPOUNDMENTS **4**
3.1 Surficial Deposits 5
3.2 Uppermost Bedrock 8
3.3 Hydrogeologic Units 8
3.4 Groundwater Movement 9
4. CONCEPTUAL MODEL **10**
4.1 Conversion Assumptions 10
4.2 Transport Processes 11
5. MATHEMATICAL MODEL **12**
5.1 Model Input and Sensitivity Analysis 13
5.1.1 Two-Dimensional Profile Setup 13
5.1.2 Closure-In-Place Model Input Parameters 13
5.1.2.1 Source Concentration 13
5.1.2.2 Base Width and Surface Width 14
5.1.2.3 Layer Thickness and Number of Sublayers 14
5.1.2.4 Dry Density and Distribution Coefficient 14
5.1.2.5 Effective Porosity 14
5.1.2.6 Darcy Velocity 15
5.1.2.7 Coefficients of Hydrodynamic Dispersion 17
5.1.2.8 Lateral Distances 20
5.1.2.9 Integration 21
5.1.3 Closure-By-Removal Model Input Parameters 21
6. CONTAMINANT TRANSPORT RESULTS **22**
6.1 Closure-In-Place 22
6.2 Closure-by-Removal 24
7. CCR SURFACE IMPOUNDMENT CONTAMINANT TRANSPORT EVALUATION RESULTS **27**
8. CONCLUSIONS **28**

FIGURES

Figure 1 – Site Location
Figure 2 – Groundwater Monitoring Network
Figure 3 – 1976 Site Survey
Figure 4 – Conceptual Model

APPENDICES

Appendix A: Hydrologic Evaluation of Landfill Performance (HELP) Output
Appendix B: MIGRATE Model Input and Output Files

EXECUTIVE SUMMARY

The Illinois Environmental Protection Agency (Illinois EPA) Standards for the Disposal of Coal Combustion Residuals in Surface Impoundments (35 Illinois Administrative Code (Ill. Adm. Code) 845) establishes requirements for the disposal of coal combustion residuals (CCRs or coal ash) in surface impoundments. The Illinois EPA standards for CCR surface impoundments include closure requirements. Pursuant to 35 Ill. Adm. Code 845 Subpart G, the owner or operator of an existing CCR surface impoundment must initiate closure where it cannot be demonstrated that the existing CCR surface impoundment:

- meets the location restrictions (35 Ill. Adm. Code 845.300 – 340),
- a safety factor assessment has not been completed or filed (35 Ill. Adm. Code 845.460), and/or
- is unlined (35 Ill. Adm. Code 845.400(f)).

A safety factor assessment has been completed and filed pursuant to Section 845.460(f); however, the existing CCR surface impoundments at the City Water, Light & Power (CWLP) impoundments fail to meet the location restrictions (Section 845.300) for distance from an aquifer and the lack of a liner meeting the requirements of CCR surface impoundments (Section 845.400(f)). As a result, the CWLP impoundments must initiate closure pursuant to Section 845.700.

In accordance with Section 845.700(c) the owner or operator of the CCR surface impoundment required to close under Section 845.700(a), or electing to close under Section 845.700(b), must immediately take steps to categorize the CCR surface impoundment under Section 845.700(g) and to comply with the closure alternatives analysis requirements in Section 845.710. Pursuant to Section 845.710(d)(2) for each alternative considered, the closure alternatives analysis must contain the results of groundwater contaminant transport modeling and calculations showing how the closure alternative will achieve compliance with the applicable groundwater protection standard (GWPS).

Two options for closure of a CCR surface impoundment are identified under Section 845.710(a). These options include either leaving the CCR in place and installing a final cover system (“closure-in-place”) or through removal of the CCR and clean closure of the CCR surface impoundment (“closure-by-removal”).

Closure-in-place is achieved by installing a final cover system over the existing CCR surface impoundments. Under this scenario, construction of the final cover system would begin October 2023 and be completed by January 2026. Following the completion of closure, a 30 year post-closure care period would commence that includes cover maintenance and continued groundwater monitoring. While closure-in-place does not remove the source material, installation of a final cover minimizes infiltration of liquids into the CCR and mitigates the potential for leaching of CCR solutes into the uppermost aquifer. This option also minimizes impacts to human health due to accidental inhalation/ingestion of wind-blown CCR from excavation and trucking activities associated with the closure-by-removal alternative.

Closure-by-removal is achieved by dewatering the surface impoundment and excavation of the CCR, transporting the material to a lined landfill or other appropriate disposal facility, reshaping the site and back-filling it, as necessary, to retain structural and/or hydrologic properties. The duration of closure-by-removal would be largely dependent on a number of factors including the logistics associated with drying out the CCR and loading it into trucks, and the amount of borrow

material that must be transported to the site to back-fill the excavation in order to retain structural stability in certain areas. Under this scenario, CCR removal activities would commence October 2023 and be completed by January 2030. Closure-by-removal does not require a post-closure care period; however, pursuant to 35 Ill. Adm. Code 845.740(b) after closure-by-removal has been completed, the owner or operator must continue groundwater monitoring under 35 Ill. Adm. Code 845 Subpart F for three years after the completion of closure or for three years after groundwater monitoring does not show an exceedance of the GWPS, whichever is longer. While closure-by-removal will result in the removal of the CCR material from the surface impoundments, this alternative will likely increase the potential for leaching of CCR solutes into the uppermost aquifer during excavation activities and increase the potential for exposure due to accidental inhalation/ingestion of wind-blown CCR from dewatering, excavation and trucking activities. Other issues not addressed in this evaluation of the closure-by-removal scenario but most notable include the generation of additional green-house gases as a result of the associated trucking and excavation activities and the increased risk of accidents as a result of the increased duration and frequency of off-site trucking.

Using the Hydrologic Evaluation of Landfill Performance (HELP v3.07) model and the two-dimensional contaminant transport model MIGRATE v9, an evaluation of the closure-in-place and closure-by-removal alternatives were conducted for the CCR surface impoundments at the CWLP facility. The conceptual model was prepared to represent the impoundment design, site-specific hydrogeology and conservative performance standards for the cover. This evaluation predicts that closure-in-place will minimize infiltration and leaching of CCR constituents into the uppermost aquifer resulting in a decrease in CCR concentrations within approximately 13 years of installation of the final cover system.

MIGRATE was also used to predict concentrations in the uppermost aquifer under the closure-by-removal option. As with the closure-in-place scenario, the conceptual model represents the facility design and site-specific hydrogeology. As expected, the contaminant transport model for the closure-by-removal option predicts a decrease in CCR concentrations within approximately 7 years after the completion of the CCR removal activities.

1. INTRODUCTION

CWLP owns two existing CCR surface impoundments subject to the Illinois EPA Standards for the Disposal of Coal Combustion Residuals in Surface Impoundments (35 Ill. Adm. Code 845).

The CWLP CCR surface impoundments are located north and east of the former Lakeside Power Generating Station and Dallman Power Generating Station in the Eastern ½ of Section 12, Township 15 North, Range 5 West, in Springfield, Illinois (see Figure 1). These CCR surface impoundments are identified as the Lakeside Ash Pond and the Dallman Ash Pond (see Figure 2).

The Lakeside Ash Pond was placed into service prior to 1958 and ceased receiving ash in 2009. It has been divided into four separate ponds, three limesludge ponds and the settling pond consisting of approximately 35.0 acres. The Lakeside Ash Pond currently receives limesludge from the CWLP Drinking Water Purification Plant, scrubber wastewater treatment plant clarifier blowdown and water from miscellaneous floor drains. Construction of new lined limesludge ponds will start in spring of 2022 and be completed by no later than fall of 2023. CWLP will initiate closure of the Lakeside Ash Pond after the water purification plant limesludge ponds are relocated.

The Dallman Ash Pond was placed into service in 1976 and is approximately 34.5 acres. The Dallman Ash Pond currently receives fly ash and bottom ash, which are sluiced with raw lake water, industrial wastewater treatment plant clarifier blowdown and landfill leachate. CWLP has shut down Dallman Units 31, 32 and will be shutting down Dallman Unit 33 no later than September 15, 2023.

The existing CCR surface impoundments fail to meet the location restrictions (Section 845.300) for distance from an aquifer and are considered unlined CCR surface impoundments (Section 845.400(f)). As a result, CWLP must initiate closure pursuant to 35 Ill. Adm. Code Subpart G.

In accordance with Section 845.700(c), the owner or operator of the CCR surface impoundments are required to close pursuant to Section 845.700(a) or Section 845.700(b) and must comply with the closure alternatives analysis requirements in Section 845.710. Pursuant to Section 845.710(d)(2), for each alternative considered, the closure alternatives analysis must contain the results of groundwater contaminant transport modeling and calculations showing how the closure alternative will achieve compliance with the applicable groundwater protection standard (GWPS).

Two options for closure of a CCR surface impoundment are identified under Section 845.710(a). These options include either by leaving the CCR in place and installing a final cover system (“closure-in-place”) or through removal of the CCR and decontamination of the CCR surface impoundment (“closure-by-removal”).

The objective of this submittal is to present the results of groundwater contaminant transport modeling and calculations showing how each closure alternative will achieve compliance with the applicable GWPS. Both closure alternatives - closure-in-place and closure-by-removal, are evaluated within this document.

For the closure-in-place contaminant transport model, vertical seepage rates as a result of installation of a final cover system were determined using the US EPA hydrologic water balance model HELP (Hydrologic Evaluation of landfill Performance v3.07). MIGRATE v9, a two-

dimensional contaminant transport model, was used for predicting contaminant concentrations for both the closure-in-place and closure-by-removal scenarios.

This evaluation demonstrates that closure-in-place with a final cover system minimizes infiltration of liquids through the CCR materials significantly reducing CCR constituent concentrations to below the established GWPSs. Similarly, the closure-by-removal closure alternative results in a reduction of CCR constituent concentrations to below the established GWPS.

The following narrative provides a discussion of the design characteristics of the CCR surface impoundments, the geologic and hydrogeologic conditions present beneath the impoundments, the development of the conceptual model, contaminant transport model and the model input parameters and results. The conceptual model for this contaminant transport evaluation is based upon the hydrogeologic setting and design characteristics of the Dallman Ash Pond as it represents the worst-case scenario for determining impacts at the down-gradient perimeter of the impoundments.

2. SURFACE IMPOUNDMENT CONSTRUCTION

The Dallman Ash Pond was placed into service in approximately 1976 and is a diked embankment, covering approximately 34.5 acres in the Sugar Creek drainageway immediately down-gradient of Spaulding Dam and Lake Springfield.

Based on the 1976 Burns and McDonnell construction topographic map (see Figure 3), Sugar Creek meandered through the property. However, the creek channel was abandoned and rerouted to its present location when the surface impoundments were constructed. Abandonment of the creek channel included the placement of different types of soil, ranging from cohesive soils characterized as silty clays, to granular fill characterized as poorly graded silty to clayey sands. Soil analyses of the recompacted channel fill materials were conducted during hydrogeological investigations performed for the adjacent CCR landfill. The hydraulic conductivity of these creek fill materials was reported to range from 7.6×10^{-8} to 2.1×10^{-5} cm/sec[1].

The berms for the Dallman Ash Pond were built to a height of approximately 27 feet (554 feet MSL), using slopes of 2.5H:1V for both the inner and outer slopes. The elevation of the base of the surface impoundments is approximately 527 feet MSL.

The berms of the surface impoundment were constructed of glacial till consisting of silty clay to clayey silt soil. Recompacted silty clay samples from the native soils have exhibited hydraulic conductivities in the 1×10^{-7} to 1×10^{-9} cm/sec range. The in-place creek sediment's soil hydraulic conductivities predominantly range from 1×10^{-6} to 1×10^{-8} cm/sec.

Prior to the area development, the upper layer of soil at the site consisted of mainly brown, light brown, and brownish-gray silty clays and clayey silts having soft to stiff consistency. This includes all eolian soils (loess) deposited near the surface, isolated pockets and lenses of fine-grained silty to clayey sand at some locations and alluvial silts and silty clays. Much of the shallow soils were displaced during area development.

[1] Patrick Engineering – Hydrogeologic Report on Flue Gas Desulfurization Sludge Landfill, June 1993.

3. SUBSURFACE CONDITIONS AT THE CCR SURFACE IMPOUNDMENTS

The subsurface conditions of the area in and surrounding the CCR surface impoundments have been characterized through multiple subsurface investigations, including those prior to the impoundment development, along the perimeter of the impoundments and including the hydrogeologic investigation at the permitted Subtitle D CCR landfill located north of the Lakeside Ash Pond and east of the Dallman Ash Pond. These investigations were as follows:

- Professional Service Industries (PSI), June 1989. Five soil borings were advanced within the east section of the south cell (Cell 1) of the CCR landfill.

- Andrews Environmental Engineering, Inc. (AEEI), February 1990. This investigation was performed for Cell 2 of the CCR landfill and consisted of the advancement of 13 soil borings. Drilling and testing were completed by PSI.

- Andrews Environmental Engineering, Inc. (AEEI), March 1990. Six wells were installed along the perimeter of the CCR landfill. Drilling and testing were completed by PSI.

- Patrick Engineering, Inc. (PEI), July 1992. This investigation was performed to further characterize the hydrogeology of the CCR landfill setting. Approximately 44 soil borings and piezometers were installed by PEI.

- Stabilize, Inc. (SI), December 2008. Three monitoring wells were installed as part of an assessment program for the CCR landfill. Drilling, soil testing, and well construction were performed by Reynolds Well Drilling.

- City Water, Light and Power (CWLP), April 2010. Four piezometers were installed on the west side of the CCR surface impoundments along Sugar Creek. Drilling and testing were completed by PSI.

- Stabilize, Inc. (SI), May 2011. Four monitoring wells were installed to further the characterization of the CCR surface impoundments. Drilling, soil testing, and well construction were performed by PSI.

- Andrews Engineering, Inc. (AEI), January 2012. This investigation was performed to replace damaged CCR surface impoundment wells and to install an additional background well. Drilling and well installation were completed by TerraDrill.

- Andrews Engineering, Inc. (AEI), July 2017. This project included drilling peripheral to the Dallman Ash Pond.

- Andrews Engineering, Inc. (AEI), September and October 2018. This project included borings peripheral to the Dallman Ash Pond.

- Andrews Engineering, Inc. (AEI), May and July 2019. This project included advancement of borings drilling peripheral to the Dallman Ash Pond.

- Andrews Engineering, Inc. (AEI), February 2021. This project included installation of four assessment wells located west of Sugar Creek.

The investigations and related work conducted to date have provided sufficient information to characterize the site hydrogeologic conditions, including the uppermost aquifer and the uppermost confining unit. Further refinement of the site-specific hydrogeologic and geochemical conditions of the site will be further enhanced with ongoing CCR groundwater monitoring and supplemental subsurface investigations.

The summary discussion below of the subsurface conditions is presented in context of 35 Ill. Adm. Code Part 845 requirements for groundwater monitoring systems. Emphasis is placed on characterizing the existing wells and hydrogeologic conditions of the uppermost aquifer and underlying confining aquitard. The uppermost aquifer is the required unit for groundwater quality monitoring pursuant 35 Ill. Adm. Code Part 845.

The occurrence of the deposits discussed below are highly variable due to the meandering nature of Sugar Creek prior to the development of Lake Springfield and Spaulding Dam. The meandering creek has resulted in sequential erosion and deposition (scour and fill) throughout much of the creek drainage system, both laterally and vertically.

3.1 Surficial Deposits

The shallow stratigraphy and lithology at the CCR surface impoundments include approximately 20 to 50 feet of Pleistocene sediments, dependent upon location. In ascending order (i.e., oldest to youngest) these materials are identified as basal sand, lower cohesive deposit, shallow sand, upper cohesive deposit and fill material. As described below, the uppermost unit directly beneath the CCR surface impoundment is the lower cohesive deposit.

The overall tendency is for the finer-grained materials (clays, silty clays and silts) to overlie the coarser-grained materials (sands and gravels). This coarsening downward is present throughout much of the site. At the majority of the borehole locations, the coarser materials rest directly on top of the weathered bedrock surface. It is this coarser material, the basal sand, which is characterized as the uppermost aquifer.

Basal Sand

In most locations, the basal sand is the lower-most surficial deposit. The basal sand is a gray colored, poorly graded, silty to clayey fine sand to well graded sand with minor amounts of fine gravel. This unit was encountered in a medium dense to dense condition. Across the CWLP facility (includes the Dallman and Lakeside Ash Ponds), the top elevation of the basal sand varies from 491 to 513 feet mean sea level (MSL) and the thickness ranges from about 0 to 12.3 feet. Along the down-gradient (northern) edge of the Dallman Ash Pond area, the basal sand was found to range from 3 to 11.5 feet in thickness, and have an average thickness of 7.3 feet. The unit was not consistently encountered, likely due to excessive erosion of the creek bottom.

The basal sand generally overlies the bedrock surface and underlies the lower cohesive deposit. There are some pockets of very hard, fine-grained silty clay to clay overlying bedrock in a few areas. The basal sand is present above these pockets of clayey deposits, thought to be weathered bedrock.

The basal sand generally consists of 0% to 34% gravel, 50% to 91% sand, and 6% to 44% silt/clay; and exhibits a horizontal hydraulic conductivity of ranging from 1.15 x 10^{-3} to

7.64 x 10^{-2} cm/sec. The geometric mean hydraulic conductivity is 1.25 x 10^{-2} cm/sec. The basal sand was saturated in all locations where it was encountered.

Lower Cohesive Deposit

The lower cohesive deposit consists of brown, gray, and brownish gray silty clays, clayey silts, and clays, having very soft to stiff consistency. Across the CWLP facility, the lower cohesive deposit is as much as 22 feet thick with an average thickness of about 15 feet. However, along the down-gradient (northern) edge of the Dallman Ash Pond the thickness of the lower cohesive deposit was found to range from 12.6 to 26.3 feet in thickness, and have an average thickness of 19.4 feet. The deposit was not encountered in isolated areas along the abandoned creek, possibly due to excessive erosion of the creek bottom in these areas. The bottom of the CCR surface impoundment rests directly on top of the lower cohesive deposit.

The lower cohesive deposit is generally overlain by the shallow sand and underlain by the basal sand. However, within the abandoned creek area, the lower cohesive deposit was encountered directly below the creek fill. In some areas the basal sand is not present and the lower cohesive deposit directly overlies the bedrock.

The soils in the lower cohesive deposit can be similar in color and texture to the soils in the upper cohesive deposit. The distinction between the two deposits was based on the presence or changes in soil consistency (as measured with a calibrated hand held penetrometer) and a marked difference in moisture content. The lower cohesive deposit is not exposed at the ground surface in the investigated area.

The lower cohesive deposit consists of 0% gravel, 8% to 48% sand, and 52% to 95% silt/clay; and has a low hydraulic conductivity. The vertical hydraulic conductivity ranges from 1.3 x 10^{-8} to 1.8 x 10^{-6} cm/sec (triaxial permeameter). Based on these values, the average vertical hydraulic conductivity is 9.07 x 10^{-7}. The horizontal hydraulic conductivity ranges from 4.6 x 10^{-5} to 7.6 x 10^{-5} cm/sec.

Shallow Sand

The shallow sand overlies the lower cohesive deposit and underlies the upper cohesive deposit. The unit consists of a brown to gray silty to clayey fine sand. It contains small lenses of silty clay and clayey silt. This unit is not continuous over the entire site. Its thickness ranges from one to three feet over most of the investigated area. The shallow sand does not underlay the CCR surface impoundment.

Laboratory tests performed on representative samples collected from the shallow sand unit during this and previous investigations indicate the shallow sand contains 0% gravel, 50% to 52% sand, and 48% to 50% silt/clay. Two CCR landfill piezometers were screened in the shallow sand unit to obtain potentiometric surface information and conduct field hydraulic conductivity tests. The hydraulic conductivity of this unit based on the slug test results ranges from 3.6 x 10^{-3} to 2.9 x 10^{-2} cm/sec.

Upper Cohesive Deposit

The upper cohesive deposit is an alluvial deposit consisting of brown, light brown to brownish-gray silty clays to clayey silts having soft to stiff consistency. The unit includes loess deposits and isolated pockets of fine-grained silty to clayey sand and at some location alluvial silts and silty clays. The upper cohesive deposit is laterally adjacent to the sidewall of the CCR surface impoundment.

The upper cohesive deposit has a low vertical hydraulic conductivity as determined by laboratory triaxial hydraulic conductivity tests from borings taken from the landfill investigation. The hydraulic conductivity values determined from the laboratory tests ranged from 1.6 x 10^{-5} cm/sec to 5.2 x 10^{-7} cm/sec. It is expected that the horizontal coefficient of hydraulic conductivity is greater than the vertical coefficient. Based on test results for the lower cohesive deposit, it is anticipated that the horizontal hydraulic conductivity for the upper cohesive deposit is in the range of 10^{-5} to 10^{-6} cm/sec.

Creek Fill Material

Borings conducted along the abandoned creek locations indicate that the creek fill materials consist of variable soils ranging from silty clays to silty sands. Cohesive soils characterized as silty clay to organic silty clay were typically encountered. In some areas, the cohesive fill materials extended down to the top of bedrock. Where encountered, the granular fill materials are typically poorly graded silty to clayey sands and contain organics or wood fragments. In some areas, the granular fill materials also extended down to the top of bedrock.

The cohesive fill material contains 0% gravel, 2% to 48% sand, and 52% to 98% silt/clay. The vertical hydraulic conductivity ranges from 7.6 x 10^{-8} cm/sec to.2.1x10^{-5} cm/sec. The granular fill materials contain 0 to 2% gravel, 55% to 65% sand and 33% to 45% silt/clay. Based on one laboratory hydraulic conductivity test performed on a Shelby tube sample obtained from berm fill, the hydraulic conductivity of the granular fill material is 3.3 x 10^{-8} cm/sec.

The creek fill materials identified during the previous landfill investigations have a significant effect on the site hydrogeologic conditions. In some areas the fill materials, consisting of either granular soils or organic silty clays, extend from existing grade to the bedrock surface and locally interconnect all three water bearing units. The upper and lower cohesive deposits are considered to act as aquitards (where present) which restrict vertical flow into the water bearing units. For all practical purposes, the bedrock is considered to be an aquiclude (i.e., an impermeable body of rock or stratum of sediment that acts as a barrier to the flow of water).

Fill materials encountered in the landfill borings range from silty clays and organic silty clays to silty sands and clayey sands. These water level measurements indicate that groundwater movement within the creek fill materials is complicated because of the highly variable hydraulic characteristics of the fill materials and random placement. In some areas, there appears to be in direct hydraulic communication between fill materials, shallow sand, and basal sand.

Four landfill piezometers are screened across the fill materials. Of these, one piezometer was installed across cohesive fill material and the other piezometers were installed across granular fill materials. Hydraulic conductivity of the granular fill materials is based on one field test resulting in a value of 6.1 x 10^{-2} cm/sec. The hydraulic conductivity of the cohesive fill material ranged from 7.1 x 10^{-5} cm/sec to 1.1 x 10^{-4} cm/sec. These values represent the hydraulic conductivity in the horizontal direction.

Laboratory hydraulic conductivity test performed on landfill cohesive fill materials ranged from 2.1x10^{-3} cm/sec to 3.3 x 10^{-8} cm/sec. The higher hydraulic conductivity values are believed to be typical of soils which contain organic matter (e.g. wood fragments). The hydraulic conductivity values based on laboratory tests are generally considered to be representative of the coefficient of hydraulic conductivity in the vertical direction because of the sample configuration during testing. However, because of the randomness of the fill, it is more likely that the hydraulic conductivity is within the range of 10^{-5} to 10^{-4} cm/sec.

3.2 Uppermost Bedrock

The bedrock at the project site consists of Pennsylvanian shales that are gray in color. The bedrock surface elevation varies from approximately 492 feet MSL near the center of the existing landfill, to approximately 554 feet MSL located on a bedrock outcrop near the landfill area (southeast corner of Cell 1). Along the down-gradient edge of the Dallman Ash Pond, the bedrock is at approximately 500 feet MSL. In general, the bedrock surface slopes from the east and west towards the center of the landfill area.

Rock Quality Designation (RQD) measurements were performed on all core samples taken from the CCR landfill area. RQDs measured from core samples collected during this investigation ranges from 80% to 100%. The RQD values indicate that the bedrock exhibits minimal to no fracturing. Two in situ hydraulic conductivity tests were performed to determine the hydraulic conductivity of the upper portions of the bedrock. Test results indicate hydraulic conductivity values of 1.8×10^{-7} cm/sec and 1.3×10^{-6} cm/sec. This shows that the bedrock encountered at the project site is a lower confining unit to the uppermost aquifer. There is good correlation between the lithology of the rocks tested and the hydraulic conductivity values obtained. The upper bedrock beneath the impoundments is expected to exhibit the same characteristics as encountered at the landfill.

3.3 Hydrogeologic Units

The uppermost aquifer and underlying confining unit control groundwater movement and the potential for CCR impacted groundwater migration at the site. A description of the uppermost aquifer and the underlying confining unit follows:

Uppermost Aquifer

The uppermost aquifer is characterized as the basal sand overlying the shale bedrock. In some CCR landfill locations the creek fill materials, ranging from silty clays and organic silty clays to silty sands and clayey sands, were found to be in direct hydraulic communication with the basal sand that directly overlies the bedrock surface. Due to the highly variable hydraulic characteristics and random placement of the creek fill materials, further characterization is difficult.

As indicated above, the hydraulic conductivity of the basal sand ranges from 1.15×10^{-3} to 7.64×10^{-2} cm/sec and exhibits a mean value of 1.25×10^{-2} cm/sec (3,942 m/a). The groundwater in the basal sand appears to be under confined, semi-confined, or unconfined conditions dependent upon location. The upper limit of the uppermost aquifer is dependent upon the seasonally fluctuating groundwater table. The potentiometric surface of the basal sand varies from 565 feet MSL at upgradient locations, south of the Lakeside Ash Pond, to 525 feet MSL at down-gradient locations near Sugar Creek, north of the Dallman Ash Pond. As a result, the saturated thickness is variably dependent upon the location and the seasonal variation in the groundwater table.

Lower Confining Unit

The uppermost bedrock at the project site is primarily Pennsylvanian age shale with isolated thin coal layers. The Pennsylvanian shale functions as a lower confining unit due to its low hydraulic conductivity and effective porosity. The lower confining unit represents a natural hydrogeologic barrier (i.e., aquitard) to the vertical movement of groundwater.

In situ hydraulic conductivity test (slug tests) indicate that the hydraulic conductivity for the upper portions of the bedrock range from 1.8×10^{-7} cm/sec (0.0568 m/a) to 1.3×10^{-6} cm/sec (0.410

m/a). There appears to be good correlation between the rock lithology and the measured values of hydraulic conductivity. The bedrock over most of the site will act as an aquiclude and prevent the downward movement of groundwater.

3.4 Groundwater Movement

Groundwater movement is controlled by recharge along topographic highs and discharge along the original stream valley. The pre-surface impoundment flow direction in the uppermost aquifer was dominantly horizontal from the adjacent banks toward the natural convergence along Sugar Creek, which formerly drained the site. The overall flow was from south to north with local deviations. This dominant flow pattern persists under present day conditions but with localized variation introduced by the hydrologic discontinuity created upon construction of the CCR landfill and CCR surface impoundments.

Existing wells at the site, including wells from the CCR landfill, were used to derive potentiometric surface maps. As discussed, overall groundwater flow is from south to north. The east perimeter of the Lakeside Ash Pond is largely up-gradient, and the west side is largely side-gradient where groundwater movement is parallel to the perimeter. Excavated areas within the landfill boundary act as a groundwater sink along the east side of the Dallman Ash Pond. Groundwater generally moves northward under the Dallman Ash Pond but also moves easterly towards the FGDS Landfill and westerly towards Sugar Creek. Groundwater beneath the Lakeside Ash Pond also moves northward and westerly along Sugar Creek.

The horizontal hydraulic gradient of the basal sand was calculated based on groundwater flow from well G120 to well RW-3. Using these wells, horizontal hydraulic gradients were calculated for four consecutive quarterly measurements dating from third quarter 2020 to second quarter 2021. As presented below, the horizontal hydraulic gradients within the basal sand ranges from 0.00794 to 0.0139. The average horizontal hydraulic gradient is 0.0119.

Groundwater Elevation and Hydraulic Gradient						
Well	**Northing**	**Easting**	**3rdQtr20**	**4thQtr20**	**1stQtr21**	**2ndQtr21**
RW-3	5613.3	1737.4	528.49	526.82	528.25	531.51
G120	5009.4	1986.9	536.49	535.91	537.12	536.7
Hyd grad G120 to AW-3			0.0122	0.0139	0.0136	0.00794
i_{avg} = 0.0119						

4. CONCEPTUAL MODEL

This section provides an overview of the simplifications to the site geology, hydrogeology, and design that are necessary to construct the conceptual model and to identify the transport processes that should be addressed.

4.1 Conversion Assumptions

To adequately express the site geology within the context of the contaminant transport model (MIGRATE v9) and to ensure a conservative approach to the selection of input parameters, several assumptions were made in the conversion of the site geology and design to the conceptual models.

1) **All geologic units and earthen structures are homogeneous and isotropic with respect to all lithologic and hydrogeologic parameters.** Most contaminant transport models are incapable of working with small-scale changes in parameter values that are typical of most geologic materials. Average parameter values were chosen to represent the lithologic and hydrogeologic parameters where variability occurs.

2) **All layers are laterally extensive and the thickness of each layer is uniform.** Layer thicknesses used within the model are more uniform than actually present at the site. The layer thicknesses used represent the average layer thickness at the down-gradient edge of the CCR surface impoundment.

3) **The geologic and hydrogeologic parameters used are mean values of site-specific data or mean values taken from the literature for similar materials. A conservative range of values is taken into consideration for parameter sensitivity.** The mean values analyzed provide a conservative analysis of the site conditions. Transport through a geologic unit with a variability of hydraulic conductivity, transmissivity, porosity, etc., will actually produce an "average" movement through the geologic unit.

4) **The competent bedrock below the uppermost aquifer is an impermeable boundary.** Hydraulic conductivity data indicate the underlying shale is relatively impermeable with hydraulic conductivities ranging from 1.8×10^{-7} cm/sec (0.0568 m/a) to 1.3×10^{-6} cm/sec (0.410 m/a).

5) **The compacted berm material consists of overburden material having a hydraulic conductivity of 1×10^{-7} cm/s or less.** Based on laboratory hydraulic conductivity test performed on a Shelby tube sample obtained from berm fill, the hydraulic conductivity of the granular fill material is 3.3×10^{-8} cm/sec (0.0104 m/a).

6) **All groundwater flow is vertical through the lower cohesive deposit (Layer 1) and horizontal through the basal sand (uppermost aquifer).** Freeze and Cherry (1979) state that in aquifer-aquitard systems with hydraulic conductivity contrast of two orders of magnitude or more, flow lines tend to become almost horizontal in the aquifers and almost vertical in the aquitards.

7) **The contaminant flux through the base of the CCR surface impoundments (i.e., lower cohesive deposit) is conservatively based on a fully saturated thickness of CCR.** The HELP model has been used to determine the vertical seepage rate and cover

performance over a 30-year period following the installation of a final cover system. The HELP (v3.07) model was developed by the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, for the U.S. Environmental Protection Agency (EPA), Risk Reduction Engineering Laboratory, Cincinnati, OH. HELP is the most widely used model to calculate the water balance of cover and bottom liner systems for landfills and other land disposal systems. HELP uses water balance computations to simulate the storage and vertical migration of leachate through a landfill design. Based on the HELP model results, where cover soils, a 200 mil geocomposite drainage layer and a 40 mil geomembrane are used as the final cover for the CCR surface impoundment, the liquid head is estimated to average approximately 297 inches (24.8 feet) in year 1, and by year 8 average 1.44 inches. The vertical seepage rates calculated by the HELP model were used to determine the vertical Darcy velocities specified in the MIGRATE model. The HELP model output file is included as Appendix A of this submittal.

8) **All angles are assumed to be 90°.** Right angle corners removes any extra thicknesses from the layers and shortens flow distances.

9) **The external stresses on the system are constant throughout time.** Minor variations in stress on the system over time cannot be accurately accounted, or estimated, for the entire model period. Therefore, potential short-term changes in heads due to construction, weather, and other flux changes are ignored.

4.2 Transport Processes

Using the site design and hydrogeologic characteristics and combining the analysis of groundwater flow information, the transport processes through each layer can be analyzed with respect to migration of CCR leachate constituents. The primary pathways to be addressed by this evaluation can be defined as vertical migration through the lower cohesive soils and horizontal (lateral) migration within the uppermost aquifer.

Vertical and lateral spreading of contaminant concentrations is denoted by the coefficients of hydrodynamic dispersion. The coefficients consists of two parts:

$$D_{xx} = D_{ex} + D_{mx}$$
$$D_{zz} = D_{ez} + D_{mz}$$

where D_{mx} and D_{mz} are the coefficients of advective (mechanical) dispersion in the longitudinal and vertical directions, and D_{ex} and D_{ez} are the coefficients of molecular diffusion in the longitudinal and vertical directions. The coefficients of advective dispersion D_{mx} and D_{mz} are defined as the product of the average linear velocity and the dispersivity ($D_m = \bar{v}\,\alpha$ [Bear, 1972]). As the velocity becomes smaller, the advective dispersion coefficient value approaches the value of the molecular diffusion coefficient. As this occurs, diffusion becomes the dominant transport mechanism.

The contaminant transport evaluation considers a conceptual scenario that simulates both vertical and horizontal contaminant transport via advective dispersion and molecular diffusion within both the lower cohesive deposit and the basal sand.

5. MATHEMATICAL MODEL

The two-dimensional advection/diffusion model chosen to represent contaminant transport at the site is MIGRATE (v9) by Rowe and Booker. This model provides for:

- advective as well as diffusive contaminant transport;
- two-dimensional transport in a horizontal and vertical orientation;
- multiple time and distance solutions to the transport equation;
- retardation of non-conservative constituents; and
- a transport solution with no space or time discretization errors (Rowe & Booker, 1988).

The principal assumptions inherent in MIGRATE are:

1) Fick's Law applies to solute diffusion within the deposit.

2) Sorption/desorption of a non-conservative species of contaminant is linearly controlled.

3) Contaminant migration in a given direction is governed by the two-dimensional contaminant transport equation presented in Rowe & Booker, 1985.

4) Multiple layers with different properties may be specified. It is assumed that there is continuity of concentration and flux at the boundary between two layers. The question of continuity is only a problem when there is insufficient diffusion to adequately “move” the contaminant across the layer boundary.

Within a given model scenario, MIGRATE has an advantage that permits a variety of top and bottom boundary conditions. For the purposes of this study, the following assumptions have been made:

1) “Top” boundary is a constant source boundary. Presuming a constant source over the entire life and post-closure of the facility for the closure-in-place alternative is a highly conservative assumption. This implies that full CCR leachate concentrations are present from day one, and no elutriation or removal of the contaminants is occurring. Studies (Farquhar, 1989) show that leachate quality improves dramatically over a relatively short time period. For the closure-by-removal alternative, the constant source is assumed to occur throughout the active life of the facility, with the source concentration being set to zero upon the completion of the CCR removal activities.

2) “Bottom” boundary is an “Aquifer” bottom boundary. This assumes that the bottom boundary is a permeable base stratum and that there is sufficient dispersion/mixing such that the concentration is uniform across the entire thickness of the base strata. The “Aquifer” bottom boundary option is applicable where the bottom layer (i.e., the lower cohesive deposit) is underlain by a permeable base stratum (i.e., the uppermost aquifer).

Using the conservative parameters listed in the conceptual model coupled with the cautious assumptions that MIGRATE offers, the model produces a conservative representation of

contaminant transport from the CCR surface impoundment for both the closure-in-place and closure-by-removal model scenarios.

5.1 Model Input and Sensitivity Analysis

Many input parameters were determined from site-specific data. Site-specific parameters include hydraulic conductivity, hydraulic gradient, unit thickness, CCR constituent concentrations and background groundwater concentrations. Where site-specific values were not available, literature values for comparable materials were selected.

5.1.1 Two-Dimensional Profile Setup

For the closure-in-place and closure-by-removal closure alternative contaminant transport evaluation, the conceptual model consists of the low permeability lower cohesive soil layer directly overlying the basal sand aquifer. The bottom of the aquifer is defined by the top of bedrock, the Pennsylvanian shale. Figure 3 depicts the cross-sectional geology and the conceptual hydrogeology of the conceptual model.

The thicknesses of the lower cohesive deposit and basal sand varies across the CWLP property. Along the down-gradient edge of the Dallman Ash Pond, the average thickness is approximately 19.4 feet (5.91 meters). The elevation of the bottom of the surface impoundment is 527 feet MSL. This places the lower cohesive deposit/basal sand interface at approximately 507.6 feet MSL. The basal sand deposit along the down-gradient edge of the Dallman Ash Pond has an average thickness of 7.3 feet (2.23 m). This places the basal sand/top of shale interface at an approximate elevation of 500 feet MSL.

Transport from the Dallman Ash Pond to the uppermost aquifer is depicted on the conceptual model drawing. Migration of the contaminants is restricted to vertical transport through the lower cohesive soils and horizontally through the basal sand, to the edge of Sugar Creek. The modeled scenario consists of 44 years of active life and 30 years of post-closure care following the placement of the final cover system.

The HELP determined seepage rates from the base of the Dallman Ash Pond were used to estimate the vertical Darcy velocities over the 30 year post-closure care period for the closure-in-place scenario.

The MIGRATE model input and output files are included in Appendix B**.**

5.1.2 Closure-In-Place Model Input Parameters

The final cover system was designed pursuant to Section 845.750. The components contained in the model are consistent with the design specified in the subject regulations.

5.1.2.1 Source Concentration

A surrogate concentration of 1 mg/L is assumed for the conceptual model scenario. Using a normalized concentration allows the use of the model predicted concentration as a multiplier of the current source concentrations.

5.1.2.2 Base Width and Surface Width

The base and surface widths were measured parallel to groundwater flow, along the eastern north-south edge of the Dallman Ash Pond. The base width measurement was taken from the toe at the up-gradient (south) side of the surface impoundment to the toe at the down-gradient (north) side of the CCR surface impoundment (see Figure 4). The surface width was measured from the 554 contour along the same path. This is the longest edge of the CCR surface impoundment. The modeled section, as illustrated on Figure 4, depicts the base width and surface width as 1,578 feet (481 m) and 1,663 feet (507 m).

5.1.2.3 Layer Thickness and Number of Sublayers

The thicknesses used for the conceptual model scenario are based upon an average of the thicknesses encountered along the down-gradient (northern) edge of the CCR surface impoundment.

Layer 1 - Lower Cohesive Deposit

As discussed in Section 3.0 above, the lower cohesive deposit ranges in thickness from 12.6 to 26.3 feet (3.84 to 8 m) along the down-gradient edge of the Dallman Ash Pond. The average thickness of the lower cohesive deposit is 19.4 feet (5.91 m).

MIGRATE permits a fixed number of sublayers to be entered or the model will determine the number (if the thickness is large and NSUB = 1). A sublayer (NSUB) value of 10 was chosen for the lower cohesive deposit. This provides predicted contaminant concentrations at approximately 2 foot intervals to the top of the basal sand (uppermost aquifer).

Aquifer - Basal Sand

The basal sand deposit along the down-gradient edge of the Dallman Ash Pond ranges in thickness from 3 to 7.25 feet (0.914 to 3.51 m). The average thickness of the basal sand is 7.25 feet (2.23 m).

The aquifer bottom boundary condition represents the case where the deposit is underlain by permeable base strata. For the aquifer bottom boundary condition, the underlying permeable base strata cannot be assigned a sublayer value. This boundary condition assumes that there is sufficient dispersion/mixing such that the concentration is uniform across the entire thickness of the aquifer.

5.1.2.4 Dry Density and Distribution Coefficient

MIGRATE uses the product of the density and the distribution (sorption) coefficient to simulate the adsorption of a non-conservative species. Adsorption is the attraction and adhesion of a layer of ions from an aqueous solution to the solid mineral surfaces with which it is in contact. The consequent adsorption results in the diminished concentration of that non-conservative species in groundwater. This contaminant transport evaluation assumes that there is no adsorption for all the species. Therefore, the adsorption is set to zero (0) in the contaminant transport model to ignore the adsorption of non-conservative species. This is a conservative approach that results in higher predicted concentrations for non-conservative species.

5.1.2.5 Effective Porosity

The effective porosity, as defined by Fetter (1980) is “the amount of interconnected pore space through which fluids can pass, expressed as a percent of bulk volume.” The effective porosity

parameter is used to calculate the seepage velocity of a geologic unit. The specific yield or the "drainable water" approximates the effective porosity (Bear, 1972). The saturation water content of a soil, which is the total water drained by gravity from a saturated soil, provides a reasonable approximation of the effective porosity of a soil.

Layer 1 - Lower Cohesive Deposit

The effective porosity of the lower cohesive deposit is based on the saturation water content data of silt, which reportedly ranges from 35% to 57% (Sharp-Hansen et al., 1990). A mean effective porosity value of 46% was selected for the model.

Aquifer - Basal Sand

For the basal sand, the effective porosity of the lower cohesive deposit is based on the saturation water content data of sand. The range provided is from 37% to 49% (Sharp-Hansen et al., 1990). A mean effective porosity value of 43% was selected for the baseline model.

5.1.2.6 Darcy Velocity

The Darcy velocity for the conceptual model scenario was derived from the vertical percolation rates reported in the HELP model results.

Layer 1 - Lower Cohesive Deposit

Based on the HELP model results for vertical percolation (q_{HELP}), the vertical Darcy velocity (v_a) varies from an initial rate of 0.651 m/a and decreasing to a value of 0.00551 m/a by post-closure year 30 (model years 51 to 80). Darcy velocity rates were grouped to provide representative vertical velocities.

Layer 1 – Vertical Darcy Velocity			
Model Period	Start Time – End Time	HELP Annual Vertical Percolcation through Layer 1, q_{HELP} (ft^3/a/acre)	Layer 1 Vertical Darcy Velocity, v_a (m/a)
1	0-50	93018	0.651
2	50-51	84545	0.592
3	51-52	76831	0.538
4	52-53	69993	0.490
5	53-54	63413	0.444
6	54-55	57616	0.403
7	55-56	45283	0.317
8	56-57	24749	0.173
9	57-58	7885	0.0552
10	58-60	5535	0.0387
11	60-62	3462	0.0242
12	62-67	2417	0.0169
13	67-72	1397	0.00979
14	72-77	874	0.00614
15	77-80	787	0.00551

Aquifer - Basal Sand

The Darcy velocity for the basal sand was determined by calculating the total horizontal flow into the up-gradient side of the uppermost aquifer combined with the vertical seepage from the overlying CCR surface impoundment. As discussed below, the total flow was used to calculate the Darcy velocity at the down-gradient edge of the CCR surface impoundment.

The horizontal Darcy velocity at the down-gradient edge of the CCR surface impoundment is termed the "Base Outflow Velocity." The "Base Outflow Velocity" is the Darcy velocity beneath the down-gradient edge of the CCR surface impoundment corresponding to the horizontal inflow at the up-gradient edge (q_{in}) plus the vertical inflow (q_a) from the overlying CCR surface impoundment.

Based on continuity of flow the initial flow in the basal sand, (q_{in}) is given by the inflow velocity (v_{in} =Ki = 3,942 m/a ·0.0119 = 46.9 m/a) multiplied by the thickness of the basal sand (H = 2.23 m) and the width of the CCR surface impoundment perpendicular to the direction of groundwater flow (W = 310 m), thus:

$$q_{in} = v_{in} \cdot H \cdot W = 46.9 \frac{m}{a} \cdot 2.23\ m \cdot 310\ m = 32{,}429\ m^3/a$$

The flow into the aquifer from the CCR surface impoundment, q_a, is the downward Darcy velocity from the overlying lower cohesive deposit (v_a = 0.651 m/a) multiplied by the length (L = 481 m) and the width (W = 310 *m*) of the CCR surface impoundment, thus:

$$q_a = v_a \cdot L \cdot W = 0.651 \frac{m}{a} \cdot 481\ m \cdot 310\ m = 97{,}051\ m^3/a$$

The outflow at the down-gradient edge of the CCR surface impoundment is:

$$q_{out} = q_{in} + q_a = 32{,}429 \frac{m^3}{a} + 97{,}051 \frac{m^3}{a} = 129{,}480\ m^3/a$$

The "Base Outflow Velocity", v_b, is the outflow divided by the width of the CCR surface impoundment (W = 310 m) and the thickness of the aquifer (H = 2.23 m), therefore:

$$v_b = \frac{q_{out}}{(W \cdot H)} = \frac{129{,}480 \frac{m^3}{a}}{310\ m\ \cdot 2.23\ m} = 187\ m/a$$

The baseline vertical Darcy velocity varies from the initial flow velocities as a result of the installation of the proposed final cover. The varying flow velocities are presented in the table below.

Aquifer – Base Velocity							
Model Period	Start Time – End Time	Layer 1 Vertical Darcy Velocity, v_a (m/a)	Layer 1 Vertical Flow In, q_a (m^3/a)	Aquifer Horizontal Darcy Velocity, v_{in} (m/a)	Aquifer Horizontal Flow In, q_{in} (m^3/a)	Aquifer Horizontal Flow Out, q_{out} (m^3/a)	Aquifer Horizontal Darcy Velocity Out, v_b (m/a)
1	0-50	0.651	97051	46.9	32429	129480	187
2	50-51	0.592	88211			120640	175
3	51-52	0.538	80161			112590	163
4	52-53	0.490	73028			105457	153
5	53-54	0.444	66163			98591	143
6	54-55	0.403	60114			92543	134
7	55-56	0.317	47246			79675	115
8	56-57	0.173	25820			58249	84
9	57-58	0.0552	8227			40656	59
10	58-60	0.0387	5776			38205	55
11	60-62	0.0242	3612			36041	52
12	62-67	0.0169	2521			34950	51
13	67-72	0.00979	1460			33888	49
14	72-77	0.00614	915			33344	48
15	77-80	0.00551	821			33250	48

5.1.2.7 Coefficients of Hydrodynamic Dispersion

As characterized in Section 4.2 above, the vertical and lateral spreading of contaminant concentrations is denoted by the coefficients of hydrodynamic dispersion, the coefficient of advective (mechanical) dispersion, D_m, and the coefficient of molecular diffusion, D_e.

Layer 1 - Lower Cohesive Deposit

Diffusion, D_e, of chemical constituents is an important contributor to dispersion where velocities are low. Freeze and Cherry (1979) characterized the one-dimensional diffusion of a non-reactive species in fine-grained geologic media. This study indicated diffusion coefficients range from 1 x 10^{-10} to 1 x 10^{-11} m^2/s. An average of 5 x 10^{-10} m^2/s or 0.0158 m^2/a was chosen for the lower cohesive deposit.

Following Illinois EPA guidance (LPC-PA2, Appendix C), acceptable sources for evaluating dispersivity, α, include Gelhar, et al (1992) for all distances, Xu and Eckstein (1995) for distances greater than 100 m, and Schulze-Makuch (2005) for distances less than 100 m. For the purposes of the Dallman Ash Pond contaminant transport model, the scale or length of flow path used to determine the vertical dispersivity for the lower cohesive deposit was assumed to be 19.4 feet (5.91 m).

Given a flow distance of 5.91 m, the following relationship between dispersivity and flow distance given by Schulze-Makuch (2005) is appropriate:

$$a_L = c(L)^m$$

where:

α = mechanical dispersivity

L = study scale (m)

c = a parameter characteristic for a geologic medium

m = scaling exponent related to the geologic medium

Schulze-Makuch (2005) developed a power law relationship relating the geologic media and study scale to dispersivity using dispersivity data compiled from 109 different authors for different types of geological media. For unconsolidated sediments, the data was divided into three reliability classes, I being high reliability, II being Intermediate reliability and III being low reliability. Over the entire range of low to high reliability data, the scaling exponent m was found to vary from 0.44 to 0.94 with the higher reliability subset of data (I) at the lower end of the observed range. For purposes of this contaminant transport model, the Schulze-Makuch (2005) high reliability values for unconsolidated deposits are c = 0.2 and m = 0.44.

Using the Schulze-Makuch (2005) power law relationship and a lower cohesive deposit flow length (vertical) of 19.4 feet (5.91 m), the longitudinal (i.e., parallel to groundwater flow) dispersivity is estimated to be:

$$a_x = c(L)^m = 0.2(5.91\ m)^{0.44} = 0.437\ m$$

In accordance with Illinois EPA, the transverse dispersivity (i.e., perpendicular to groundwater flow) was calculated by multiplying the longitudinal value by 10 percent.

$$a_z = 0.437 \cdot 0.1 = 0.0437\ m$$

The coefficient of advective dispersion for the lower cohesive deposit, D_{mx}, is defined as the product of the average linear velocity and the dispersivity (Bear, 1972).

$$D_{mx} = \frac{a_x \cdot v}{n_e} = \frac{0.437 \cdot 0.651}{0.46} = 0.618\ m^2/a$$

The horizontal coefficient of advective dispersion is:

$$D_{mz} = \frac{a_z \cdot v}{n_e} = \frac{0.0437 \cdot 0.651}{0.46} = 0.0618\ m^2/a$$

As discussed above, the hydrodynamic dispersion (D) is a result of the mechanical mixing and the molecular diffusion. The coefficient of hydrodynamic dispersion can be expressed as follows:

$$D = D_m + D_e$$

Within the lower cohesive deposit, both diffusion and dispersion are important transport processes. The coefficients of hydrodynamic dispersion, D_v and D_h, are estimated as:

$$D_v = D_{mx} + D_e = 0.618\, m^2/a + 0.0158\, m^2/a = 0.634\, m^2/a$$

$$D_h = D_{mz} + D_e = 0.0618\, m^2/a + 0.0158\, m^2/a = 0.0776\, m^2/a$$

However, as the vertical velocity changes with time, due to the decrease in infiltration due to the proposed final cover, the coefficient of hydrodynamic dispersion will vary as provided in the table below.

Layer 1 - Coefficient of Hydrodynamic Dispersion						
Model Period	Start Time – End Time	Longitudinal Mechanical Dispersion, D_{mx} (m^2/a)	Transverse Mechancial Dispersion, D_{mz} (m^2/a)	Molecular Diffusion, D_e (m^2/a)	Vertical Coefficient of Hydrodynamic Dispersion, D_v (m^2/a)	Horizontal Coefficient of Hydrodynamic Dispersion, D_h (m^2/a)
1	0-50	0.618	0.0618	0.0158	0.634	0.0776
2	50-51	0.562	0.0562		0.578	0.0720
3	51-52	0.511	0.0511		0.527	0.0669
4	52-53	0.465	0.0465		0.481	0.0623
5	53-54	0.422	0.0422		0.437	0.0580
6	54-55	0.383	0.0383		0.399	0.0541
7	55-56	0.301	0.0301		0.317	0.0459
8	56-57	0.165	0.0165		0.180	0.0323
9	57-58	0.0524	0.00524		0.0682	0.0210
10	58-60	0.0368	0.00368		0.0526	0.0195
11	60-62	0.0230	0.00230		0.0388	0.0181
12	62-67	0.0161	0.00161		0.0319	0.0174
13	67-72	0.00930	0.00093		0.0251	0.0167
14	72-77	0.00583	0.000583		0.0216	0.0164
15	77-80	0.00523	0.000523		0.0210	0.0163

Basal Sand – Uppermost Aquifer

Based on Freeze and Cherry (1997) diffusion coefficients , D_e, of a non-reactive species for coarse-grained unconsolidated materials can be somewhat higher than 1 x 10^{-10} m^2/sec but are less than the coefficients for chemical species in water (2 x 10^{-9} m^2/s). A value of 1 x 10^{-9} m^2/s or 0.0315 m^2/a was chosen to represent the basal sand diffusion.

In the basal sand the flow length is based upon the distance from the center of the CCR surface impoundment to the down-gradient edge of the CCR surface impoundment, 789 feet (240.5 m). When using the aquifer bottom boundary condition, the concentration is assumed to be the same vertically across the entire layer. As such, there is no input for the transverse dispersivity. Given

a flow distance of 240.5 m, the following relationship between dispersivity and flow distance given by Xu and Eckstein (1995) is appropriate and the longitudinal dispersivity is estimated to be:

$$a_x = 0.83[log_{10}(240.5)]^{2.414} = 6.74\ m$$

The coefficient of advective dispersion for the basal sand, D_{mx}, is:

$$D_{mx} = \frac{a_x \cdot v}{n_e} = \frac{6.74 \cdot 187}{0.43} = 2{,}936\ m^2/a$$

The coefficient of hydrodynamic dispersion for the basal sand is estimated as:

$$D = D_{mx} + D_e = 2{,}936\ m^2/a + 0.0315\ m^2/a = 2{,}936\ m^2/a$$

As with the lower cohesive deposit, the base flow velocity changes with time, due to the decrease in infiltration as a result of the proposed final cover, the coefficient of hydrodynamic dispersion will vary as provided in the table below.

Aquifer - Coefficient of Hydrodynamic Dispersion					
Model Period	Start Time – End Time	Base Flow Velocity, v_b (m/a)	Longitudinal Mechanical Dispersivity, D_{mx} (m^2/a)	Molecular Diffusion, D_e (m^2/a)	Coefficient of Hydrodynamic Dispersivity, D_h (m^2/a)
1	0-50	187	2936	0.0315	2936
2	50-51	175	2735		2735
3	51-52	163	2553		2553
4	52-53	153	2391		2391
5	53-54	143	2235		2235
6	54-55	134	2098		2098
7	55-56	115	1807		1807
8	56-57	84	1321		1321
9	57-58	59	922		922
10	58-60	55	866		866
11	60-62	52	817		817
12	62-67	51	792		792
13	67-72	49	768		768
14	72-77	48	756		756
15	77-80	48	754		754

5.1.2.8 Lateral Distances

Three lateral distances were specified for the determination of solute concentrations. The distances selected are: 1) at the inside toe of slope of the CCR surface impoundment berm of the Dallman Ash Pond; 2) at 33 meters, half of the distance from the inside toe of the slope of the CCR surface impoundment to the closest edge of Sugar Creek; and 3) at the edge of Sugar

Creek, 66 meters from the inside toe of the slope of the CCR surface impoundment. These distances in the MIGRATE model results are expressed as 240.5 m, 273.5 m and 306.5 m. These distances are from the center of the CCR surface impoundment.

5.1.2.9 Integration

To find the solution to the advective-dispersion equation MIGRATE uses a Laplace transform. The numerical inversion of the Laplace transform depends on the Talbot parameters. Default values for the for the Laplace and Gauss integration parameters can be used, or they can be specified by the user. Acceptable results will generally be produced by the default values of these parameters. Occasionally, a solution may require more than the default integration parameters if negative values are produced or if erroneous concentrations appear at the surface outside of the source area. The model authors, Rowe and Booker, acknowledge that these negative values may be merely a poor numerical approximation of zero concentration or flux and can be eliminated by increasing the integration parameters. However, using very high integration parameters usually results in an increase the computational time without significantly improving the solution.

For the model used in this evaluation it was found that the model results were unstable using the normal integration parameters, yielding negative predicted concentration values. Using the selected option and specifying an increase in Gaussian integration resulted in fewer non-negative predicted concentrations.

5.1.3 Closure-By-Removal Model Input Parameters

The baseline contaminant transport model prepared to evaluate the closure-in-place alternative, was modified to evaluate the closure-by-removal alternative. Under the closure-by-removal scenario, it is assumed that the vertical diffusion, horizontal diffusion, vertical velocity, aquifer outflow velocity and aquifer dispersion remains constant throughout the entire model life. For the closure-by-removal alternative, the model geometry, and hydrogeologic characteristics of the uppermost aquifer do not change. However, the source concentration will vary. It was assumed that the source concentration will remain constant (1 mg/L) until the completion of CCR removal, represented by model year 54 (2029), at which time the source concentration will be 0 mg/L, beginning model year 55 (2030) to the end of the after closure-by-removal period represented by model year 84 (2059).

The closure-by-removal contaminant transport model was organized into two model periods. Model period 1 represents the active life of the surface impoundment from year 1 to year 54 (1976 to 2029). Model period 2 represents year 54 through year 84 (2029 through 2059), the after closure-by-removal period.

Closure-By-Removal Input Parameters							
Model Period	Start Time – End Time	Layer 1 - Vertical Coefficient of Hydrodynamic Dispersion, D_v (m^2/a)	Layer 1 - Horizontal Coefficient of Hydrodynamic Dispersion, D_h (m^2/a)	Layer 1- Vertical Darcy Velocity, v_a (m/a)	Aquifer Horizontal Darcy Velocity Out, v_b (m/a)	Coefficient of Hydrodynamic Dispersivity, D_h (m^2/a)	Normalized Initial Concentration (mg/L)
1	0-54	0.634	0.0776	0.651	187	2936	1
2	54-84						0

The lateral distances and integration for the closure-by-removal alternative were consistent with those used for the closure-in-place alternative. This allows the direct comparison of the result for the closure-by-removal alternative.

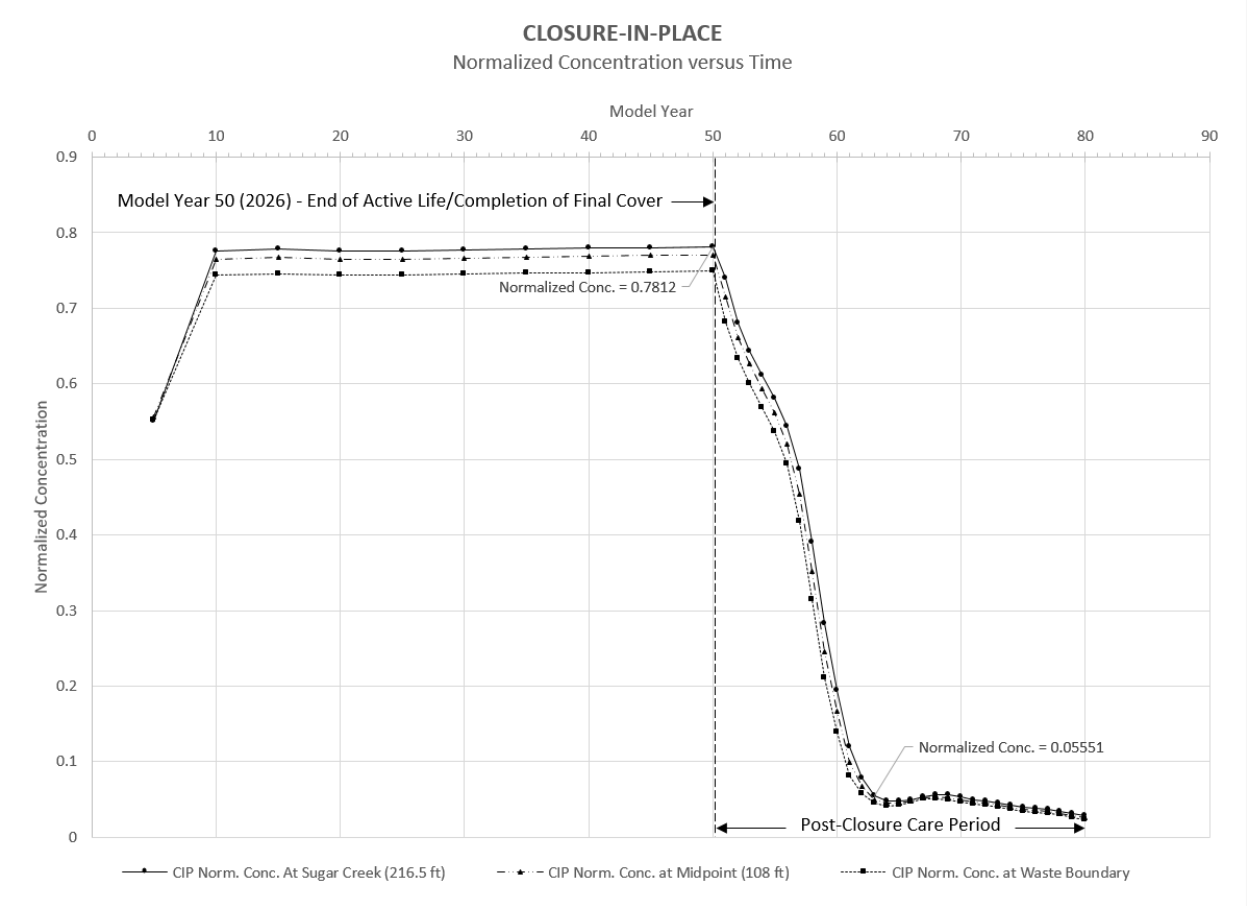
6. CONTAMINANT TRANSPORT RESULTS

The MIGRATE simulations for both the closure-in-place and the closure-by-removal alternatives were completed for the active period and the post-closure care/ after closure-by-removal period for the Dallman Ash Pond CCR surface impoundment for three locations - at the edge of the CCR surface impoundment, at one-half the distance to the nearest section of Sugar Creek, and at the edge of Sugar Creek. For the active period, concentrations were generated at five year intervals for the closure-in-place scenario and at six year intervals for the closure-by-removal scenario. For the subsequent post-closure care and after closure-by-removal periods, concentrations were generated on a yearly interval for both closure alternatives modeled.

For both model scenarios, the predicted concentration reached steady state conditions, approximately 0.78 mg/L, during the second model period (model year ten for the closure-in-place scenario and model year twelve for the closure-by-removal scenario). Subsequent predicted concentrations during the active life for both model scenarios did not continue to increase. This stabilization of the concentration within the uppermost aquifer was expected due to the flushing velocity of the uppermost aquifer. Based upon this observation, it can be reasoned that the length of the active life, at least for times beyond ten years, does not have an impact on the maximum concentration within the uppermost aquifer.

6.1 Closure-In-Place

The concentrations generated by MIGRATE are normalized. The peak normalized concentration during the first 50 years of active-life is approximately 0.7812 mg/L. Upon installation of the final cover system (model year 50 or calendar year 2026) the predicted normalized concentrations exhibit a decrease. The normalized concentrations continue to decrease to approximately one-tenth (0.0551 mg/L) of the initial concentration by model year 63 (2039), approximately 13 years after the completion of final cover construction activities.

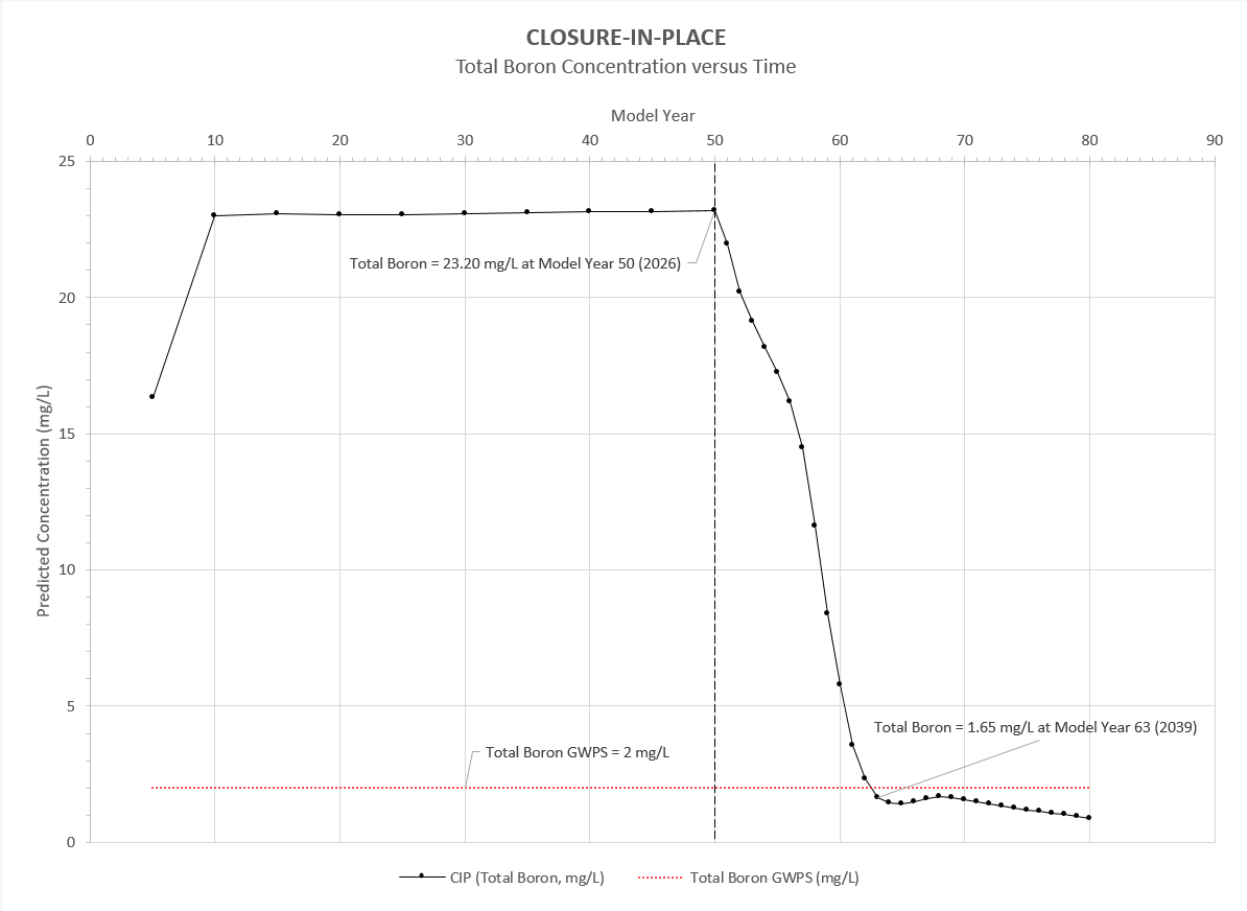


The normalized concentration (C_{norm}) at the end of the active period (0.7812 mg/L) is used with the groundwater concentration (C_{gw}) measured at AP-14 for the CCR parameter total boron (23.2 mg/L as determined for the second quarter 2021 sampling event to determine a prediction factor (F_p).

$$F_p = \frac{C_{norm}}{C_{gw}} = \frac{0.7812\,\frac{mg}{L}}{23.2\,\frac{mg}{L}} = 0.0337\ mg/L$$

Using the prediction factor, F_p, it can be seen that total boron concentration at the edge of Sugar Creek (C_{norm} = 0.05551 mg/L) is predicted to be at or below the GWPS (2 mg/L) by model year 63 (2038), approximately 13 years following the installation of the final cover system.

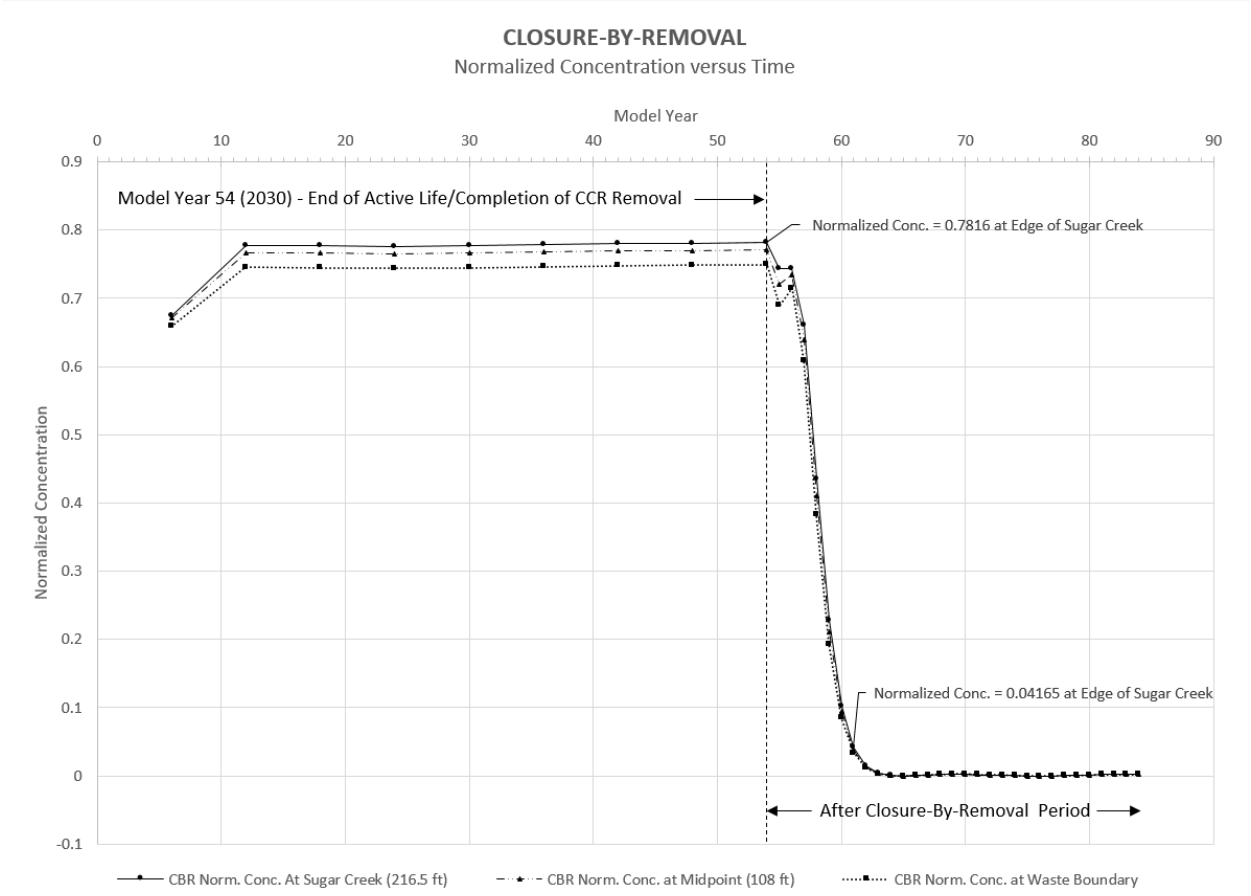
$$C_{gw} = \frac{C_{norm}}{F_p} = \frac{0.05551\,\frac{mg}{L}}{0.0337\,\frac{mg}{L}} = 1.65\ mg/L$$



Assuming that model year 50 (2026) represents the completion of the installation of a final cover system for the closure-in-place scenario, the contaminant transport model predicts compliance with the GWPS by model year 63 (2039).

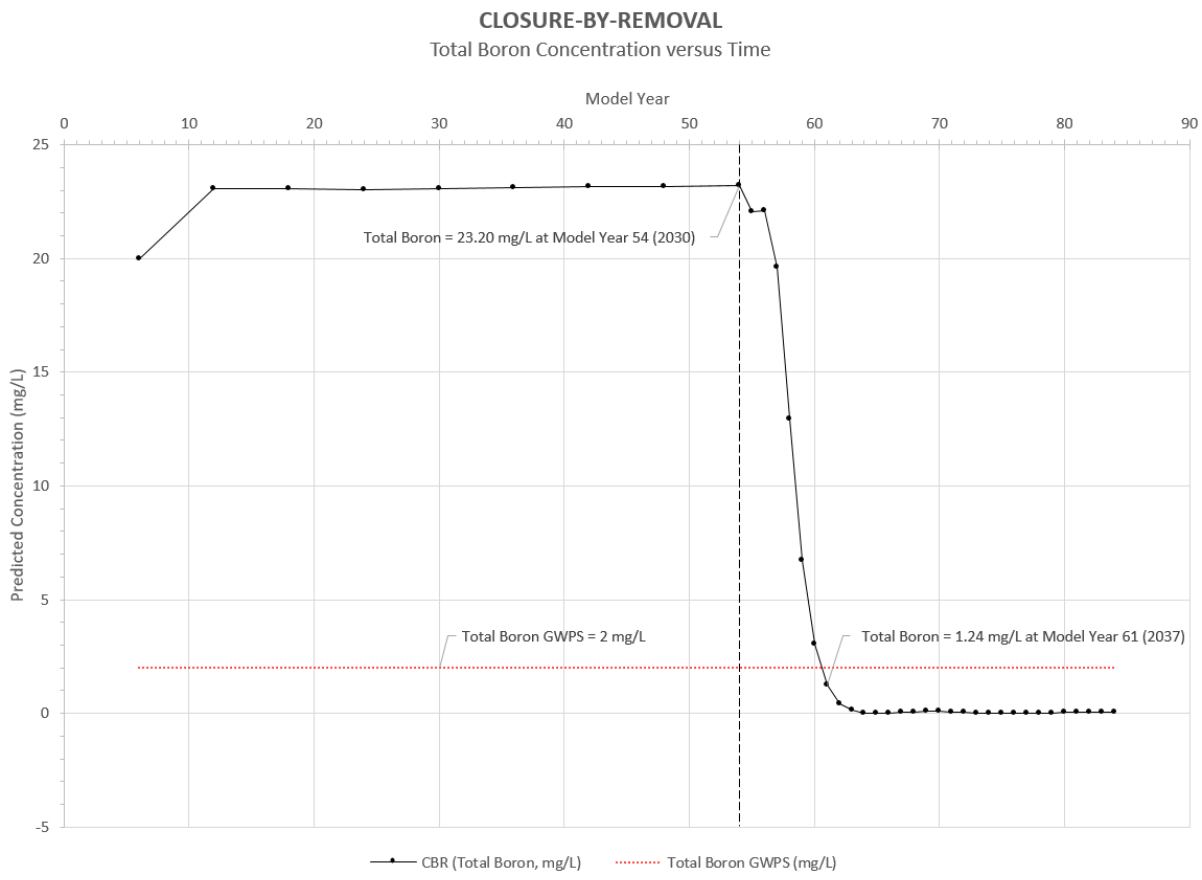
6.2 Closure-by-Removal

As with the closure-in-place contaminant transport model results, the concentrations generated by MIGRATE for the closure-by-removal evaluation are normalized. The normalized concentration at the end of the first 54 years of active-life is approximately 0.7816 mg/L. As expected, this concentration is in agreement with the active life results for the closure-in-place contaminant transport modeling effort. The predicted normalized concentrations exhibit a decrease following the removal of the source concentration and by model year 61 are approximately one-tenth of the initial concentration (0.04165 mg/L).



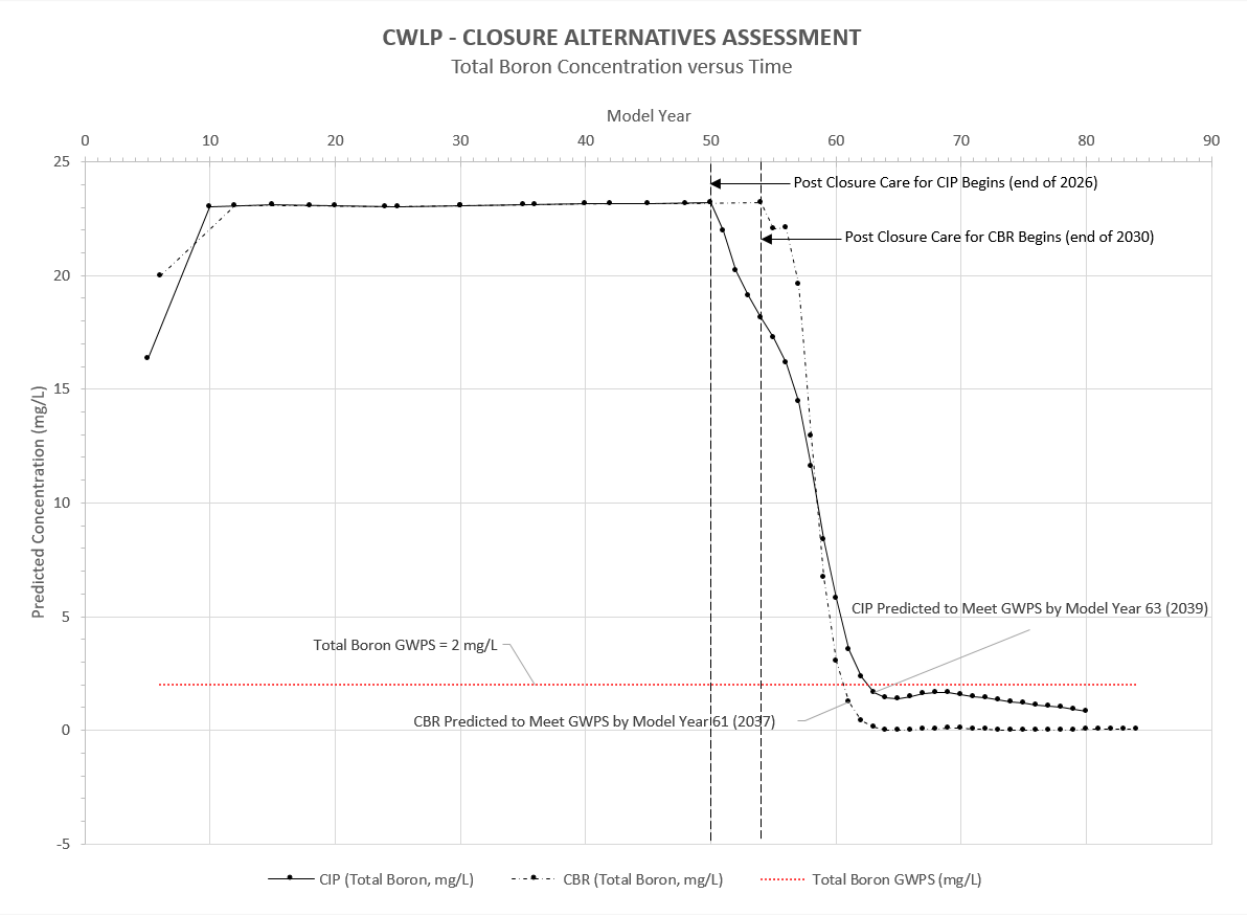
As presented above, the normalized concentration (C_{norm}) at the end of the active period (0.7816 mg/L) is used with the maximum concentration (C_{gw}) for the CCR parameter total boron (23.2 mg/L), to determine a prediction factor (F_p). As expected, F_p for this scenario is also equal to 0.0337 for this scenario.

Using the prediction factor, F_p, a graph was prepared to represent total boron concentrations following the closure-by-removal activities. As shown in the graph, concentrations following the completion of the CCR removal display a decrease. Based upon the closure-by-removal evaluation total boron concentrations are predicted to be at or below the groundwater protection standard, 2 mg/L, by model year 61 (2037).


CLOSURE-BY-REMOVAL
Total Boron Concentration versus Time
Model Year
0
10
20
30
40
50
60
70
80
90
25
20
15
10
5
0
-5
Predicted Concentration (mg/L)
Total Boron = 23.20 mg/L at Model Year 54 (2030)
Total Boron GWPS = 2 mg/L
Total Boron = 1.24 mg/L at Model Year 61 (2037)
CBR (Total Boron, mg/L)
Total Boron GWPS (mg/L)

7. CCR SURFACE IMPOUNDMENT CONTAMINANT TRANSPORT EVALUATION RESULTS

The MIGRATE model predicts compliance with the groundwater protection standard within approximately 13 years following installation of a final cover system on the Dallman Ash Pond, or 4 years following the completion of the removal of CCR from the Dallman Ash Pond. Throughout the remainder of the 30 year post-closure care/after closure-by-removal period the predicted concentrations remain below the groundwater protection standard for both alternatives evaluated. The MIGRATE input and output files are provided in Appendix B of this evaluation document. The files titled BASE005.I/O are for the closure-in-place scenario and BASE014.I/O are for the closure-by-removal scenario. The groundwater protection standards were determined pursuant to the 35 Ill. Adm. Code 845 CCR final rules.



8. CONCLUSIONS

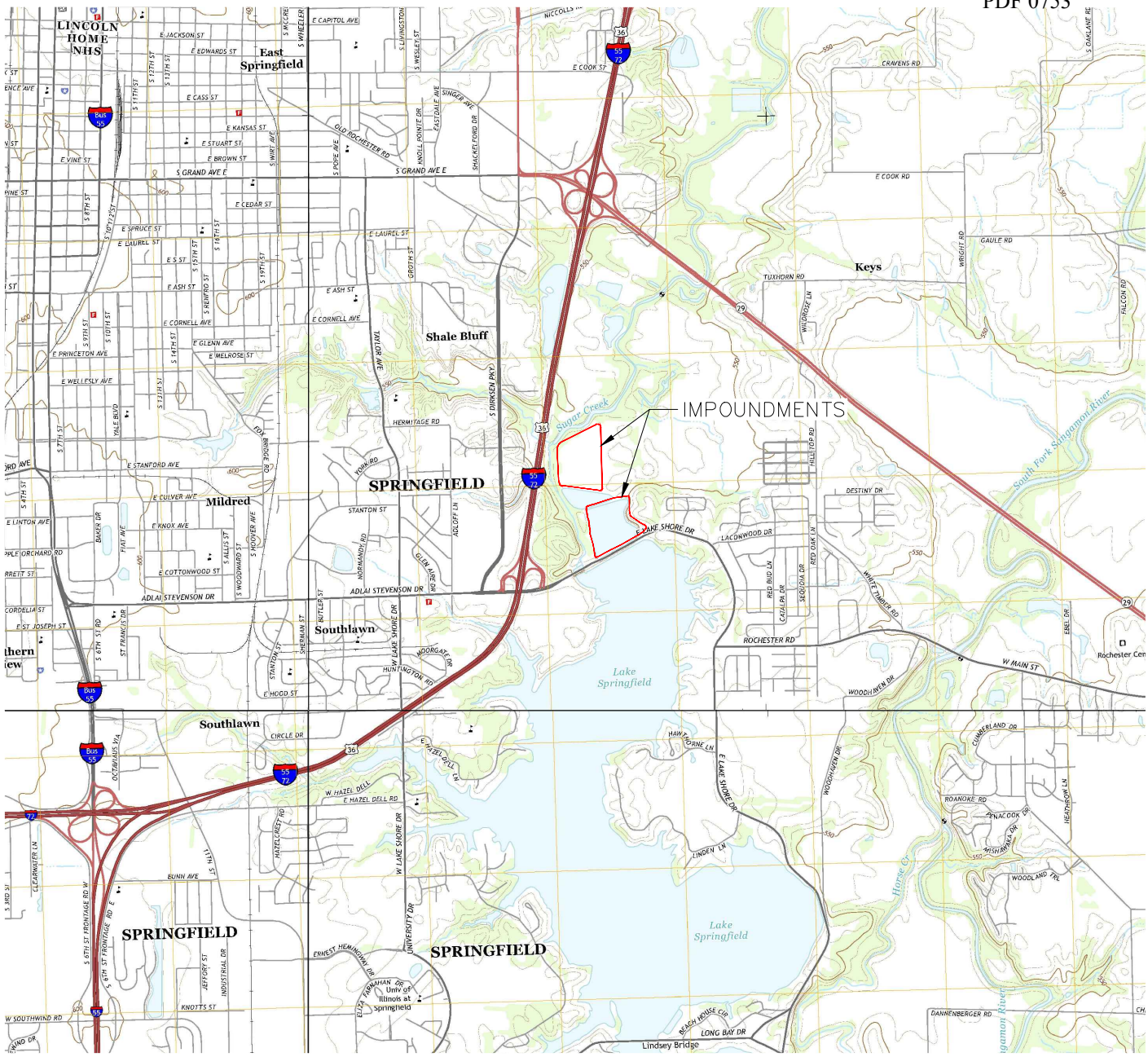
The two-dimensional contaminant transport model MIGRATE has been used to evaluate the effect on groundwater quality as a result of the installation of a final cover system for the Dallman Ash Pond CCR surface impoundment, the “closure-in-place” alternative. Additionally, the contaminant transport modeling effort includes the evaluation of the removal of the CCR material from the CCR surface impoundments, the “closure-by-removal” alternative. The input parameters Darcy velocity, coefficient of diffusion, dispersivity, coefficient of hydrodynamic dispersion, liner leakage and layer thickness were selected to conservatively and most accurately represent the site-specific design and hydrogeologic conditions for both of these closure alternatives. Therefore, this groundwater contaminant transport evaluation is a conservative representation of the expected impact of the CCR surface impoundment on the groundwater at the site.

The results of the contaminant transport model simulation for both the closure-in-place alternative and the closure-by-removal demonstrate that the GWPS may be obtained through either minimizing infiltration through CCR material by the placement of a final cover system or through the removal of the CCR material. Both scenarios use total boron as the surrogate leachate parameter.

For the closure-in-place scenario, the GWPS was obtained for total boron within 13 years (2039) following implementation of the final cover system. With the installation of a final cover system, the model demonstrates that leaching of COCs from the CCR material will be reduced or eliminated and that COC concentrations in groundwater will decrease over time.

For the closure-by-removal scenario, the GWPS was obtained for total boron within approximately 7 years (2037) following completion of the CCR removal activities. The removal of the CCR material will result in a reduction and eventual elimination of the leaching of COCs to groundwater. COC concentrations in groundwater will decrease over time.

FIGURES



NOTE:
BACKGROUND IMAGE COURTESY OF
UNITED STATES GEOLOGICAL SURVEY.

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APPROVED BY: BJH	DESIGNED BY: BJH	DRAWN BY: MPN

SITE LOCATION

PLANS PREPARED FOR
CITY, WATER, LIGHT & POWER
SPRINGFIELD, SANGAMON COUNTY, ILLINOIS

DATE: JULY 2021
PROJECT ID: 200387/0026
SHEET NUMBER:
FIGURE 1

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UNIT 2
(UNDEVELOPED)

UNIT 1

UNIT 2 LANDFILL

UNIT 1 LANDFILL (CLOSED)

LAKE SPRINGFIELD

LIME SOFTENING POND

EAST LAKE SHORE DRIVE

LAKESIDE ASH POND

LIME SOFTENING POND

LIME SOFTENING POND

DALLMAN ASH POND

CLARIFICATION POND

SUGAR CREEK

SUGAR CREEK

I-55

I-55

1545 ft

AP-1
AP-2
AP-3
AP-4
AP-5
AP-6
AP-7
AP-8
AP-9
AP-10
AP-11
AP-12
AP-13
AP-14
RW-3

LEGEND

- CCR SURFACE IMPOUNDMENT
- EXISTING MONITORING WELL
- TEMPORARY ASSESSMENT WELL

N

0 150 300 600

SCALE: IN FEET

NOTE

IMAGE SOURCE: GOOGLE EARTH PRO, IMAGE DATE JUNE 7, 2016.

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DESIGNED BY: DG | DRAWN BY: MPN | APPROVED BY: DG

GROUNDWATER MONITORING NETWORK

PLANS PREPARED FOR
CITY, WATER, LIGHT AND POWER
SPRINGFIELD, SANGAMON COUNTY, ILLINOIS

DATE: JULY 2021

PROJECT ID: 200387/0026

SHEET NUMBER:

FIG. 2

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Tab: Layout1 Last Saved: July 28, 2021, by Mike Nguyen Plotted: Wednesday, July 28, 2021 11:12:08 AM
J:\S\Springfield CWLP\dwg\2021\Groundwater Monitoring Network.dwg

NO.	DATE	BY	REVISION
1	12-7-76	TKP	(M.T.) Added 6" topsoil to dike. (L, B) Removed Bench Mark and note 8.
	12-10-76	LNB	ISSUED
2	4-27-77	MGH	Changed Dike Access Rd. and exist. dike alignments. Removed 300' pipe. Moved exist. pipe
	4-27-77	LNB	ISSUED
3	5-26-77	DEV	Revised dike alignment along So. side of Ash Pond.
	5-31-77	LNB	ISSUED
4	2-1-79	DLM	CONFORMING TO CONSTRUCTION RECORDS

REVISION DESCRIPTION | DATE | NO. | BY

CONTRACT NO.: 3333
ASH POND
V.Y. DALLMAN POWER STATION - UNIT 33
SPRINGFIELD, ILLINOIS
GRADING PLAN

Burns & McDonnell
Engineers - Architects - Consultants
KANSAS CITY, MISSOURI

DATE AUG. 13, 1976	DRAWING NO. Y29 - 4 REV.
DESIGNED LNB	
DETAILED TRP	PROJECT 73-008-1
CHECKED	SHEET OF SHEETS

Notes:

1. Two 36" Corrugated Metal Pipe, 16 gauge, Bituminous coated, 2 2/3" x 1/2" corrugations, with 2 end sections required.
2. Excavate for Relocated Sugar Creek from Sta. 2+00 to Sta 19+00 leaving excavation isolated from Sugar Creek waters until excavation and Rip Rap is completed between the above mentioned stations.
3. Area is susceptable to flooding.
4. Over excavate at least 4.0 feet under dike below exist. channel banks and bottom. (See cross sections) Fill with cohesive material and compact to at least 90%
5. Form a greenbelt on both sides of Relocated Sugar Creek. Areas within Greenbelt which are cleared or disturbed during construction, shall be topsoiled & seeded. Plant trees on 25ft. On ground above el. 540.0 10 trees shall be Hard Maple the rest shall be Sycamore. On ground that is below El. 540.0 10 trees shall be Pin Oak the rest shall be Sycamore. Do not plant trees on ash pond dike
6. Clearing Limits as shown outside of Purchasers property lines. Within Purchasers property clear as necessary for const. keep clearing limits as close to work as possible
7. Waste area is north of Green Belt. Grade waste area to drain and to present a neat appearance. Seed and mulch Waste Area.
8. Borrow material for the Ash Pond Dikes shall come from center of pond and Relocated Sugar Creek. If in the opinion of the engineer material taken from the pond area is too wet to dry in a reasonable length of time the contractor will be permitted to borrow from the area directly north of Relocated Sugar Creek as long as the borrow area is topsoiled, seeded, and mulched, and graded to drain as directed by engineer.

0 120 240 480
SCALE: IN FEET

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DESIGNED BY: MTH | DRAWN BY: MPN | CHECKED BY: BJH | APPROVED BY: BJH

1976 SITE SURVEY

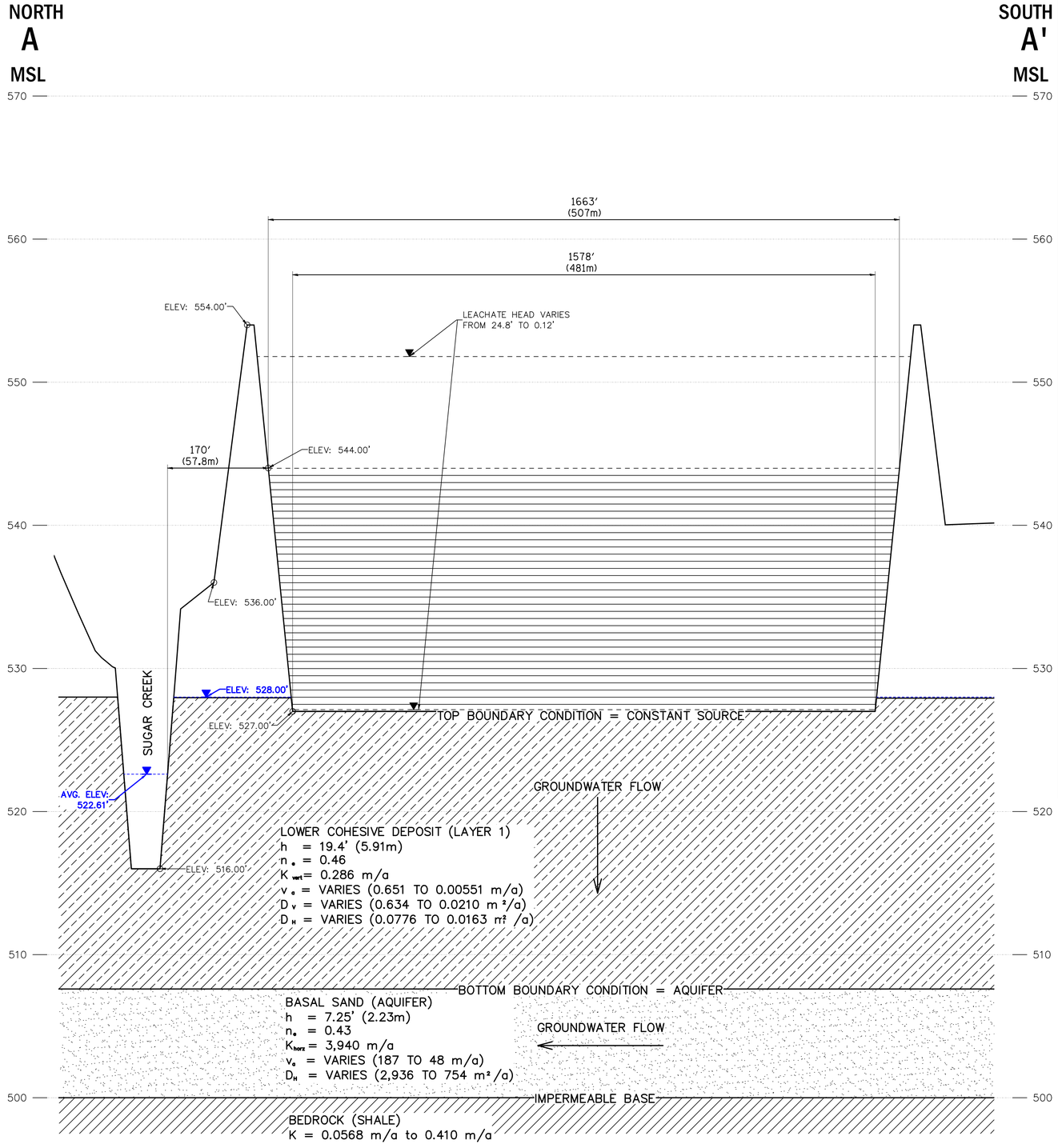
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DATE: AUGUST 2021
PROJECT ID: 200325/0003
SHEET NUMBER:

FIG. 3

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Tab: Layout1 Last Saved: August 23, 2021, by Mike Nguyen Plotted: Monday, August 23, 2021 10:07:43 AM
J:\S\Springfield CWLP\CWLP Ash Pond\DWG\2021\1974 Site Survey.dwg





SITE MAP
SCALE: AS SHOWN

USGS 05576250 SUGAR CREEK NEAR SPRINGFIELD, IL
Latitude 39°46'45", Longitude 89°35'34" NAD83

YEAR	00065, Gage height, feet, Monthly mean in ft (Calculation Period: 2015-04-01 -> 2019-11-30)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2015				11.013	12.546	17.542	15.499	9.956				14.498
2016			11.267	11.357		10.668	10.664	11.37	9.883	9.884	10.02	
2017				13.68	19.377	10.777	9.907	9.878	9.862	9.893	10.012	
2018			10.506	11.397	10.066	12.779	9.885	10.06	9.978	9.928	10.427	11.943
2019		14.74	14.911	15.704	18.004	15.983	10.691	9.978	11.178	10.884	11.482	
Mean of monthly Gage height		14.74	12.23	12.63	15	13.55	11.33	10.25	10.23	10.15	10.49	13.22

** No Incomplete data have been used for statistical calculation

Gage Datum, feet above NAVD88	510.44
Average Gage Height, feet	12.165
Sugar Creek Average Surface Water Elevation, feet above NAVD88	522.61

NO.	DATE	REVISION DESCRIPTION	BY

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APPROVED BY: BJH | DESIGNED BY: MTH | DRAWN BY: MPN

CONCEPTUAL MODEL

PLANS PREPARED FOR
CITY, WATER, LIGHT, AND POWER
SPRINGFIELD, SANGAMON COUNTY, ILLINOIS

DATE: AUGUST 2021

PROJECT ID: 200325/0003

SHEET NUMBER:

FIG. 4

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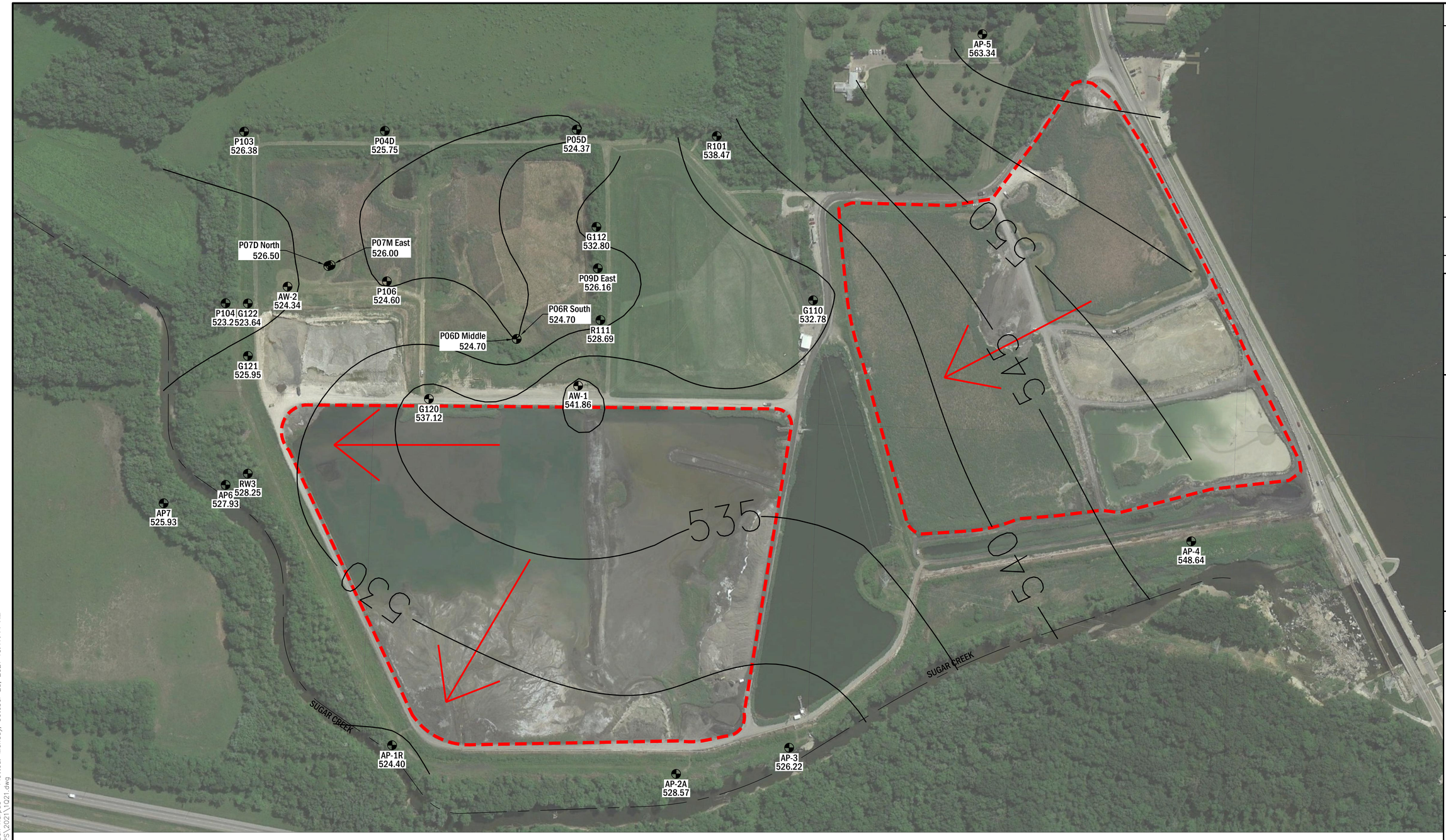
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J:\S\Springfield CWLP\CWLP Ash Pond\DWG\2021\Conceptual Model.dwg

TABLE 1
PROPOSED MONITORING NETWORK
Power Plant Ash Impoundment
CWLP

Well ID	Monitoring Position	Material Screened	Ground Surface Elevation (ft MSL)	Top of Casing Elevation (ft MSL)	Approximate Screen Interval (ft MSL)		
AP-1R	Downgradient	Bedrock/unconsolidated interface	533.1	535.6	511.5	-	502.1
AP-2A	Downgradient	Bedrock/unconsolidated interface	533.6	536.1	525.2	-	515.1
AP-3	Downgradient	Bedrock/unconsolidated interface	533.7	535.6	523.7	-	513.7
AP-4	Upgradient	Bedrock/unconsolidated interface	553.9	554.6	503.9	-	493.9
AP-5	Upgradient	Bedrock/unconsolidated interface	581.6	583.9	563.8	-	553.9
AP-6	Downgradient	Bedrock/unconsolidated interface	534.8	537.8	508.4	-	498.6
AP-7	Downgradient	Bedrock/unconsolidated interface	536.1	539	506.8	-	497.1
AP-8	Downgradient	Bedrock/unconsolidated interface	537.2	540.3	508.6	-	498
AP-10	Downgradient	Bedrock/unconsolidated interface	534.4	537.5	509.7	-	499.9
AP-14	Downgradient	Bedrock/unconsolidated interface	536.8	539.6	519.5	-	509.8
RW-3	Downgradient	Bedrock/unconsolidated interface	536.8	539.3	505.5	-	495.9

APPENDIX C

POTENTIOMETRIC SURFACE MAPS



LEGEND

- MONITORING WELL LOCATION
- CCR SURFACE IMPOUNDMENT
- GROUNDWATER FLOW DIRECTION

NOTES

1. SAMPLING DATE: 1/22/2021
2. CONTOUR INTERVAL = 5 FEET
3. IMAGE SOURCE: GOOGLE EARTH PRO, IMAGE DATE MAY 15, 2012
4. ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM (1988).

N

0 150 300 600

SCALE: IN FEET

NO.	DATE	REVISION DESCRIPTION	BY

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APPROVED BY: BJH | DESIGNED BY: MTH | DRAWN BY: BCK

POTENTIOMETRIC SURFACE MAP
1ST QUARTER 2021

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SPRINGFIELD, SANGAMON COUNTY, ILLINOIS

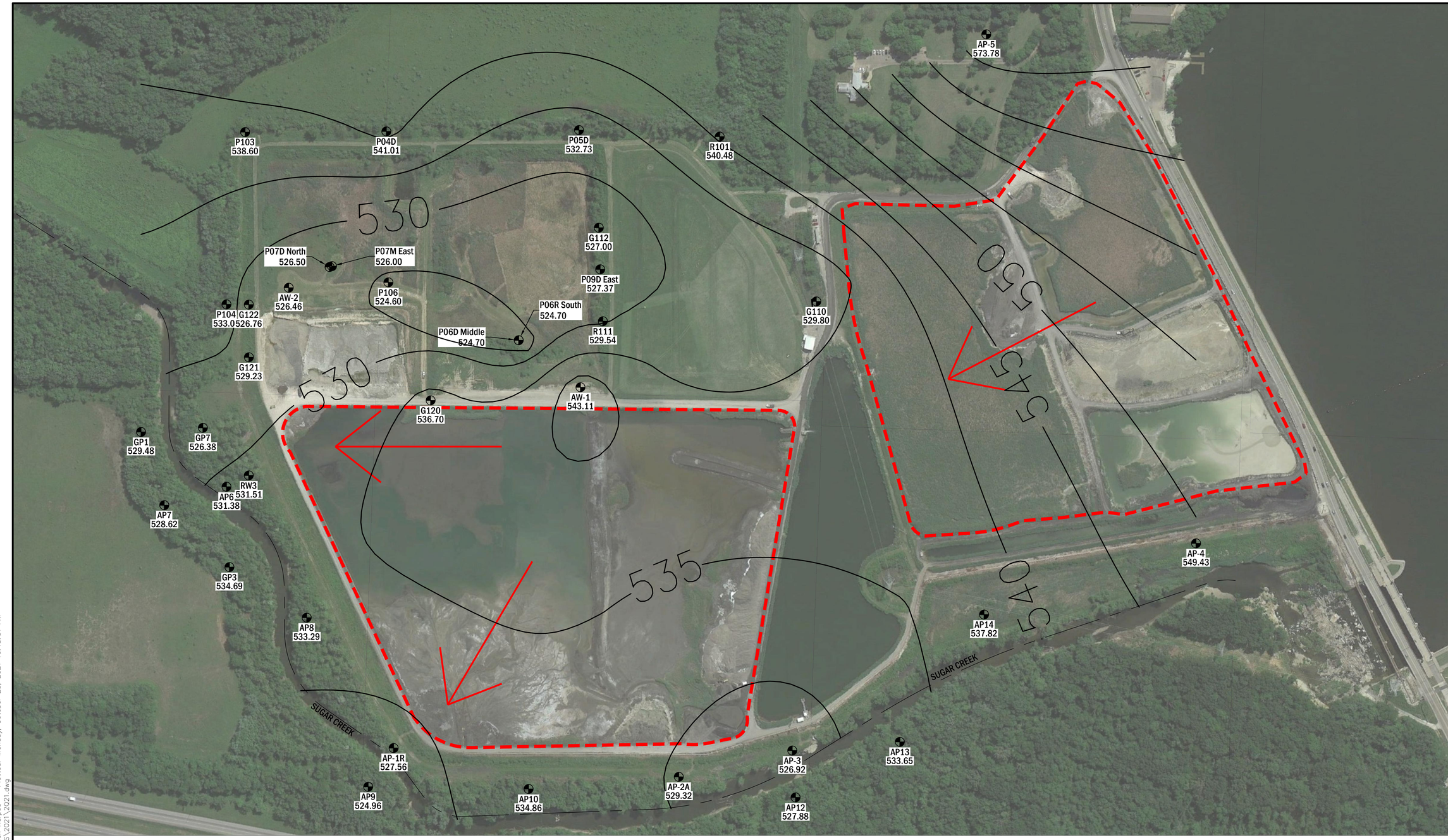
DATE: OCTOBER 2021

PROJECT ID: 150077/0021

SHEET NUMBER: 1Q21

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Tab: 1Q21 Last Saved: October 25, 2021, by Ben Korpus Plotted: Monday, October 25, 2021 10:17:44 AM
J:\S\Springfield CWLP\CWLP Ash Pond\DWG\PMAPS\2021\1Q21.dwg



LEGEND

- MONITORING WELL LOCATION
- CCR SURFACE IMPOUNDMENT
- GROUNDWATER FLOW DIRECTION

NOTES

1. SAMPLING DATE: 5/27/2021
2. CONTOUR INTERVAL = 5 FEET
3. IMAGE SOURCE: GOOGLE EARTH PRO, IMAGE DATE MAY 15, 2012
4. ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM (1988).

N

0 150 300 600

SCALE: IN FEET

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DESIGNED BY: MTH | DRAWN BY: BCK | APPROVED BY: BJH

POTENTIOMETRIC SURFACE MAP
2ND QUARTER 2021

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DATE: OCTOBER 2021

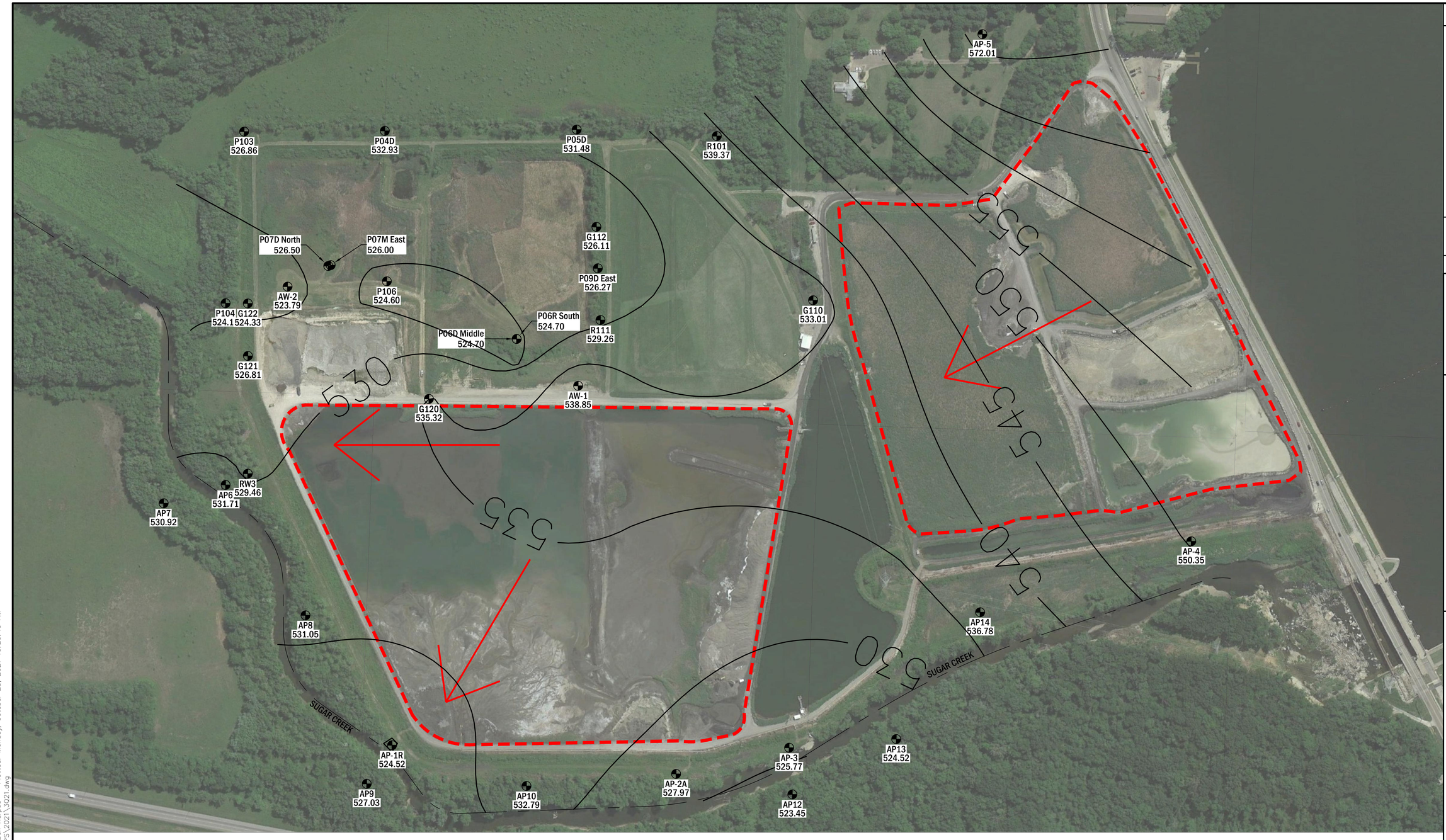
PROJECT ID: 150077/0021

SHEET NUMBER:

2Q21

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J:\S\Springfield CWLP\CWLP Ash Pond\DWG\PMAPS\2021\2021.dwg



LEGEND

- MONITORING WELL LOCATION
- CCR SURFACE IMPOUNDMENT
- GROUNDWATER FLOW DIRECTION

NOTES

1. SAMPLING DATE: 8/25/2021
2. CONTOUR INTERVAL = 5 FEET
3. IMAGE SOURCE: GOOGLE EARTH PRO, IMAGE DATE MAY 15, 2012
4. ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM (1988).

N

0 150 300 600

SCALE: IN FEET

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APPROVED BY: BJH | DESIGNED BY: MTH | DRAWN BY: BCK

POTENTIOMETRIC SURFACE MAP
3RD QUARTER 2021

PLANS PREPARED FOR
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SPRINGFIELD, SANGAMON COUNTY, ILLINOIS

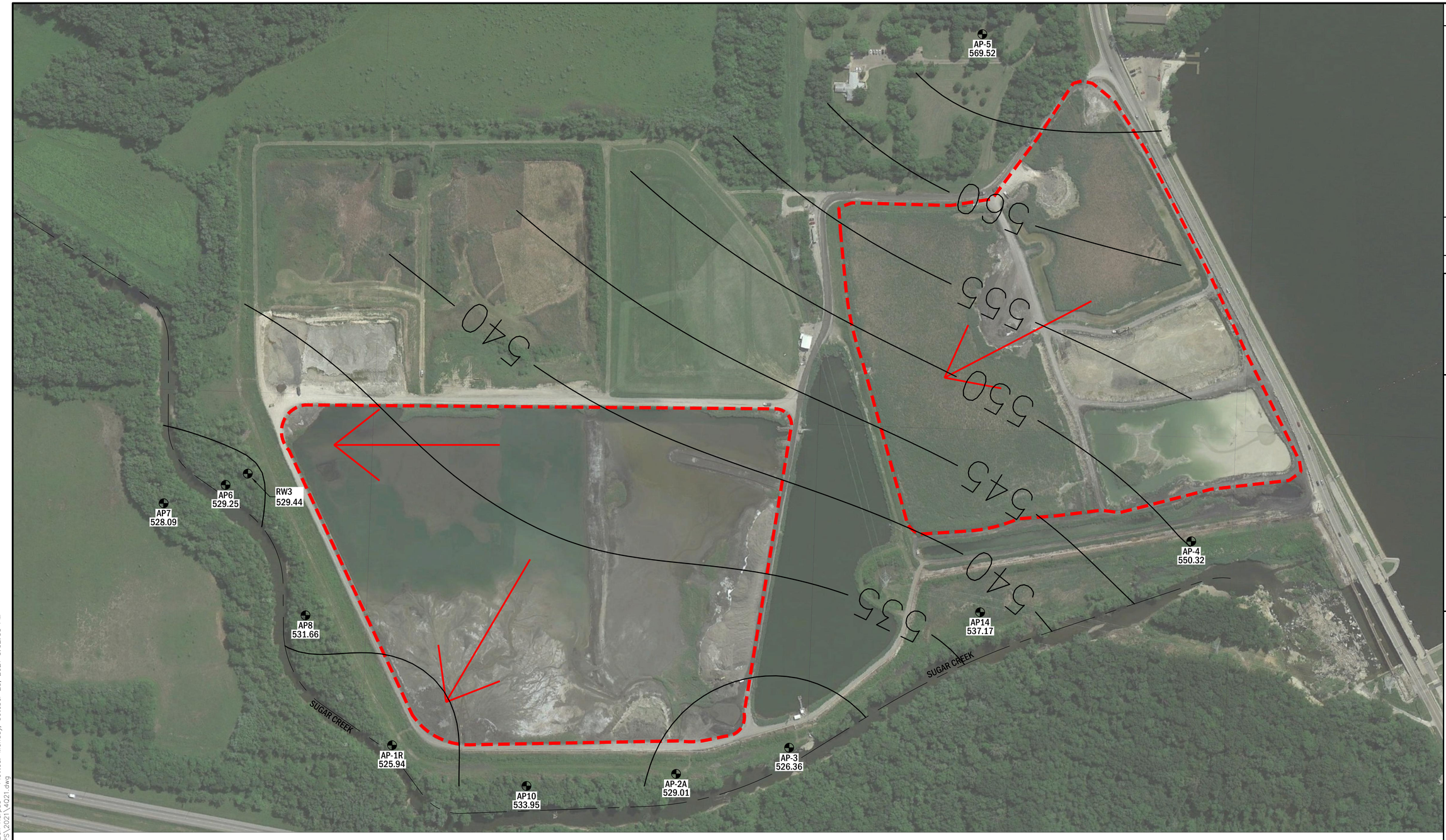
DATE: OCTOBER 2021

PROJECT ID: 150077/0021

SHEET NUMBER: 3Q21

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Tab: 3Q21 Last Saved: October 25, 2021, by Ben Korpus Plotted: Monday, October 25, 2021 10:20:40 AM
J:\S\Springfield CWLP\CWLP Ash Pond\DWG\PMAPS\2021\3Q21.dwg



LEGEND

- MONITORING WELL LOCATION
- CCR SURFACE IMPOUNDMENT
- GROUNDWATER FLOW DIRECTION

NOTES

1. SAMPLING DATE: 10/21/2021
2. CONTOUR INTERVAL = 5 FEET
3. IMAGE SOURCE: GOOGLE EARTH PRO, IMAGE DATE MAY 15, 2012
4. ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM (1988).

N

0 150 300 600

SCALE: IN FEET

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APPROVED BY: BJH | DESIGNED BY: MTH | DRAWN BY: BCK

POTENTIOMETRIC SURFACE MAP
4TH QUARTER 2021

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DATE: OCTOBER 2021

PROJECT ID: 150077/0021

SHEET NUMBER: 4Q21

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J:\S\Springfield CWLP\CWLP Ash Pond\DWG\PMAPS\2021\4Q21.dwg