SECTION 2.0

Introduction

2.0 INTRODUCTION

Section 2 is provided to demonstrate that the proposed landfill expansion design and operations comply with Title 35 Illinois Administrative Code (35 Ill. Adm. Code) Part 811.

Regulatory standards at the federal, state, and local levels, as well as engineering and geological expertise and judgement, were utilized in analyzing and completing the proposed facility design, in choosing its location, and in planning its operations.

The following sections are provided within this report:

2.1 – Location
2.2 – Hydrogeologic Investigation
2.3 – Design
2.4 – Stormwater Management Plan
2.5 – Construction Quality Assurance Plan
2.6 – Operating Plan
2.7 – Groundwater Impact Assessment
2.8 – Environmental Monitoring
2.9 – Closure and Post-Closure Plan

An IEPA Compliance Summary Table is located in Appendix A to assist in the IEPA's review of this application.

SECTION 2.1

Location

2.1 LOCATION

Introduction

The proposed Site 2 North Expansion (Expansion) at Zion Landfill (Landfill) is appropriately located to be protective of the public health, safety, and welfare. This section of the application provides a brief summary of the character of the surrounding area and demonstrates that all applicable location standards have been met or exceeded. Key findings include the following:

- The proposed Expansion meets all federal, state, and local location criteria specified in the applicable Illinois landfill regulations.
- Operations have been conducted at the Landfill since at least 1976. An expansion of the Landfill is consistent with historical uses.

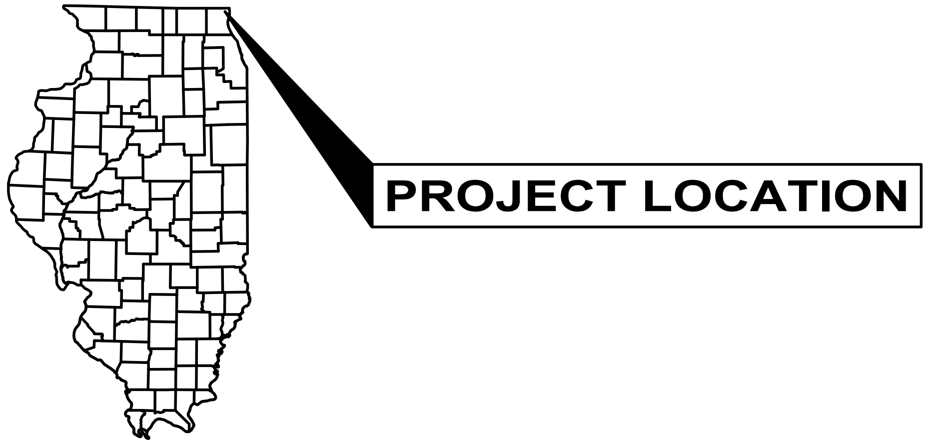
Proposed Location

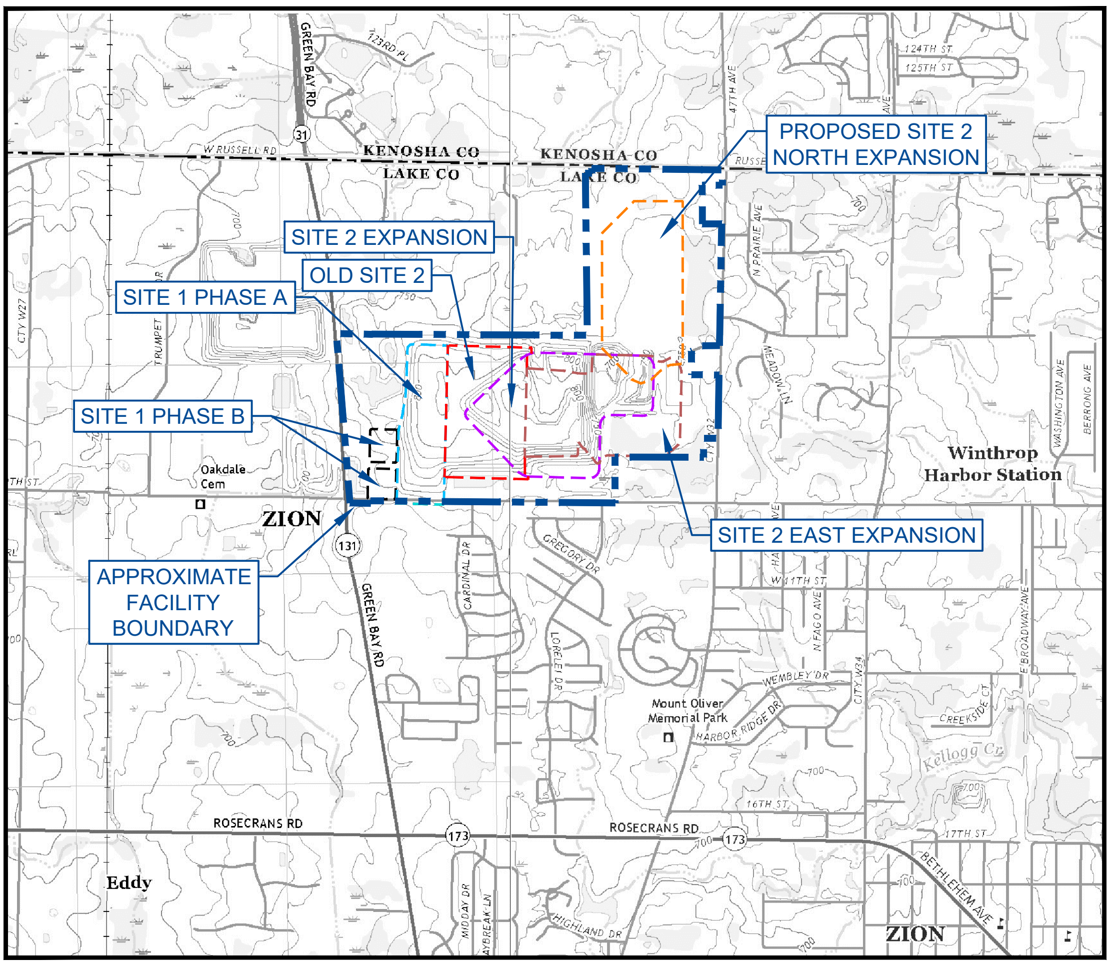
The existing Landfill is located within the City of Zion, Lake County, Illinois. The existing Landfill is situated within Section 7, Township 46 North, Range 12 East of the Third Principal Meridian. The existing Landfill facility is generally bounded to the east by Kenosha Road, to the west by Green Bay Road, to the south by 9th Street and to the north by a tree nursery, golf course, and residential properties along Kenosha Road. The location of the existing Landfill was evaluated during the siting and permitting process of its Site 2 East Expansion to ensure that it met all federal, state, and local location criteria specified in the applicable Illinois landfill regulations.

Similar to the existing Landfill, the proposed Expansion will also be wholly located within the City of Zion, Lake County, Illinois. The Expansion will increase the footprint of the existing Landfill to the north in the location of the existing tree nursery and current residential properties along Kenosha Road that are owned by Zion Landfill, Inc. The proposed Expansion area will be bounded to the west by a golf course, to the north by Russell Road, and to the east by Kenosha Road and residential properties. The proposed Expansion is located within Section 6 of Township 46 North, Range 12 East of the Third Principal Meridian. The existing Landfill and the proposed Expansion boundaries are shown on a topographic map on **Figure 2.1-1** and on an aerial photograph on **Figure 2.1-2**.

The general land uses around the overall proposed Landfill are residential, commercial, recreational, agricultural, and industrial. The proposed Landfill will be bounded by the North Shore Sanitary District landfill and a metal scrap yard to the west, a golf course and agricultural lands to the north, and residences and agricultural lands to the east and south. A parcel of land owned by the City of Zion Park District is adjacent to the southeast portion of the Facility.

A site location Map containing the private well location information in a one-mile radius around the existing Landfill and proposed Expansion is provided on **Drawing No. G2**. A topographic map of the proposed Landfill boundary and its location relative to various features described in subsequent text is shown on **Drawing No. D2**.





0 3000'

GRAPHIC SCALE

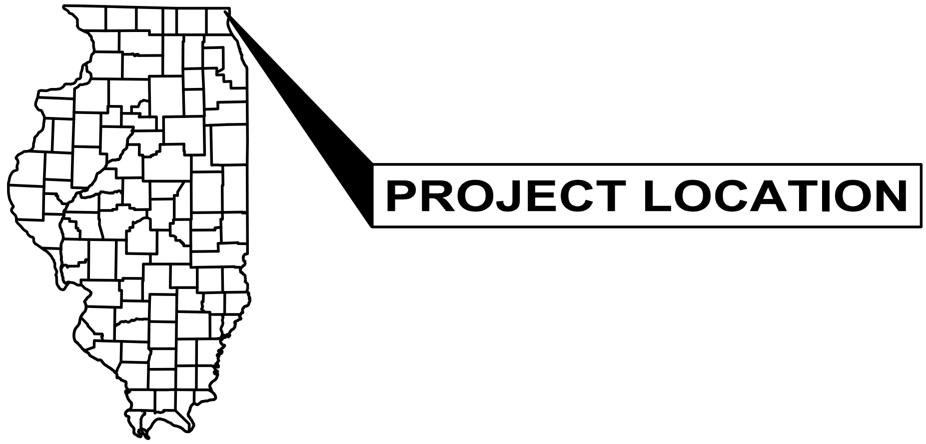
FIGURE ADAPTED FROM USGS TOPOGRAPHIC QUADRANGLES FOR ZION (2018) AND WADSWORTH (2018).

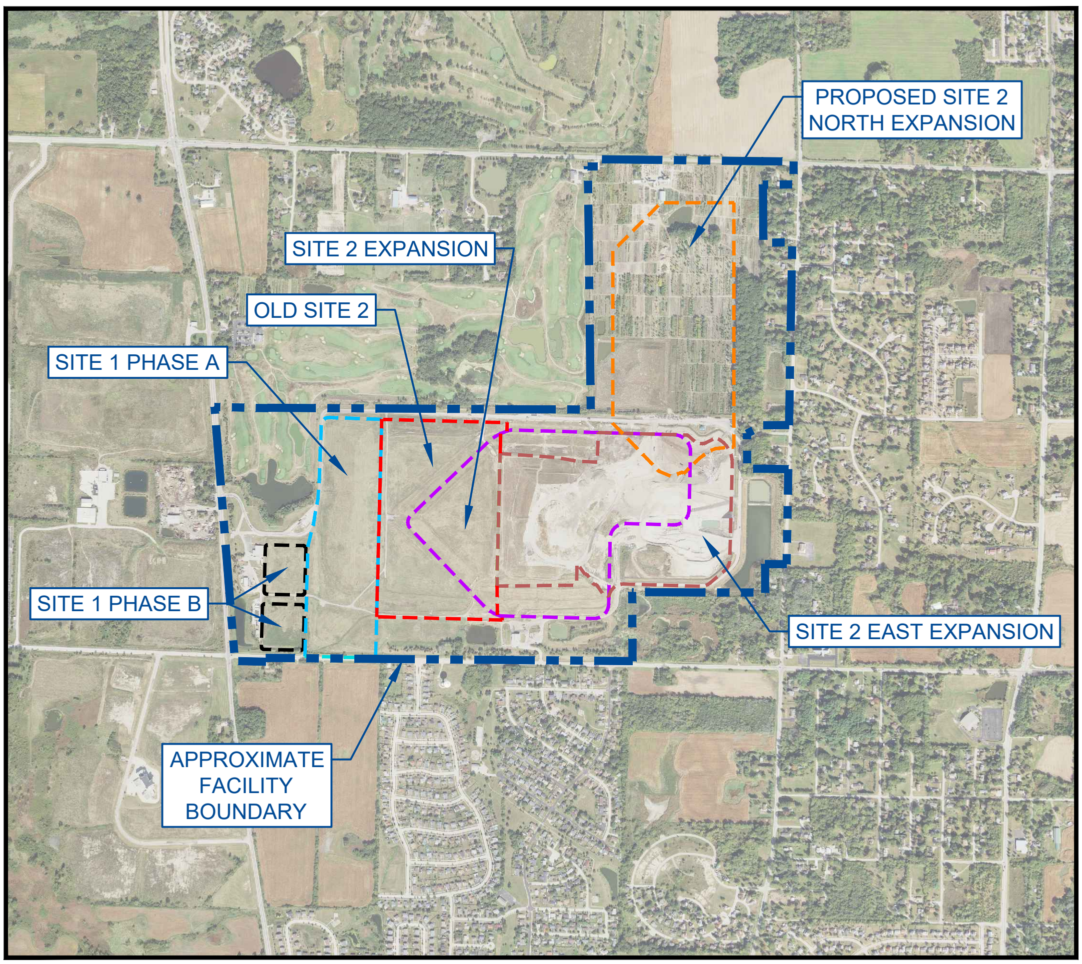
APTIM Environmental & Infrastructure, LLC

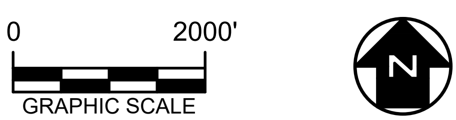
ZION LANDFILL SITE 2 NORTH EXPANSION SITE LOCATION REPORT

FIGURE 2.1-1
SITE TOPOGRPAHIC MAP

APPROVED BY:	MNF	PROJ. NO.:	631020105	DATE:	MAY 2022







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APTIM

ZION LANDFILL SITE 2 NORTH EXPANSION SITE LOCATION REPORT

FIGURE 2.2-2
SITE AERIAL MAP

APPROVED BY: MNF	PROJ. NO.: 631020105	DATE: MAY 2022

Site History

Zion Landfill was initially permitted in 1976 and owned by BFI Waste Systems of North America, Inc. BFI operated the site until July 30, 1999 when Allied Waste Industries, Inc. acquired Browning-Ferris Industries, Inc., which was the parent company of BFI Waste Systems of North America, Inc. On March 31, 2000, Allied sold the site to Superior Zion Landfill, Inc. On the same day, Superior Zion Landfill, Inc. changed its name to Onyx Zion Landfill, Inc. On July 1, 2006, Onyx Zion Landfill, Inc. changed its name to Veolia E.S. Zion Landfill, Inc. On December 26, 2012, Veolia E.S. Zion Landfill, Inc. changed its name to Advanced Disposal Services Zion Landfill, Inc. On October 29, 2020, Advanced Disposal Services Zion Landfill, Inc. changed its name to Zion Landfill, Inc. and is the present owner and operator of the Zion Landfill.

As depicted on **Figure 2.1-1** and **Figure 2.1-2**, the existing Facility consists of two older units that have ceased acceptance of waste and are closed (Site 1 Phase A and Site 1 Phase B), as well as the currently active unit referred to as the Site 2 Landfill (Landfill). The currently active Site 2 Landfill includes an older, closed section (Old Site 2), as well as two prior expansion areas that comprise the open, operating portions of the Facility. The proposed Site 2 North Expansion that is the subject of this application will be the third expansion of the Site 2 Landfill. The Landfill is permitted by the Illinois IEPA (Site No. 0978020002).

The original area of the Site 2 Landfill, referred to as Old Site 2, is a non-hazardous solid waste unit that was regulated under 35 IAC, Part 807. Old Site 2 commenced landfilling operations on December 23, 1981, pursuant to IEPA Permit No. 1980-24-DE. In 1993, a final cover system was constructed over the site. Siting approval for the first Site 2 Expansion (initially identified as Site 3 at that time) was granted by the Zion City Council on April 17, 1995 which approved a new landfill unit east of Old Site 2 including a “piggyback” onto the eastern portion of Old Site 2. The Site 2 Expansion was originally permitted under 35 IAC, Part 812, Subparts A and C, and is now regulated under 35 IAC, Part 811 regulations, which meet or exceed Subtitle D Federal landfill regulations.

A second expansion, referred to as the Site 2 East Expansion, included vertical and an approximate 26.5-acre horizontal expansion to the east of the previous Site 2 Expansion footprint. The initial phase of the Site 2 East vertical expansion was permitted on June 3, 2011, with the remainder of the expansion approved for development on June 13, 2014. The Site 2 East Expansion is regulated under 35 IAC, Part 811 regulations.

Proposed Facility Overview

The proposed Expansion includes a horizontal and vertical component. The proposed horizontal Expansion will advance the existing Landfill to the north within the tree nursery and other properties owned by Zion Landfill, Inc. The horizontal Expansion will expand the waste unit boundary of the existing Landfill by 62.2 acres and will expand the overall facility boundary 124 acres to the north. The proposed vertical Expansion will tie into the Site 2 East Expansion portion of the existing Landfill by vertically expanding over its north sideslopes. The proposed Expansion will have an approximate peak elevation of 892 ft MSL, which is roughly 38 feet lower in elevation than the Site 2 East Expansion peak elevation of 930 ft MSL.

The horizontal component of the Expansion will expand the waste footprint of the existing Site 2 East Expansion approximately 62.2 acres to the north, as shown on **Figure 2.1-1**. An aerial overview of the Expansion area is provided as **Figure 2.1-2**. **Drawing No. D3** shows an overview of the proposed Landfill and its location relative to the location standard features described in subsequent text.

Legal Description

Legal descriptions of the existing Facility boundary, existing Facility waste footprint, and proposed Expansion footprint are provided in **Appendix D**. A plat of survey for the proposed Facility is also included.

Site Location Standards

Illinois landfill regulations contain standards that restrict where landfills may be developed (35 IAC, Sections 811.102 and 811.302). Federal regulations and statutes also contain location requirements. The collective purpose of each of these location standards and requirements is to protect public health, safety and welfare, the environment, and the structural integrity of the engineered landfill. For example, some standards specify a minimum setback distance between landfills along with other land uses such as airports or schools. Other standards specify minimum setback distances from environmentally sensitive areas such as wetlands, rivers and streams, and scenic and natural areas. Additionally, other standards specify minimum distances from conditions that could affect the structural integrity of the landfill, such as seismic zones.

The proposed Expansion will comply with all applicable federal, state, and local site location standards. **Drawing No. D2** shows the location of the proposed Facility and demonstrates that the Facility falls outside the applicable setback distances. **Appendix F** supplements these drawings when other maps, such as floodplain maps, are more appropriate to display setback compliance.

The following text summarizes the location requirements and demonstrates that the facility complies with all requirements. The facility will meet or exceed all requirements for federal, state, and local location criteria. Documentation supporting the conclusions presented in this section is included in **Appendix F**.

Airport Standards

Regulatory Requirements

Illinois Administrative Code (IAC) Title 35 Section 814.302 (c)

- *The facility shall not be located within 10,000 feet of any runway used by turbojet aircraft unless demonstration is placed in the operating record that the landfill unit is designed and operated so as not to pose a bird hazard to aircraft.*
- *The facility shall not be located within 5,000 feet from any runway used by piston aircraft unless demonstration is placed in the operating record that the landfill unit is designed and operated so as not to pose a bird hazard to aircraft.*
- *The owner or operator proposing to locate a lateral expansion within a five-mile radius of a public use airport runway must notify the airport and the FAA when a permit is being filed with the IEPA.*

49 U.S.C. § 44718(d), as amended by Section 503 of the Wendell H. Ford Aviation Investment and Reform Act for the 21st Century, Public Law No. 106-181

- *Prohibits the establishment of a new landfill within 6 miles of a public airport served by general aviation aircraft with regularly scheduled flights of aircraft designed for 60 passengers or less unless exempted by the state aviation agency and FAA.*
- *This prohibition is not applicable to expansions or modifications of landfills which were constructed or established prior to the date of enactment (April 5, 2000).*
- *FAA Advisory Circular No. 150/5200-34A dated January 26, 2006 provides guidance for FAA application of this regulation. FAA Advisory Circular No. 150/5200-33B dated August 28, 2007 provides guidance on land uses that have the potential to attract hazardous wildlife on or near airports, including landfills.*

Compliance with the Standard

There are no public or private use air operations areas used by turbojet aircraft located within 10,000 feet of the proposed Landfill, including the proposed Expansion area. The nearest airport listed on the Illinois Department of Transportation Airport Inventory Report 2012 is the Waukegan National Airport and is approximately 4.1 miles south of the proposed Expansion.

The Maas Landing Strip, a privately-owned landing strip, is located over 5,000 feet from the overall Landfill boundary to the southwest. The Maas Landing Strip is used only by piston aircraft.

The Wendall H. Ford Act, 39 USC 44718(d)(2), specifically permits the expansion of any landfill that was established before April 5, 2000. The existing Landfill Site 2 Expansion was permitted in March 1997 by the IEPA. Furthermore, the Waukegan National Airport does not primarily accept scheduled flights of aircraft designed for 60 passengers or less. Nonetheless, notification of the proposed expansion will be provided to the FAA and the Waukegan National Airport at the time of submission of this IEPA permit application.

References

1. **Drawing No. D2**
2. IDOT, Airport Inventory Report 2012, June 2012, **Appendix F.1**
3. Public and Private Airport Listings, Kenosha County, WI and Lake County, IL, www.tollfreeairline.com, **Appendix F.1**

Floodplain Standards

Regulatory Requirements

IAC Title 35 Section 811.102 (b)

- *The facility shall not 1) restrict the flow of a 100-year flood, 2) reduce the temporary water storage capacity of the 100-year floodplain, or 3) result in washout of solid waste from the 100-year flood.*

Section 39.2 (a)(iv) of the Illinois Environmental Protection Act

- *The facility (landfill or waste disposal site) is located outside the boundary of the 100-year floodplain, or if the Facility is a facility described in subsection (b) of Section 22.19 (a), the site is flood-proofed.*

Compliance with the Standard

The Landfill, including the proposed Expansion area, is not within a 100-year floodplain, as defined by the Federal Emergency Management Agency (FEMA). Furthermore, the proposed Expansion will not restrict the flow of a 100-year flood from off-site areas, reduce the temporary water storage capacity, or result in washout of solid waste. The site stormwater management plan has also been designed to control surface water flow on and around the existing Landfill and proposed Expansion to minimize flooding.

The overall proposed Landfill therefore meets requirements of both Title 35 Section 811.102(b) and Section 39.2(a)(iv) of the Act. Please refer to **Appendix F.2** for identified floodplains within the vicinity of the proposed Facility, as provided by FEMA through the Flood Insurance Rate Map (FIRM) Panels 17097C0057K and 17097C0076K, September 17, 2013.

References

1. Floodplain Location Map, **Appendix F.2**
2. FEMA NFHL FIRM Database for Lake County, Illinois, FIRM IDs 17097C0057K and 17097C0076K, September 17, 2013

Wetlands and Waters of the U.S.

Regulatory Requirements

IAC Title 35 Section 811.102(e)

- *The facility shall not cause a violation of Section 404 of the Clean Water Act.*

Compliance with the Standard

A wetland delineation study was completed by Hampton, Lenzini, and Renwick, Inc. (HLR), in accordance with the current U.S. Army Corps of Engineers (USACOE) methodology to determine whether wetlands and Waters of the U.S. are located within the proposed Expansion development footprint. HLR identified wetlands at the facility. A Preliminary Jurisdictional Determination (PJD) was completed to determine which waters were Waters of the U.S. and which were considered Isolated Waters of Lake County. A copy of the PJD and the concurrence of findings is provided in **Appendix F.3** identifying Sites 2, 3, 4, and 7 as Waters of the U.S. and Sites 1, 5, and 6 as Isolated Waters of Lake County. Though Site 6, a permitted detention basin for a temporary soil stockpile, exhibited wetland characteristics and the PJD indicated that it may be an Isolated Water of Lake County, the Zion Landfill has since coordinated with the Lake County Stormwater Management Commission and received their approval to exclude Site 6 as an Isolated Water of Lake County. Interagency correspondence indicating the exclusion of this site as an Isolated Water of Lake County is provided in **Appendix F.3**. In total, approximately 1.18 acres are classified as Waters of the U.S. and 1.41 acres are classified as Isolated Waters of Lake County. The PJD issued in November 2019 also indicated a potential variation to the acreage of sites classified as Waters of the U.S., with direction to coordinate with the Lake County Stormwater Management Commission to confirm wetland site boundaries. This coordination was completed in late November 2019 with submittal of the revised report contained in **Appendix F.3** in December 2019, which reflects the boundaries and acreages as agreed with the Lake County Stormwater Management Commission.

An Individual Permit from the USACOE (complying with Section 404 of the Clean Water Act) will be required prior to the development of the proposed Expansion, which will include mitigating loss of Waters of the U.S. and/or jurisdictional wetlands as required by the USACOE. No construction will take place within jurisdictional wetlands or Waters of the U.S. prior to receiving this permit. Similarly, a Wetland Development Permit will be secured from the Lake County Stormwater Management Commission prior to development, which will include mitigating loss of Isolated Waters of Lake County.

References

1. Hampton, Lenzini, and Renwick, Inc., Wetland Delineation Report, December 2019 and associated communication, **Appendix F.3**

Fault Areas

Regulatory Requirements

IAC Title 35 Sections 811.304 and 811.305

- *The facility shall not be located within 200 feet of faults that have displaced during the Holocene Epoch (10,000 years), without the approval of the State.*

Compliance with the Standard

There are no known faults that have displaced during the Holocene Epoch within 200 feet of the Landfill, including the proposed Expansion area.

References

1. Nelson, W. John, Structural Features in Illinois, Illinois State Geological Survey, Bulletin 100, 1995
2. Hydrogeologic Report, **Section 2.2**

Unstable Areas

Regulatory Requirements

IAC Title 35 Sections 811.304 and 811.305

- *The Facility (landfill or waste disposal unit) shall not be located in an unstable area unless engineering measures have been incorporated to ensure the integrity of the structural components.*

Compliance with the Standard

There are no documented unstable areas beneath the excavation of the existing Landfill or proposed Expansion. Based on an ISGS coal mine map for Lake County, there are no recorded coal mines within the vicinity of the existing landfill or proposed Expansion. Site specific studies have not identified site characteristics conducive to the formation of karst features nor the presence of coal mining.

References

1. Hydrogeologic Report, **Section 2.2**

Seismic Impact Zones

Regulatory Requirements

IAC Title 35 Section 811.304

- *The facility shall not be located within a seismic impact zone (10% or greater chance of exceeding 0.10g in 250 years) unless all containment structures are designed to restrict the maximum horizontal acceleration for the site.*

Compliance with the Standard

The proposed Expansion has been designed to safely withstand the maximum horizontal acceleration anticipated at the Facility. The proposed Expansion has been designed to achieve a safety factor greater than 1.3 against slope failure under seismic conditions. The proposed facility is not located within a seismic impact zone (having a 10% or greater chance of exceeding 0.10g in 250 years) as illustrated in **Figure F.4** (refer to **Appendix F.4**). There is a 10% or greater chance of earthquake-induced horizontal ground motions exceeding 0.0461g in 250 years.

References

1. Map of Horizontal Acceleration, **Appendix F.4**
2. United States Geological Survey, Earthquake Hazards Program - National Seismic Hazard Mapping Project, **Appendix F.4**
3. Hydrogeologic Report, **Section 2.2**
4. Design Report, **Section 2.3**

Wild and Scenic Rivers

Regulatory Requirements

IAC Title 35 Sections 811.102(a)

- *The facility shall meet all requirements under the Wild and Scenic River Act.*

Compliance with the Standard

There are no rivers designated for protection under the National Wild and Scenic Rivers Act within the proposed Expansion watershed.

The only river in Illinois classified as wild and/or scenic on the National System List is the Middle Fork of the Vermillion River (Refer to #114 in below referenced table), which is not located in Lake County.

References

1. River Mileage Classification for Components of the National Wild and Scenic Rivers System, http://www.rivers.gov/publications/riverstable.pdf, January 2016, **Appendix F.5**

Historic and Natural Areas

Regulatory Requirements

IAC Title 35 Section 811.102(c)

- *The facility shall not pose a threat of harm or destruction to features for which a: 1) Historic Site, 2) Archaeological Site, 3) Natural Landmark, or 4) Natural Area was designated.*

Compliance with the Standard

In 2019, a Phase I Archaeologic Investigation was completed by the University of Illinois at Urbana-Champaign Public Service Archaeology and Architecture Program. The Phase I Evaluation included all proposed facility expansion areas beyond the currently permitted facility boundary. This investigation identified one homestead (potential archaeological site) and three structures located within the site. The Phase I recommends that all structures should be determined “not eligible” for listing on the National Register of Historic Places. However, a Phase II Archaeologic Investigation was subsequently completed to determine whether the site is eligible to be listed on the National Register of Historic Places. The Phase II Investigation involved entering the structures and completing subsurface investigations of the homestead area. Based on these investigations, it was the opinion of the investigating archaeologist that the structures and homestead were unlikely to be eligible to be designated for National Register for Historic Places Listing. A formal determination was requested by the State Historic Preservation Office of the Illinois Department of Natural Resources. Within this determination request, the archaeologist provides his recommendation that the site is not eligible for listing on the National Register of Historic Places. At the time of this application, no response has been received. However, Zion Landfill, Inc. provides assurance as part of this application that, in the unlikely event that a historic site is designated within the expansion development area, appropriate steps will be taken to ensure compliance with all appropriate regulations.

Previous Phase I Archaeological Surveys were performed in 2007 by Allied Archaeology and in 1994 by the Public Service Archaeology Program. The Phase I studies were conducted as part of the Site 2 East Expansion application and the 1995 Expansion application. Neither survey found evidence of materials that meet the requirements of Section 4 of the Illinois State Agency Historic Resources Preservation Act. In correspondence dated September 9, 2007, the Illinois Historic Preservation Agency agreed with the findings of the most recent Phase I Archaeological Survey and determined that there are no significant historical, architectural, or archaeological resources or sites located within the proposed Expansion project area. Similar correspondence was also provided by the Illinois Historic Preservation Agency on December 8, 1994, stating that there were no significant historical, architectural or archaeological resources within the 1994 investigation area.

No national natural landmarks will be impacted as part of this Expansion. Only three national natural landmarks exist in Lake County, including: Volo Bog Nature Preserve, Wauconda Bog Nature Preserve, and Illinois Beach Natural Preserve. None of these sites are within the proposed Landfill boundary (see **Appendix F.6**). The closest national landmark is Illinois Beach Nature Preserve, approximately six miles southeast of the Expansion, and therefore will not be impacted by Expansion development and operations.

The State of Illinois Ecological Compliance Assessment Tool (EcoCAT) was accessed for records of Illinois Natural Area Inventory sites. The EcoCAT report stated that the Illinois Natural Heritage Database contains no record of Illinois Natural Area Inventory sites in the vicinity of the proposed Expansion (see **Appendix F.7**).

References

1. Public Service Archaeology & Architecture Program of the University of Illinois at Urbana-Champaign, Archaeological Reconnaissance of the 125-Acre Zion Landfill 2019 Expansion in Lake County, Illinois, September 2019, **Appendix F.6**
2. Previous Archaeologic Investigations and Communications Associated with Existing Facility, **Appendix F.6**
3. EcoCAT report, **Appendix F.7**
4. National Park Service website, http://www.nature.nps.gov/nnl/nation.cfm, accessed January 25, 2019, **Appendix F.6**

Endangered Species

Regulatory Requirements

IAC Title 35 Sections 811.102(d)

- *The facility shall not jeopardize or take any endangered species, result in the destruction of critical habitat for such species, or contribute to the taking of endangered or threatened species.*

Compliance with the Standard

The State of Illinois Ecological Compliance Assessment Tool (EcoCAT) was accessed for records of State-listed threatened or endangered species, Illinois Natural Area Inventory sites, dedicated Illinois Nature Preserves, and registered Land and Water Reserves. The EcoCAT report stated that the Illinois Natural Heritage Database contains no record of State-listed threatened or endangered species, Illinois Natural Area Inventory sites, dedicated Illinois Nature Preserves, or registered Land and Water Reserves in the vicinity of the proposed Facility.

The facility will consult with the U.S. Fish and Wildlife Service as necessary prior to development of the proposed Expansion to ensure that the development and operations of the existing and proposed Landfill will not impact any potentially endangered or threatened species.

References

1. EcoCAT report, **Appendix F.7**

Water Quality Management Plan

Regulatory Requirements

IAC Title 35 Sections 811.102(f)

- *The facility shall not cause a violation of any area-wide or state-wide water quality management plan for non-point source pollution.*

Compliance with the Standard

No area-wide or state-wide water quality management plans are enforced in the location of the proposed expansion. However, several steps will be taken to ensure that non-point source pollution does not occur. A National Pollutant Discharge Elimination System (NPDES) permit will be obtained prior to the commencement of any construction activities which disturb more than one acre. Stormwater from the developed portions of the landfill will be directed to detention basins until final cover is established. The detention basins will discharge via engineered outlet structures. The outlet structures will be identified as point sources in the NPDES permit. Therefore, the facility will not violate an area-wide or state-wide water quality management (WQM) plan for non-point source pollution.

Prior to developing the landfill expansion, a major development permit will be secured from the Lake County Stormwater Management Commission following the Lake County Watershed Development Ordinance requirements. The extensive stormwater management features constructed during the proposed Expansion development will reduce the potential for downstream flooding and improve the runoff rate against existing conditions.

References

1. Stormwater Management Plan, **Section 2.4**

Water Supply Wells Setback

Regulatory Requirements

IAC Title 35 Sections 814.302(a)

- *No part of a new unit shall be located within the setbacks established in Sections 14.2 and 14.3 of the Act, i.e., within 200 feet of a potable water supply well. A maximum setback zone may be established for a community water supply well in accordance with Section 14.2 of the Act.*

Compliance with the Standard

No known water supply wells are located within 200 feet of the proposed expanded landfill waste boundary, nor are there any community water supply wells located within 1,000 feet of the proposed Landfill waste boundary, per the setback zones defined in the Illinois Environmental Protection Act.

The locations of all the known and recorded wells in the surrounding area of the site are shown on **Drawing No. G2** and discussed in the hydrogeological report.

References

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1. Hydrogeologic Report, **Section 2.2**
2. **Drawing No. G2**
3. Illinois State Geological Survey and Illinois State Water Survey Well Logs, **Appendix G**

Sole-Source Aquifers and Regulated Recharge Areas

Regulatory Requirements

IAC Title 35 Sections 811.302(b)

- *No part of a unit shall be located within 1,200 feet vertically or horizontally of a sole source aquifer, unless an impermeable situation exists below the unit.*

Section 39.2(a)(ix) of the Act

- *If the facility will be located within a regulated recharge area, any applicable requirements specified by the Board for such areas have been met.*

Compliance with the Standard

On March 11, 2015 the U.S. Environmental Protection Agency (USEPA) designated a portion of the Mahomet Aquifer system as a sole source aquifer. The Mahomet Aquifer is the only sole source aquifer located in Illinois and is more than 100 miles south of the proposed expanded landfill. Please refer to **Appendix F.8** for a map of all USEPA Region 5 sole-source aquifers. Illinois is an USEPA Region 5 State.

The only regulated recharge area in Illinois is the Pleasant Valley Public Water District of Peoria County (PVPWD), as identified in 35 Ill. Admin. Code Part 617 and by correspondence received by the Illinois EPA, dated October 30, 2014. The PVPWD regulated recharge area is located approximately 150 miles southwest of the proposed Expansion. Thus, the facility is not located within a regulated recharge area.

References

1. Region 5 Sole Source Aquifer Map, **Appendix F.8**
2. 35 Ill. Admin. Code Part 617

Roads and Highways

Regulatory Requirements

IAC Title 35 Sections 811.302(c)

- *A facility (landfill or waste disposal site) that is located within 500 feet of a township or county road or state or interstate highway shall have its operations screened from view by a barrier no less than 8 feet in height.*

Compliance with the Standard

The proposed Expansion is sheltered from view by screening berms and fencing located along the facility boundary on all sides. Screening berms and fencing are a minimum 8-feet in height, as shown in the **Design Drawings**.

References

1. **Design Drawings**

Occupied Dwellings, Schools, Retirement Homes, Hospitals, Etc.

Regulatory Requirements

IAC Title 35 Sections 811.302(d)

- *No part of a unit shall be located closer than 152 meters (500 feet) from an occupied dwelling, school, or hospital that was occupied on the date when the operator first applied for a permit to develop the unit or the Facility containing the unit, unless the owner of such dwelling, school, or hospital provides permission to the operator, in writing, for a closer distance.*

Compliance with the Standard

There are no occupied dwellings, schools, retirement homes, hospitals, or similar institutions within 500 feet of the existing Landfill and proposed Expansion waste boundaries.

References

1. **Drawing No. D2**

SECTION 2.2

Hydrogeologic Investigation

2.2 HYDROGEOLOGIC INVESTIGATION

Introduction

On behalf of Zion Landfill, Inc., Aptim Environmental & Infrastructure, LLC (APTIM) performed a hydrogeologic investigation for the proposed Zion Landfill Site 2 North Expansion (Site 2 North Expansion) in order to supplement information previously collected for characterization and permitting of the existing facility. Ultimately, the collected information was evaluated to determine the suitability of the site for development of a landfill expansion.

The design of the Site 2 North Expansion is supplemented by existing geologic features to provide a high level of environmental safety. The naturally present clay beneath the site will work in conjunction with the engineered features of the expansion to protect groundwater resources in the vicinity of the site. This investigation, along with previous investigation activities, demonstrates that the proposed Site 2 North Expansion is located and designed so as to protect the public health, safety, and welfare. Key findings include the following:

- A significant amount of hydrogeologic investigation activities have been conducted at the existing landfill prior to the most recent investigation. Data collected during the previous hydrogeologic investigation activities was obtained through the advancement of over 260 borings (over 110 of which were continuously sampled) and the installation of over 200 monitoring wells.

- The most recent site investigation included a review of previous site investigations and the advancement of an additional 15 borings. The geology beneath the Site 2 North Expansion area, as characterized by the 15 borings, is consistent with the geology encountered beneath the existing Facility and the geologic setting which is described in regional publications[1], providing additional support to the findings of this investigation. The continuity observed from boring to boring demonstrates that the investigation activities were adequate in extent to verify the geologic and hydrogeolgic features beneath the site. Fourteen of the 15 borings were converted to piezometers and 9 additional nested piezometers were installed to supplement the hydrogeologic information for the site.

- A low-permeability cohesive soil (Wadsworth Formation) is present across the proposed site which will separate the footprint of the proposed Site 2 North Expansion from the uppermost aquifer. This low permeability cohesive soil (clayey till) has an average thickness of approximately 83.5 feet in the expansion area with maximum and minimum thicknesses of 95.0 feet and 71.9 feet, respectively. Field and laboratory test results and field observations indicated that this soil will effectively restrict vertical and horizontal movement of groundwater and will serve as an additional environmental safeguard at the proposed expansion. The average thickness of the Wadsworth Formation includes discontinuous lenses of silt, sand, and gravel (Intra-Till Sediments) which are contained within the till.

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[1]Csallany and Walton (1963), Frye and Willman (1975), Hansel and Johnson (1996), Horberg (1950), Johnson, et al. (1985), Kammerer, et al. (1998), Larsen (1973), Leetaru et al. (2003), Piskin, et al. (1975), Thwaites (1927), Visocky, et al. (1985), Willman, et al. (1975), and Willman (1971).

- As discussed in the design report, the engineered liner system beneath the expansion area will include 5 feet of recompacted clay and a high-density polyethylene (HDPE) liner. Such a liner exceeds the requirements of the U.S. EPA and has been accepted by the Illinois Environmental Protection Agency (IEPA) and other experts in the landfill field as providing a high level of environmental safety. Although the 5 feet of recompacted clay exceeds requirements and is not necessary in the construction of the liner, it has been included for continuity with the 5 feet of recompacted clay utilized in historical expansions of the original landfill. The natural clay that is present on the site below the liner system will act as a second, natural liner system for the landfill expansion.

- The investigation report was created in general accordance with the requirements contained in 35 Ill. Admin. Code, Section 811.315, 812.314, and 812.315. These regulations specify the necessary content of a hydrogeologic investigation report submitted to the IEPA as part of an application for a landfill expansion permit.

- The proposed Site 2 North Expansion is located in an area that is classified by Berg and Kempton (1984) as Map Unit E (low aquifer sensitivity with respect to land burial of municipal solid waste) with uniform, relatively impermeable silty or clayey till at least 50 feet thick. The site is also located in an area that has been classified by Larson (1973) as being geologically optimal for the development of a landfill within Lake County.

- Based on discussions with the site operator and the CQA Engineer the geologic interpretations that have been established within this report are consistent with the conditions observed during the development of large-scale excavations at the existing facility. The site-specific observations verify the thickness of the clayey till and discontinuous nature of the intra-till sediments as described within this analysis. IEPA review and approval of construction documentation reports supports this as well.

- The hydrogeologic conditions at the site will allow a comprehensive groundwater monitoring system to be implemented which will be able to adequately verify groundwater resources are protected.

Objectives of the Investigation

The most recent hydrogeologic investigation was conducted from November 2018 through February 2019. The objectives of the hydrogeologic investigation and subsequent Hydrogeological Investigation Report were: 1) to meet the general requirements set forth in criterion ii from Sec. 39.2(a)(2) of the Illinois Environmental Protection Act, which requires that the facility be designed, located, and operated so that the public health, safety and welfare will be protected; 2) to meet the applicable requirements set forth in the City of Zion Pollution Control Facility Siting Ordinance, 3) to provide the geotechnical and hydrogeologic information necessary for facility design, and 4) to meet the requirements of 35 Ill. Admin. Code, Sections 811.315, 812.314, and 812.315.

Prior to mobilization to the site, a detailed literature survey of the available regional hydrogeologic information was performed in accordance with 35 Ill. Admin. Code Section 811.315(c) requirements. In addition to the regional hydrogeologic information, information collected at the site during previous hydrogeologic investigations was reviewed. The regional information and previous hydrogeologic investigations assisted in devising a field investigation plan and understanding site geology. This plan enabled an accurate determination of the stratigraphic, physical, and hydrogeologic properties of the geologic materials beneath the proposed site.

Once the available regional and site hydrogeological information was studied and a field investigation plan was established, the field investigation was initiated. The field investigation included the advancement of 15 new borings and the installation of 23 new piezometers within and near the proposed expanded horizontal waste boundary. One of the borings, B-08-18, was advanced through the Wadsworth Formation, Intra-Till Sediments (within the Wadsworth Formation), Shallow Drift (Uppermost Aquifer), Lower Till, Intra-Till Sediments (within the Lower Till), lacustrine deposits, Basal Drift, and 3.6 feet into bedrock. The depth to the top of bedrock was approximately 202.4 feet below ground surface. Additionally, an existing boring (B-6-07) located to the south of the proposed expanded horizontal waste boundary, was originally advanced through these materials to a depth of approximately 215.2 feet below ground surface to the top of bedrock, and then 4.8 feet into bedrock.

A detailed description and discussion of the most recent investigation is presented in the following sections along with the supporting data and information from previous investigations. Conclusions derived in this report revealed that the geologic and hydrogeologic conditions at the proposed site are suitable and favorable for development of the Site 2 North Expansion that will protect the public health, safety, and welfare.

Project Background

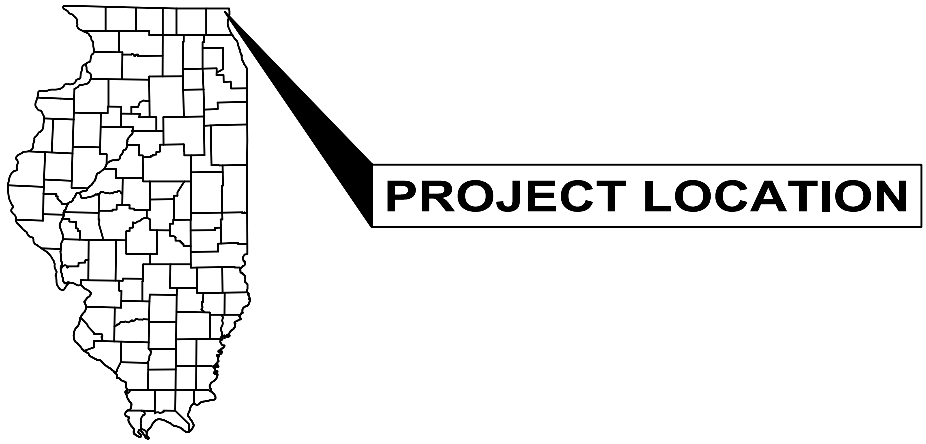
This section describes information related to the location and the physical setting of the proposed site and surrounding area.

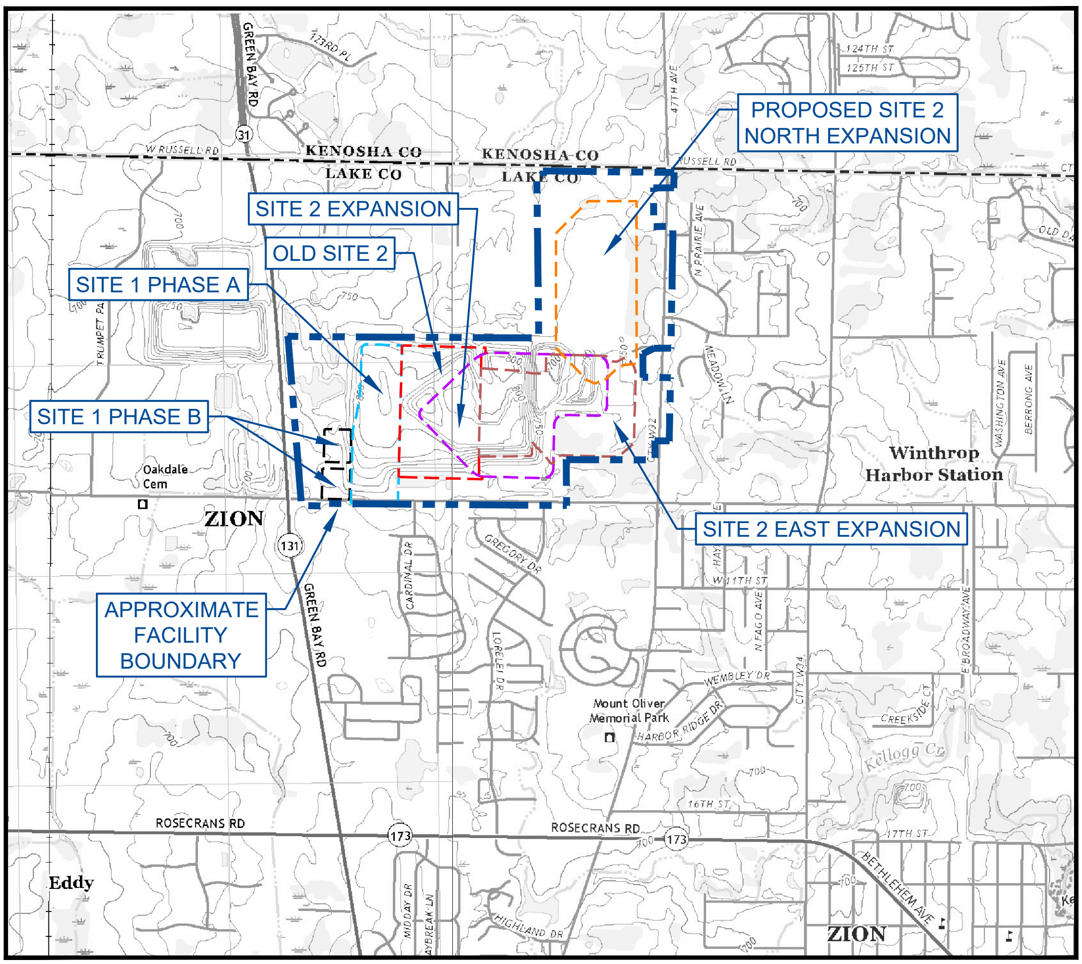
Proposed Location

The proposed Site 2 North Expansion is located within the City of Zion, Illinois in the southeast quarter of Section 6 of Township 46 North, Range 12 East of the 3rd Principal Meridian (Refer to **Figure 2.2-1**). **Figure 2.2-2** illustrates the location of the proposed waste expansion area in relation to the surrounding area on an aerial photograph. The proposed waste expansion area includes a component to the north of the existing and permitted Site 2 and Site 2 East areas. The proposed waste expansion area also includes a component which will expand vertically onto Site 2 and Site 2 East areas.

Site History

The Zion Landfill was initially permitted in 1976 and owned by BFI Waste Systems of North America, Inc. (BFI). BFI operated the site until July 30, 1999 when Allied Waste Industries, Inc. (Allied) acquired Browning-Ferris Industries, Inc., which was the parent company of BFI Waste Systems of North America, Inc. On March 31, 2000, Allied sold the site to Superior Zion Landfill, Inc. which concurrently changed its name to Onyx Zion Landfill, Inc. On July 1, 2006, Onyx Zion Landfill, Inc. changed its name to Veolia E.S. Zion Landfill, Inc. On December 26, 2012, Veolia E.S. Zion Landfill Inc. changed its name to Advanced Disposal Services Zion Landfill, Inc. On October 29, 2020, Advanced Disposal Services Zion Landfill, Inc. changed its name to Zion Landfill, Inc. The Zion Landfill, Inc. facility consists of several older units that have ceased acceptance of waste and are closed, as well as the currently active unit.





0 3000'

GRAPHIC SCALE

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FIGURE ADAPTED FROM USGS TOPOGRAPHIC QUADRANGLES FOR ZION (2018) AND WADSWORTH (2018).

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ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-1
REGIONAL SITE LOCATION MAP

APPROVED BY: TFA	PROJ. NO.: 631020105	DATE: MAY 2022

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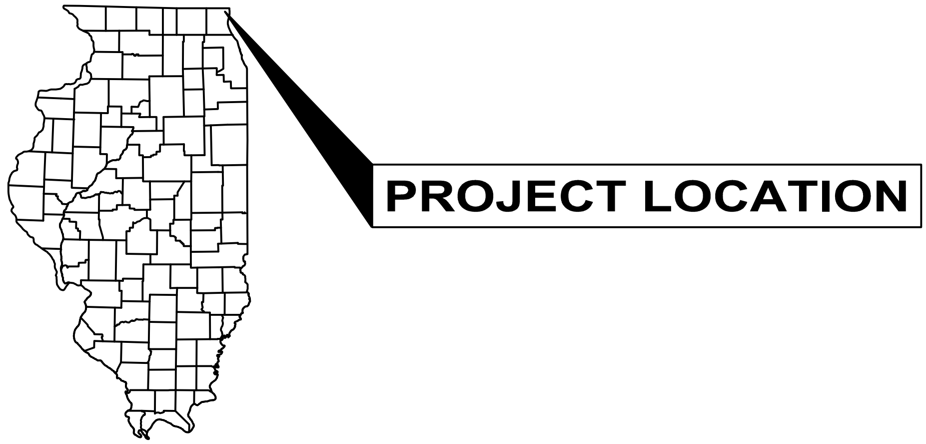

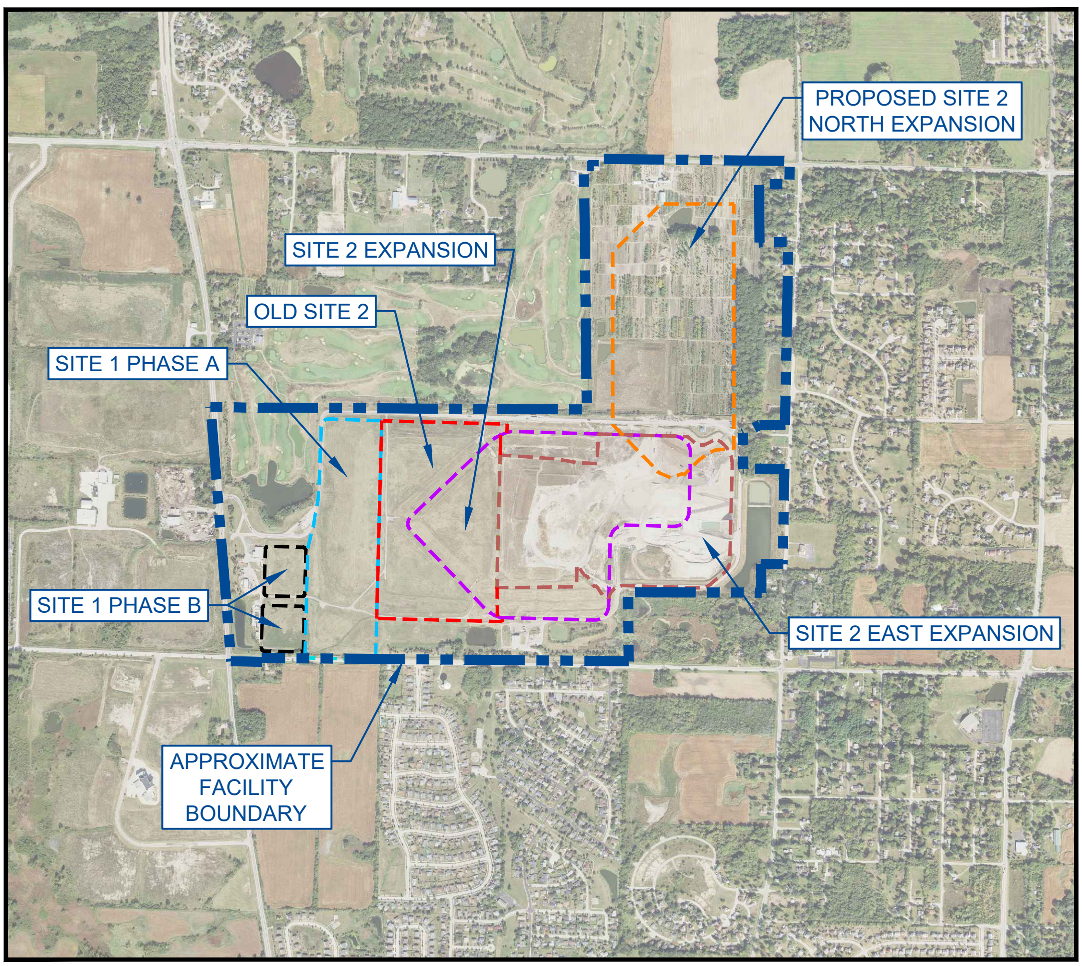

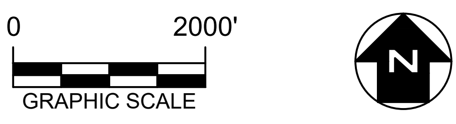

AERIAL IMAGERY OBTAINED FROM USGS NAIP
DIGITAL ORTHO PHOTO IMAGE DATABASE.

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ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-2
SITE LOCATION MAP

APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

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The portion of the facility referred to herein as the Zion Landfill consists of 3 areas individually referred to as Old Site 2, Site 2, and Site 2 East. Old Site 2 is a non-hazardous solid waste unit that was regulated under 35 IAC, Part 807. Old Site 2 commenced landfilling operations on December 23, 1981, pursuant to IEPA permit No. 1980-24-DE. In 1993, a final cover system was constructed over the site. Siting approval for Site 2 (initially identified as Site 3 at that time) was granted by the Zion City Council on April 17, 1995 which approved a new landfill unit to the east of Old Site 2 including a "piggyback" onto the eastern portion of Old Site 2. Site 2 was originally permitted under 35 IAC, Part 812, Subparts A and C, and is now regulated, along with Site 2 East, under 35 IAC, Part 811 regulations, which meet or exceed Subtitle D[2] Federal landfill regulations. . Collectively, Old Site 2, Site 2, and Site 2 East are referred to as the Zion Landfill. This application proposes to expand horizontally to the north of the currently permitted landfill and vertically onto the previously permitted Site 2 and Site 2 East fill areas**. Figures 2.2-1 and 2.2-2** illustrate the location of the various landfill units.

Climate Data

The city of Zion has a continental climate typical of northeastern Illinois. Annual normal precipitation averages 36.1 inches, more than half of which normally falls during the growing season from May through September. The average yearly temperature is 47.9 degrees Fahrenheit with average normal minimum and maximum temperatures of 40.4 and 55.4 degrees Fahrenheit, respectively. Average climatic data (obtained from the National Climatic Data Center (NCDC) in Asheville, North Carolina) was recorded between 1981 and 2010 in Kenosha, WI, located approximately 4 miles north of the proposed expansion. This data has been summarized in **Table 2.2-1**. **Appendix G** contains the data obtained from the NCDC.

TABLE 2.2-1
AVERAGE MONTHLY TEMPERATURE EXTREMES AND PRECIPITATION FOR KENOSHA, WI

(1981-2010)

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Ann
Normal Max Temp (^{0}F)	30.4	33.7	42.8	52.8	62.7	73.3	79.3	78.1	71.0	59.7	47.1	34.3	55.4
Normal Min Temp (^{0}F)	16.4	20.2	28.2	37.5	46.3	56.6	63.5	63.4	55.4	43.9	32.8	20.7	40.4
Average Temp (^{0}F)	23.4	27.0	35.5	45.1	54.5	64.9	71.4	70.7	63.2	51.8	40.0	27.5	47.9
Normal Precip. (in)	1.76	1.39	2.57	3.77	3.94	3.63	3.63	4.05	3.47	2.99	2.82	2.12	36.1

Regional Geology and Hydrogeology

The regional geology and hydrogeology were interpreted prior to performing any additional site specific investigation. The regional hydrogeologic investigation provided a better understanding of the regional geologic conditions, including unconsolidated deposits, bedrock, and groundwater characteristics. These sections describe the methods used to review the available regional geologic and hydrogeologic information and provide a detailed description of the results of the review.

Methodology

Existing published information on the area was obtained from several general sources. The first source was the available water well logs obtained from the Illinois State Geological Survey (ISGS) and Illinois State Water Survey (ISWS) in Champaign, Illinois, and the Wisconsin Geological and Natural History Survey (WGNHS) in Madison, Wisconsin. Additionally, the Lake County Health Department (LCHD) was contacted. The second source consisted of statewide and regional reports and maps available from the United States Geological Survey (USGS), ISGS, ISWS, and Federal Emergency Management Agency (FEMA). These publications were utilized in the development of the regional hydrogeologic investigation and creation of this report. The publications used to prepare this section are provided in the Hydrogeologic References at the end of this report.

Water Wells

The water well logs on file with the ISGS, ISWS, WGNHS, and LCHD were requested for all wells located within approximately 1 mile of the proposed Facility boundary. The LCHD indicated that all records that they receive are forwarded to the ISWS and, therefore, did not provide logs.

A total of 334 well logs were obtained from the various state agencies. The locations of these wells were plotted on USGS 7.5 minute quadrangle maps based primarily on the location information provided on the Well Construction Report (County, Section, Township, Range, Quarter, and quarter of the previous Quarter). In addition to the 334 well logs which were obtained from the state agencies, 220 probable well locations for which no log was available were identified in the field within 1 mile of the facility. Probable well locations were identified at residences not served by public water and not having a corresponding well construction report.

Although they can sometimes provide useful information, it should be noted that historical well records are known to include old data, lack detail, or in some cases include inaccurate information. Due to this lack of precision, it is not always possible to accurately determine the exact location of wells or the formations in which all of the wells are screened. The water well data was obtained to assist with the understanding of regional geology and hydrogeology, and is not meant to supersede the extensive geological and hydrogeological data collected on the site.

A road-level field reconnaissance was performed to verify the approximate location of wells identified from well logs, and to determine whether there are additional wells for which well logs do not exist. Using the USGS 7.5 minute quadrangle maps discussed above, all roads that are easily accessible were driven to field verify the location of the plotted wells. Verifying the location meant driving by the property and seeing a house, a grove of trees where a house may have stood, a barn, a concrete pad, a distant irrigation well, or other signs that indicate that a well currently exists or may have once been located at each respective location. Because the intended purpose of the well log data was to assist with characterization of regional geology and hydrogeology, it was not judged necessary to gain access to each property and visually observe a well. The information contained on these logs is consistent with the geology and hydrogeology described in regional publications and confirms the site geology and hydrogeology encountered during the site specific hydrogeologic investigation detailed within this report.

No wells were identified to exist within 200 feet of the waste boundary of the proposed Site 2 North Expansion. If any wells are subsequently found to exist within 200 feet of the waste boundary during construction of the proposed landfill, they will be properly abandoned in accordance with all applicable IEPA and Illinois Department of Public Health (IDPH) regulations prior to the start of operations in those areas.

The locations of the water wells are illustrated in **Drawing No. G2**. Copies of the well logs and a summary table are provided in **Appendix G**.

Physiography and Relief

The proposed Site 2 North Expansion is located within the physiographic division known as the Wheaton Morainal Country of the Great Lakes Section of the Central Lowland Province as seen in **Figure 2.2-3**. The Wheaton Morainal Country is characterized by glacial morainic topography which includes a series of broad parallel morainic ridges which encircle Lake Michigan. Due to the morainic topography, relief in the vicinity of the site is highly variable. The total relief in Lake County is approximately 377 feet, with the high elevation being approximately 957 feet above MSL in the northwest corner of the County on Gander Mountain, and the low elevation being less than 580 feet above MSL on the Lake Michigan shore near Waukegan.

Surficial Soils

Figure 2.2-4 illustrates the locations of various soil associations in the vicinity of the proposed expansion. As illustrated in **Figure 2.2-4**, the major soil type at the site is the Ashkum Silty Clay Loam which consists of colluvium and underlying till and is poorly drained (Calsyn, 2003). Other significant soil types at the site include the Beecher, Ozaukee, and Wauconda Silt Loams.

Regional Bedrock Stratigraphy

The regional bedrock consists of a succession of sedimentary rocks over 2,000 feet thick overlying Pre-Cambrian basement rock. A generalization of the stratigraphic column for the northeastern Lake County region is provided in **Figure 2.2-5**. The limited samples of crystalline basement rock in Illinois consist predominantly of granite with a few other granitic type rocks. The Mt. Simon Formation is a Cambrian age sandstone which unconformably overlies the crystalline bedrock surface. The unconformity that separates the crystalline basement rock from sedimentary rock represents more than 500 million years. The Mt. Simon Formation is generally described as consisting of fine to predominantly coarse grained, more angular, predominantly white, friable sandstone which also may be reddish, yellowish, or light greenish gray (Willman et al., 1975).

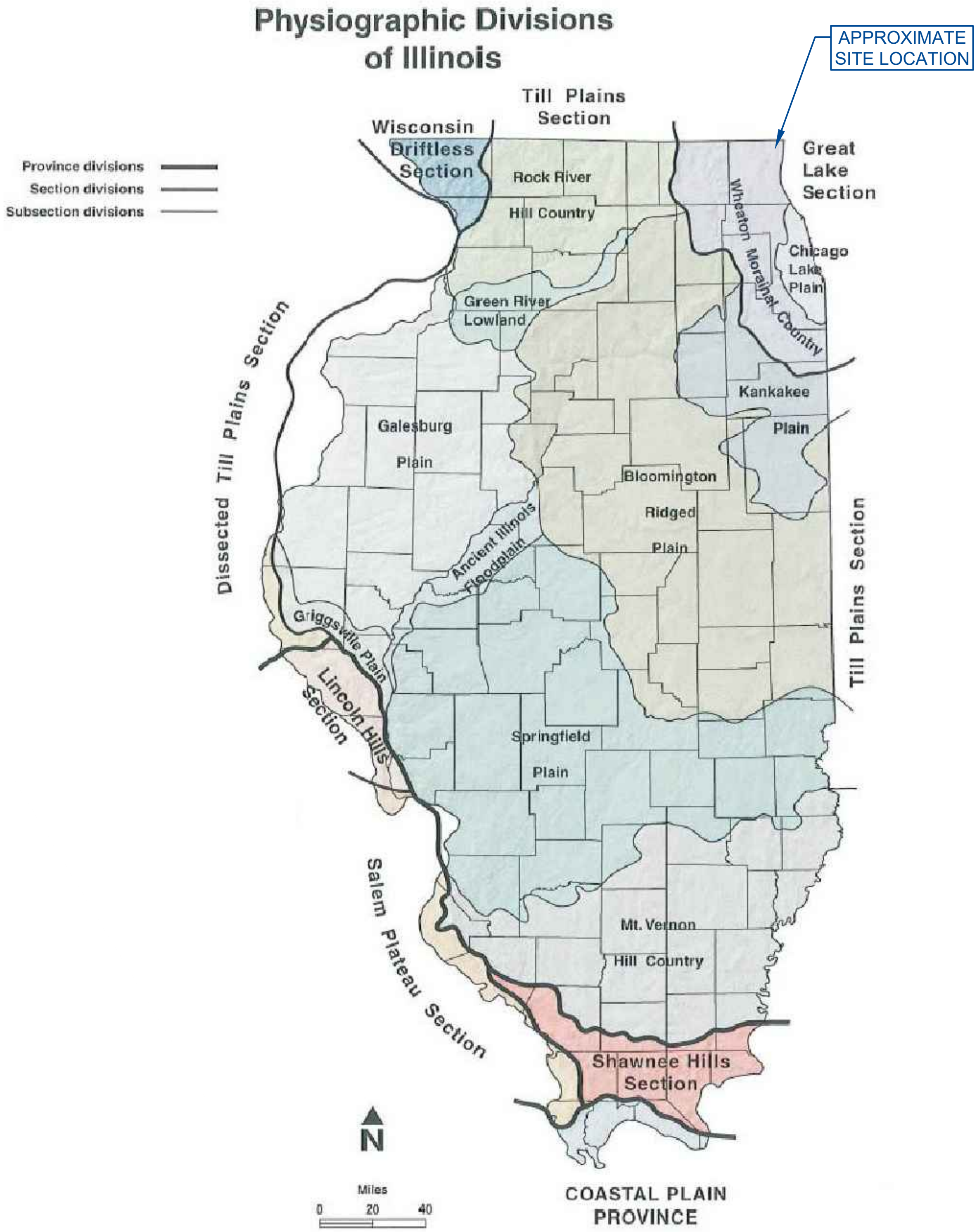

FIGURE OBTAINED FROM ISGS WEBSITE.

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ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-3
PHYSIOGRAPHIC DIVISION OF ILLINOIS

APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

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LEGEND

APPROXIMATE FACILITY BOUNDARY

APPROXIMATE EXPANSION WASTE BOUNDARY

NOTES

1. FIGURE ADAPTED FROM CUSTOM SOIL RESOURCE REPORTS FOR LAKE COUNTY, ILLINOIS.
2. SOILS DEPICTED ON THIS DRAWING HAVE FORMED WITHIN THE UPPER PORTION OF THE PEORIA SILT WHERE IT IS PRESENT. THE PEORIA SILT IS A RELATIVELY THIN LAYER OF SANDY OR CLAYEY SILT (AND SMALL AMOUNTS OF EOLIAN SAND) THAT WAS PREDOMINATELY DERIVED FROM GLACIAL MELTWATER AND HAS SINCE BEEN MODIFIED BY EROSIONAL AND SOIL FORMATION PROCESSES.

67A	HARPSTER SILTY CLAY LOAM, 0 TO 2 PERCENT SLOPES
153A	PELLA SILTY CLAY LOAM, 0 TO 2 PERCENT SLOPES
232A	ASHKUM SILTY CLAY LOAM, 0 TO 2 PERCENT SLOPES
298A	BEECHER SILT LOAM, 0 TO 2 PERCENT SLOPES
298B	BEECHER SILT LOAM, 2 TO 4 PERCENT SLOPES
530B	OZAUKEE SILT LOAM, 2 TO 4 PERCENT SLOPES
530B2	OZAUKEE SILT LOAM, 2 TO 4 PERCENT SLOPES, ERODED
530C2	OZAUKEE SILT LOAM, 4 TO 6 PERCENT SLOPES, ERODED
531B	MARKHAM SILT LOAM, 2 TO 4 PERCENT SLOPES
697A	WAUCONDA SILT LOAM, 0 TO 2 PERCENT SLOPES
698A	GRAYS SILT LOAM, 0 TO 2 PERCENT SLOPES
979B	GRAYS AND MARKHAM SILT LOAM, 2 TO 4 PERCENT SLOPES
W	WATER
830	LANDFILLS

REV. NO.	DATE	DESCRIPTION

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ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGICAL INVESTIGATION

FIGURE 2.2-4

GENERAL SOIL MAP FOR THE VICINITY OF THE PROPOSED EXPANSION

DRAWN BY:	NV	APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

SYSTEM	SERIES AND MEGAGROUP	GROUP AND FORMATION	HYDROSTRATIGRAPHIC UNITS Aquigroup	HYDROSTRATIGRAPHIC UNITS Aquifer/aquitard	LOG	APPROX. THICKNESS (ft)	DESCRIPTION
Quaternary	Pleistocene	Undifferentiated	Prairie	Pleistocene		200+	Unconsolidated glacial deposits - pebbly clay (till) silt, and gravel. Loess (windblown silt), and alluvial silts, sands and gravels.
Silurian	Niagaran	Port Byron Fm Racine Fm Waukesha Ls Joliet Ls	Upper Bedrock	Silurian dolomite aquifer		200	Dolomite, silty at base, locally cherty.
	Alexandrian	Kankakee Ls Edgewood Ls					
Ordovician	Cincinnatian	Maquoketa Shale Group		Maquoketa confining unit		225	Shale, gray or brown; locally dolomite and/or limestone, argillaceous.
	Mohawkian (Ottawa Ls Megagroup)	Galena Group Decorah Subgroup Platteville Group		Galena-Platteville unit		300	Dolomite and/or limestone, cherty. Dolomite, shale partings, speckled. Dolomite and/or limestone, cherty, sandy at base.
		Ancell Gr: Glenwood Fm		Ancell aquifer		200-275	Sandstone, fine- and coarse-grained; little dolomite; shale at top.
	Chazyan	Ancell Gr: St. Peter Ss					Sandstone, fine- to medium-grained; locally cherty red shale at base.
Cambrian	St. Croixian (Knox Megagroup)	Eminence Fm - Potosi Dolomite	Midwest Bedrock	Middle confining unit: Eminence-Potosi		70-95	Dolomite, white, fine-grained, geodic quartz, sandy at base.
		Franconia Fm		Middle confining unit: Franconia			Dolomite, sandstone, and shale, glauconitic, green to red, micaceous.
		Ironton Ss		Ironton-Galesville aquifer		110	Sandstone, fine- to medium-grained, well sorted, upper part dolomitic.
		Galesville Ss					
		Eau Claire Fm	Basal Bedrock	Eau Claire		410	Shale and siltstone; dolomite, glauconitic; sandstone, dolomitic, glauconitic.
		Mt. Simon Fm		Elmhurst-Mt.Simon aquifer		1,000-1,500	Sandstone, coarse grained, white, red in lower half; lenses of shale and siltstone, red micaceous
Pre-Cambrian			Crystalline				No aquifers in Illinois

THIS COLUMN HAS BEEN MODIFIED FROM VISOCKY ET AL., 1985. APPROXIMATE THICKNESSES ARE ESTIMATED AS DESCRIBED IN THE TEXT.

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FIGURE 2.2-5
GENERALIZED STRATIGRAPHIC COLUMN IN THE VICINITY OF THE PROPOSED EXPANSION

DRAWN BY: NV	APPROVED BY: TFA	PROJ. NO.: 631020105	DATE: MAY 2022

Regional publications report that in the vicinity of the site, the unit ranges in thickness from approximately 950 to 1,000 feet, with a surface elevation of approximately 975 feet below mean sea level. **Figure 2.2-6** illustrates the top elevation of the Mt. Simon Formation in Lake County.

The Cambrian and Ordovician age Knox Dolomite Megagroup unconformably overlies the Mt. Simon Formation. The Knox Dolomite Megagroup consists of the Cambrian Age Eau Claire Formation, Galesville Sandstone, Ironton Sandstone, Fraconia Formation, Potosi Dolomite, and the Eminence Formation, conformably. It should be noted that the Eau Claire Formation and Ironton-Galesville Sandstone have been attributed to the Knox Megagroup within this report based upon information obtained from Willman et al. (1975). The report published by Visocky, et al., (1985) does not include these units within the Knox Megagroup as shown on **Figure 2.2-5**.

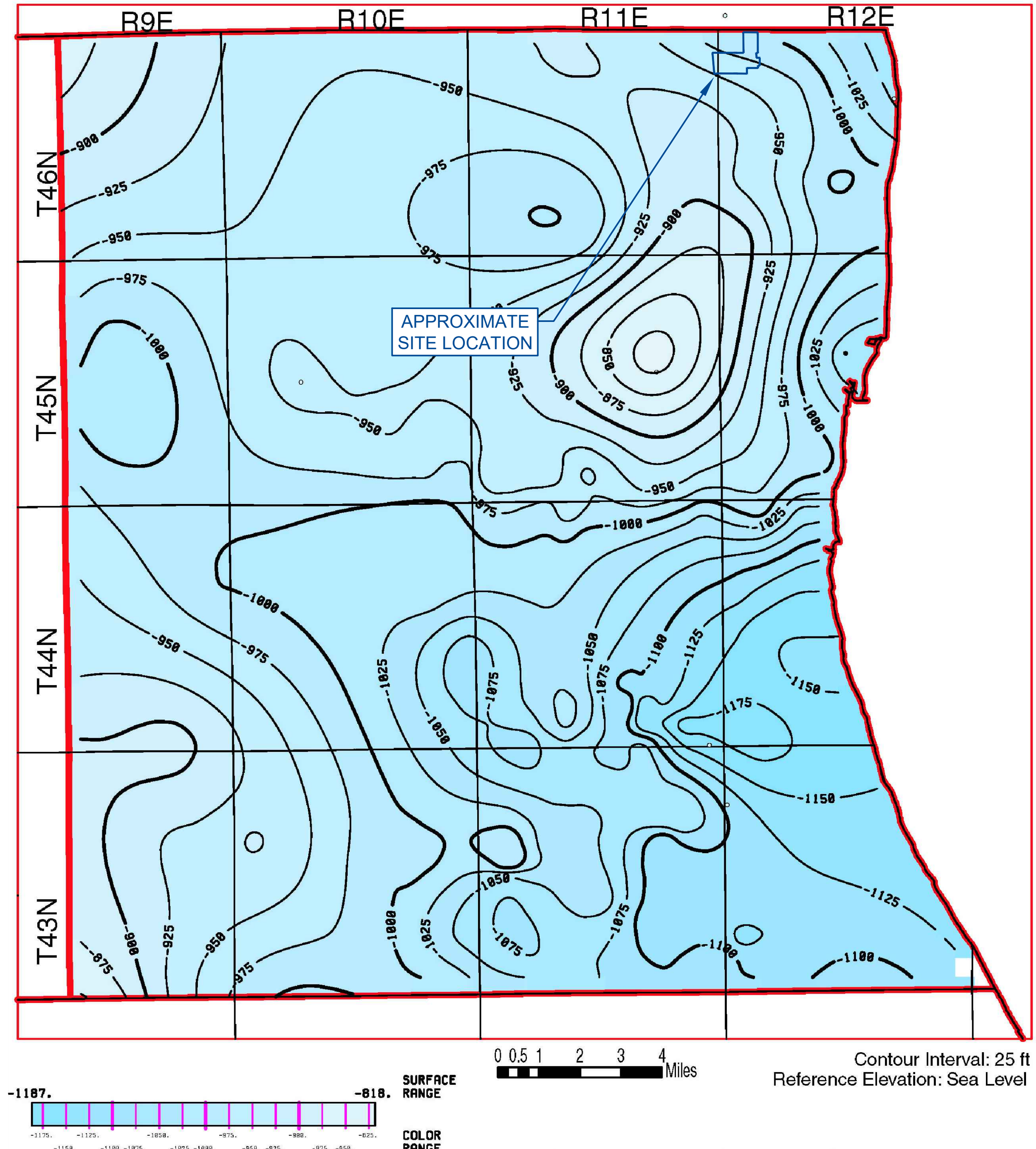
The Eau Claire Formation consists predominantly of gray dolomitic sandstone, which may include shaley siltstone and silty, sandy, glauconitic, brownish gray dolomite (Willman et al., 1975). Regional publications indicate that the Eau Claire Formation has an approximate thickness of 410 feet in the vicinity of the site and can be found at an approximate elevation of 575 feet below mean sea level (Willman et al., 1975 and Leetaru et al., 2003). The Eau Claire Formation is an aquitard, which acts as a confining unit to the underlying Mt. Simon Formation (Visocky, et al., 1985). **Figure 2.2-7** illustrates the top elevation and thickness of the Eau Claire Formation in Lake County.

The Ironton-Galesville Sandstone conformably overlies the Eau Claire Formation. The Galesville Sandstone is described as white to light buff, fine grained, moderately well-sorted sandstone. The Ironton Sandstone is described as a light pinkish-buff, medium grained, poorly sorted, dolomitic sandstone. Regional publications report the Ironton-Galesville Sandstone near the proposed site is approximately 110 feet thick and can be found at an approximate elevation of 475 feet below mean sea level (Leetaru et al., 2003). This sandstone is the most productive unit of the Midwest Bedrock Aquigroup with yields over 500 gallons per minute in northern Illinois (Visocky, et al., 1985). **Figure 2.2-8** illustrates the top elevation and thickness of the Ironton-Galesville Formation in Lake County.

The Franconia Formation in northern Illinois is described as gray fine-grained dolomitic sandstone. The lowermost part of the Franconia Formation (Davis Member) becomes increasingly shaley and the uppermost part grades to silty and sandy dolomite (Derby-Doerun Member). In the vicinity of the proposed expansion, the Franconia Formation has an approximate range in thickness of 50 to 75 feet (Willman, et al., 1975 and Anderson, 1919).

The Potosi Dolomite is brown to pinkish gray, fine crystalline dolomite that may be argillaceous and glauconitic. The Eminence Formation consists of light gray to brown or pink, sandy, fine to medium grained dolomite that contains oolitic chert and thin sandstone strata. In the vicinity of the proposed expansion, the combined thickness of the Eminence and Potosi Formations is approximately 20 feet (Willman et al., 1975).

The Ordovician age Ancell Group is separated from the underlying Knox Megagroup by a distinctive unconformity. The unconformity is classified as major, resulting in an irregular erosional surface and rubble zone at the base of the Ancell Group. Erosion which took place prior to deposition of the Ancell Group removed the entire Ordovician Prairie du Chien Group and Cambrian Jordan Sandstone and truncated the Potosi and Eminence Formations in the vicinity of the proposed expansion. The Ancell Group includes the St. Peter Sandstone and Glenwood Formation. In the vicinity of the proposed expansion, the St. Peter Sandstone is composed entirely of the Tonti Sandstone Member.



Top of the Mt. Simon Sandstone

FIGURE ADAPTED FROM ISGS OPEN FILE SERIES 2003-2012

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FIGURE 2.2-6
TOP ELEVATION OF MT. SIMON SANDSTONE

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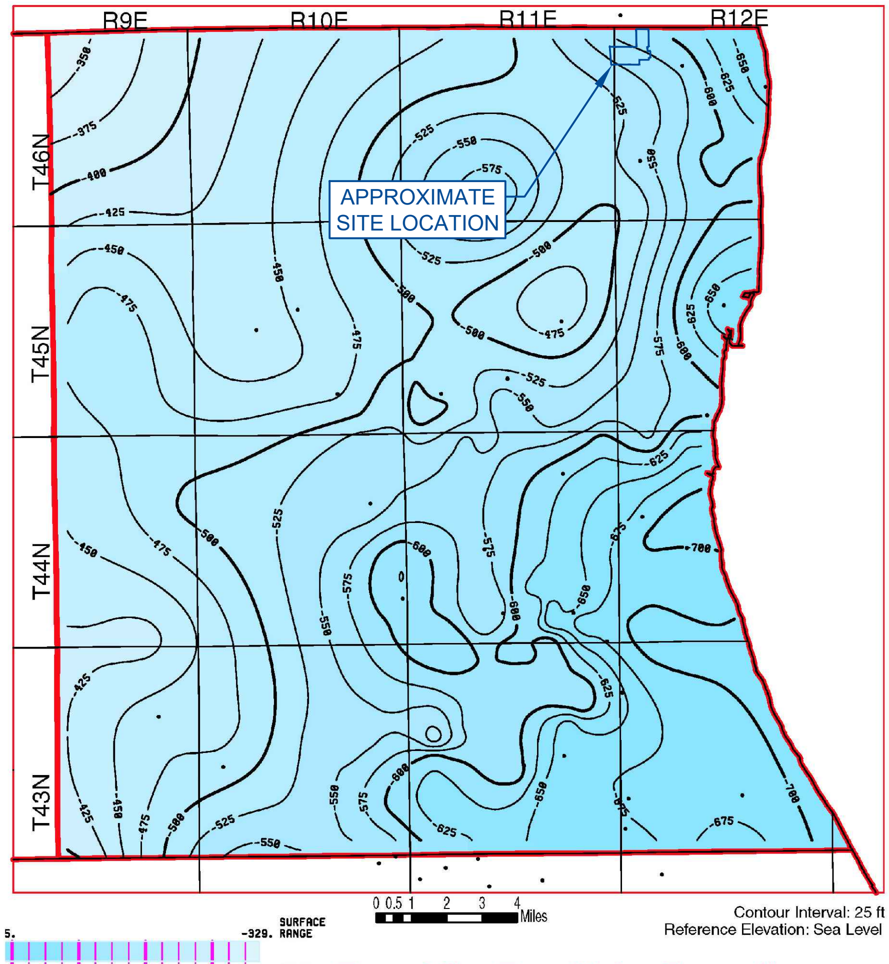
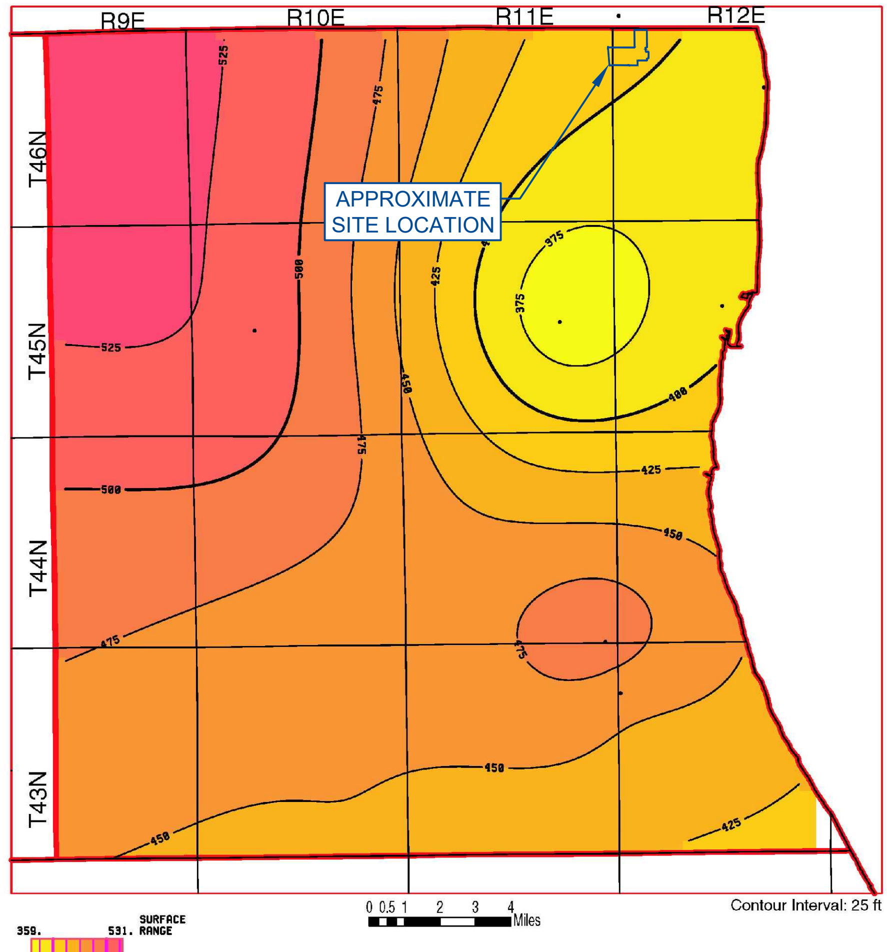


Top of the Eau Claire Formation



Thickness of the Eau Claire Formation

FIGURE ADAPTED FROM ISGS OPEN FILE SERIES 2003-2012

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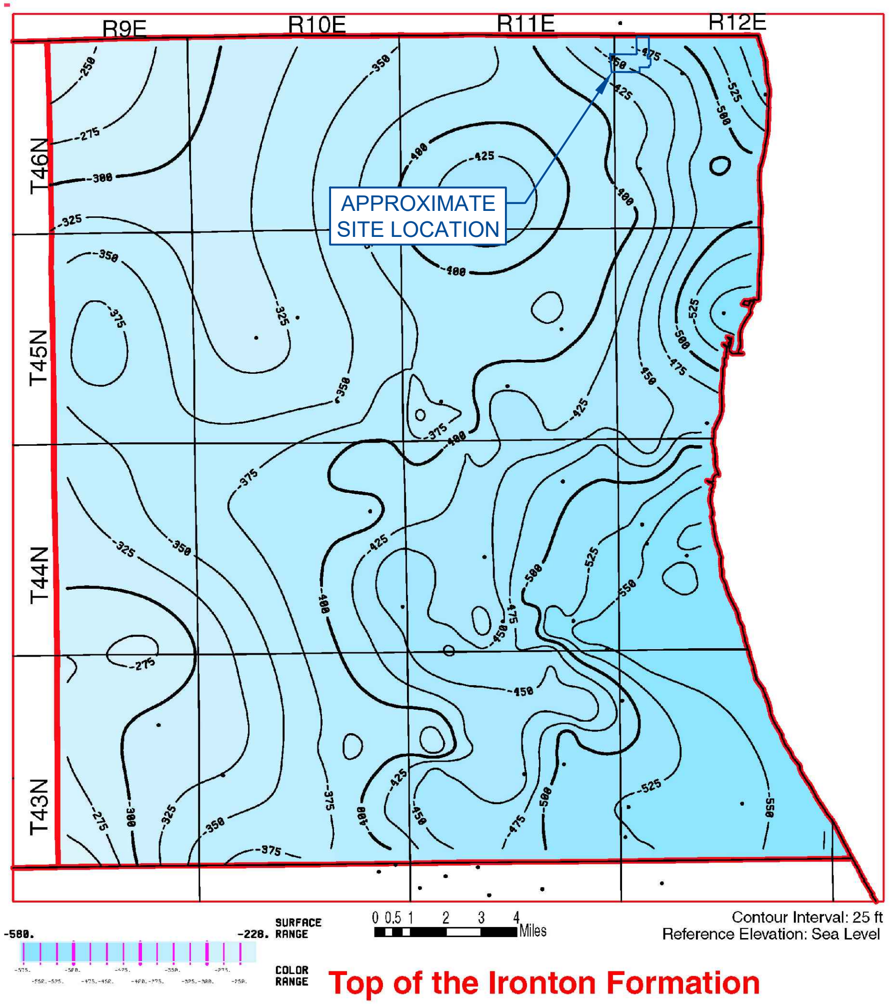
ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-7
ELEVATION AND THICKNESS OF THE EAU CLAIRE FORMATION IN LAKE COUNTY

DRAWN BY:	NV	APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022



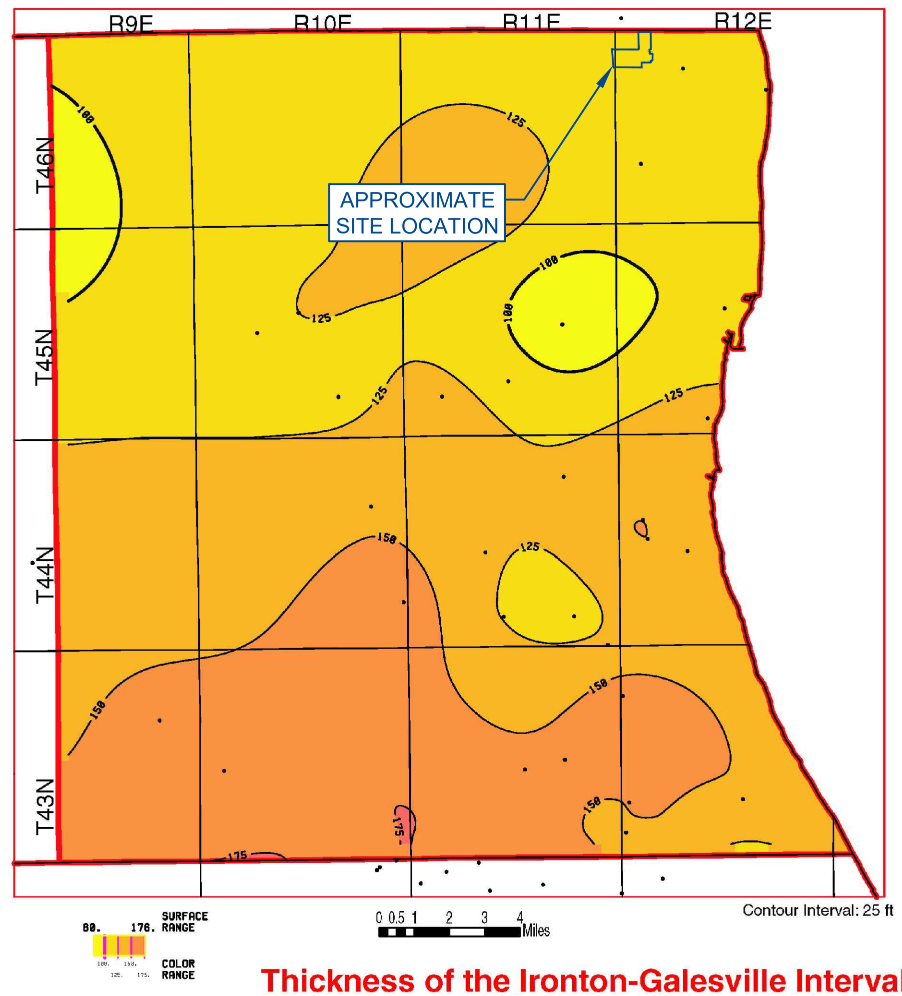


FIGURE ADAPTED FROM ISGS OPEN FILE SERIES 2003-2012

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ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-8
ELEVATION AND THICKNESS OF THE IRONTON-GALESVILLE FORMATION IN LAKE COUNTY

DRAWN BY:	NV	APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

The Tonti Sandstone Member is white, fine-grained, well sorted, friable, highly porous sandstone. Available literature indicates that the St. Peter Sandstone is friable and crumbles easily (Willman et al., 1975).

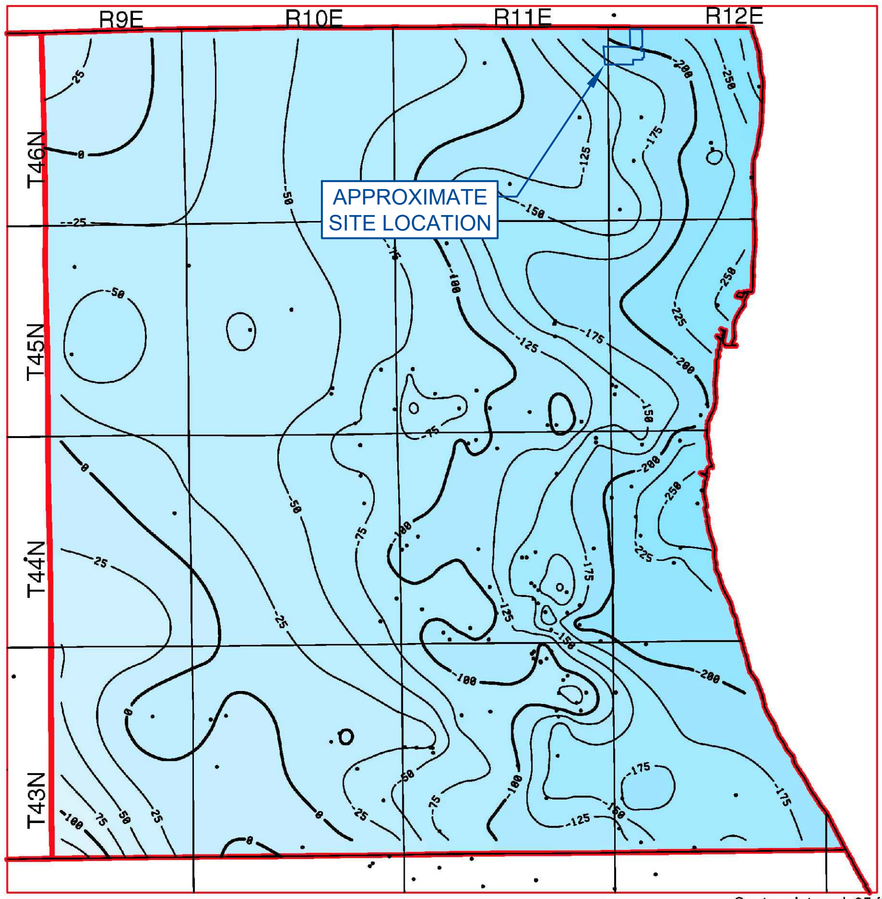
As the sandstone is generally poorly cemented, lithification of the sandstone is primarily the result of compaction by the weight of the overlying strata. The overlying Glenwood Formation is described as a highly varied unit of poorly sorted sandstone, impure dolomite, green shale, and dolomitic, fine to medium grained sandstone.

The thickness of the Ancell group, varies greatly, as the St. Peter Sandstone was deposited on an irregular erosional surface. Regional publications indicate that the thickness of the Ancell Group near the proposed site is approximately 200 to 275 feet (Willman et al., 1975, Anderson, 1919, and Leetaru et al., 2003). The top of the Ancell Group can be found approximately 200 feet below mean sea level in the vicinity of the proposed expansion (Leetaru et al., 2003). Where tapped to provide water supplies, the Ancell Group typically provides small to moderate quantities of potable water (Visocky, et al., 1985). **Figure 2.2-9** illustrates the top elevation and combined thickness of the Ancell, Franconia, Eminence and Potosi units in Lake County.

The Ordovician age deposits of the Galena and Platteville groups overlie the Ancell Group in the vicinity of the proposed expansion (**Figure 2.2-5**). The Galena and Platteville Groups, which are comprised of numerous dolomite and limestone formations of varying composition, are referred to as simply the Galena-Platteville Group or Galena-Platteville Dolomite within this report. In the vicinity of the proposed site, several formations of the Galena-Platteville Group are present. These include the Pecatonica, Mifflin, Grand Detour, Nachusa, Quimbys Mill, Spechts Ferry, Guttenberg, Dunleith, Wise Lake, and Dubuque Formations.

The Pecatonica Formation is the basal unit in the Galena-Platteville Group and is characterized by brown, relatively pure, cherty dolomite. The Mifflin Formation consists of gray very fine grained to lithographic limestone or fine grained dolomite in thin wavy beds separated by beds of shale. The Grand Detour Formation varies laterally from medium grained dolomite to lithographic limestone and vertically from pure to argillaceous and shaley. The Nachusa Formation is a light gray, medium-grained, vuggy dolomite or lithographic limestone. The Quimbys Mill Formation consists of medium to thin-bedded argillaceous to shaley limestone or dolomite. The Spechts Ferry Formation is dominantly shale, containing thin beds of limestone and bentonite. The Guttenberg Formation consists of thin-bedded fine grained limestone interbedded with brown-red shale. The Dunleith Formation is slightly argillaceous, thin to medium bedded mostly cherty dolomite with a gray to light brown appearance. The Wise Lake Formation is pure light brown vesicular to vuggy dolomite. The Dubuque Formation is a light brownish gray to buff, fine grained dolomite that is strongly argillaceous and is characterized by well defined, flat bedding (Willman and Kolata, 1978). The Galena-Platteville Dolomite has been found to be greater than 300 feet thick in the vicinity of the proposed expansion with a surface elevation of approximately 100 feet above mean sea level (Leetaru et al., 2003). **Figure 2.2-10** illustrates the top elevation of the Galena Group and the thickness of the combined interval of the Galena and Platteville Groups in Lake County.

The Galena-Platteville Dolomite is unconformably overlain by the Maquoketa Shale Group (Maquoketa Group). The Maquoketa Group consists of a lower unit comprised of predominantly shale (Scales Shale), overlain by a middle limestone (Fort Atkinson Limestone), and 2 upper shales (Brainard Shale and Neda Formation).



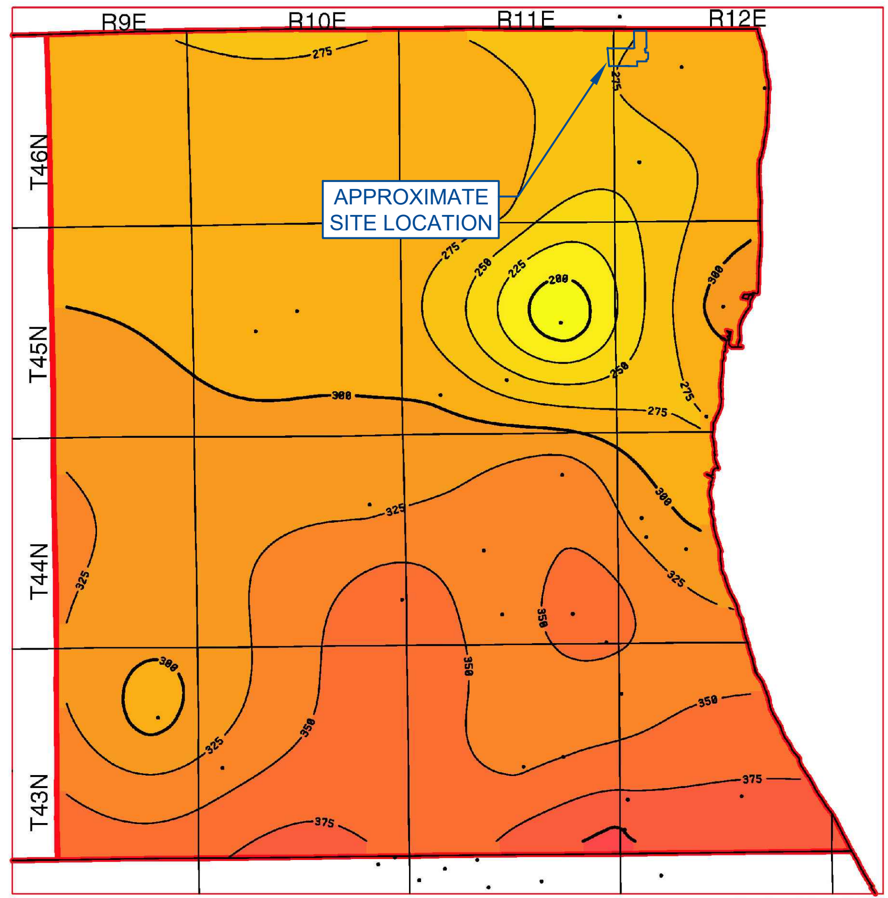
Contour Interval: 25 ft
Reference Elevation: Sea Level

0 0.5 1 2 3 4 Miles

-288. 144. SURFACE RANGE

COLOR RANGE

Top of the Ancell Formation



Contour Interval: 25 ft

0 0.5 1 2 3 4 Miles

186. 404. SURFACE RANGE

COLOR RANGE

Thickness of the Ancell to Ironton Formations

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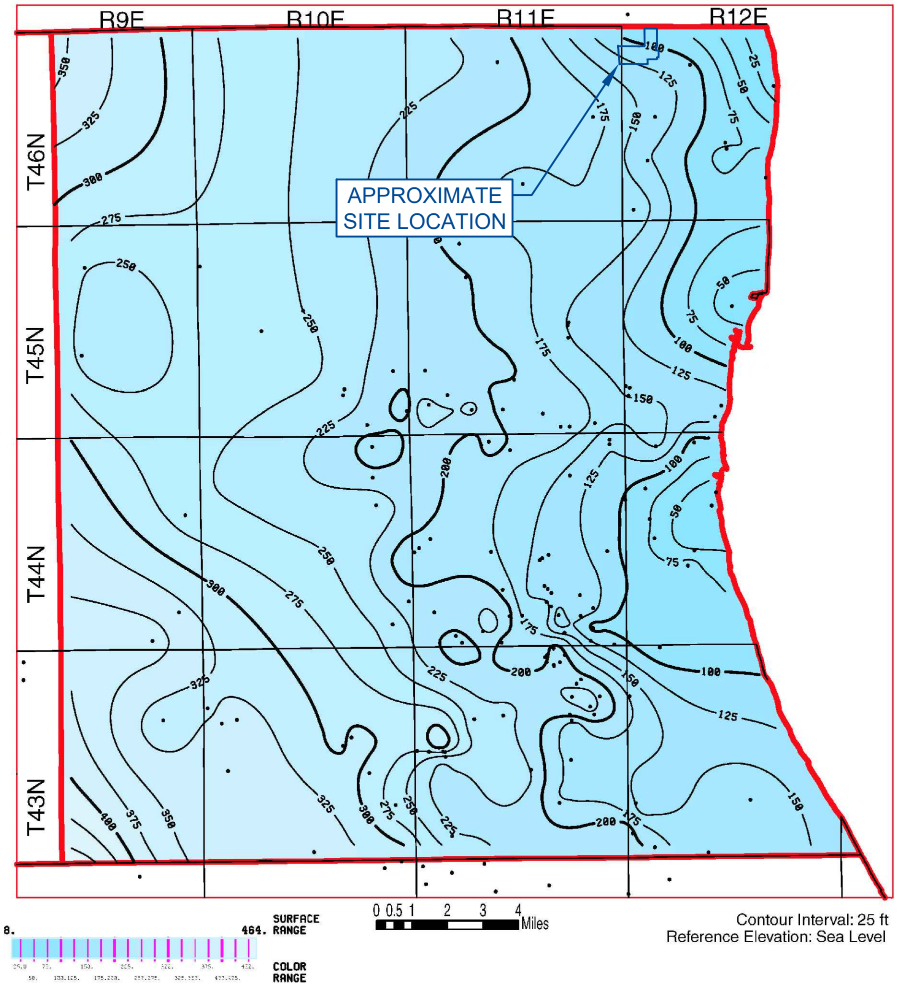
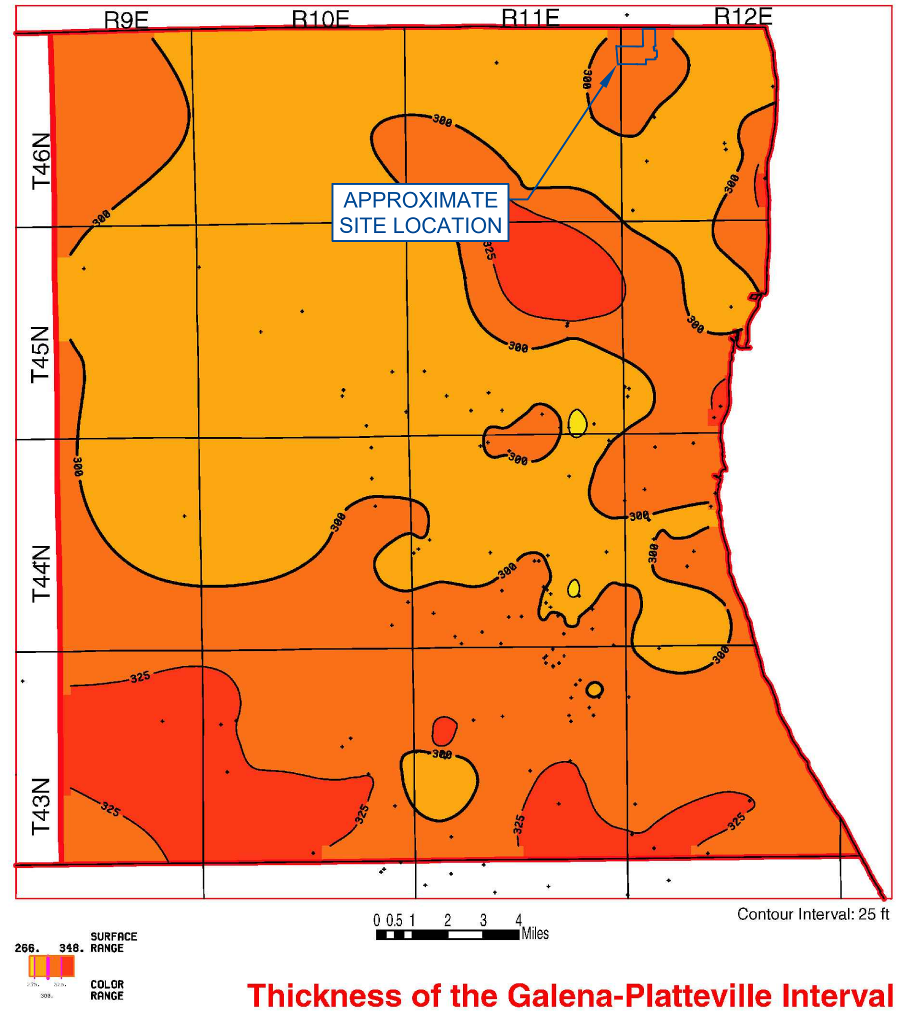
FIGURE 2.2-9
ELEVATION AND THICKNESS OF THE ANCELL, FRANCONIA, POTOSI, AND EMINENCE UNITS IN LAKE COUNTY

DRAWN BY: NV	APPROVED BY: TFA	PROJ. NO.: 631020105	DATE: MAY 2022



Top of the Galena Limestone



Thickness of the Galena-Platteville Interval

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FIGURE 2.2-10
ELEVATION AND THICKNESS OF THE GALENA-PLATTEVILLE DOLOMITE IN LAKE COUNTY

DRAWN BY:	NV	APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

The Maquoketa Group is approximately 225 feet thick in the vicinity of the proposed expansion and has a surface elevation of approximately 325 feet above mean sea level (Leetaru et al., 2003). **Figure 2.2-11** illustrates the top elevation and thickness of the Maquoketa Group in Lake County.

In Lake County, the Silurian dolomite lies unconformably above the Maquoketa Group and directly beneath the glacial drift. It has been subdivided into the Niagaran Series and the underlying Alexandrian Series. In the area of the site, the Silurian dolomite is approximately 200 feet thick with an approximate elevation of 525 feet above mean sea level (Leetaru et al., 2003). It is extremely argillaceous, silty, and cherty to exceptionally pure dolomite. The upper portion of the dolomite has numerous fractures, crevices, and solution cavities. **Figure 2.2-12** illustrates the top elevation and thickness of the Silurian dolomite in Lake County.

Regional Bedrock Topography

As is illustrated in **Figure 2.2-13**, the bedrock surface in northern Illinois is an undulating plane on which valleys have been incised by glacial and pre-glacial erosion. **Figure 2.2-13** reveals a system of valleys which have been carved deep into the bedrock surface. These bedrock valleys were carved by the fluvial processes of the ancient Illinois, Princeton, Rock, and Troy Rivers. **Figure 2.2-12** illustrates the bedrock surface in Lake County, Illinois. In the vicinity of the site, the bedrock surface is greater than 200 feet below ground surface at an approximate elevation of 530 feet above mean sea level.

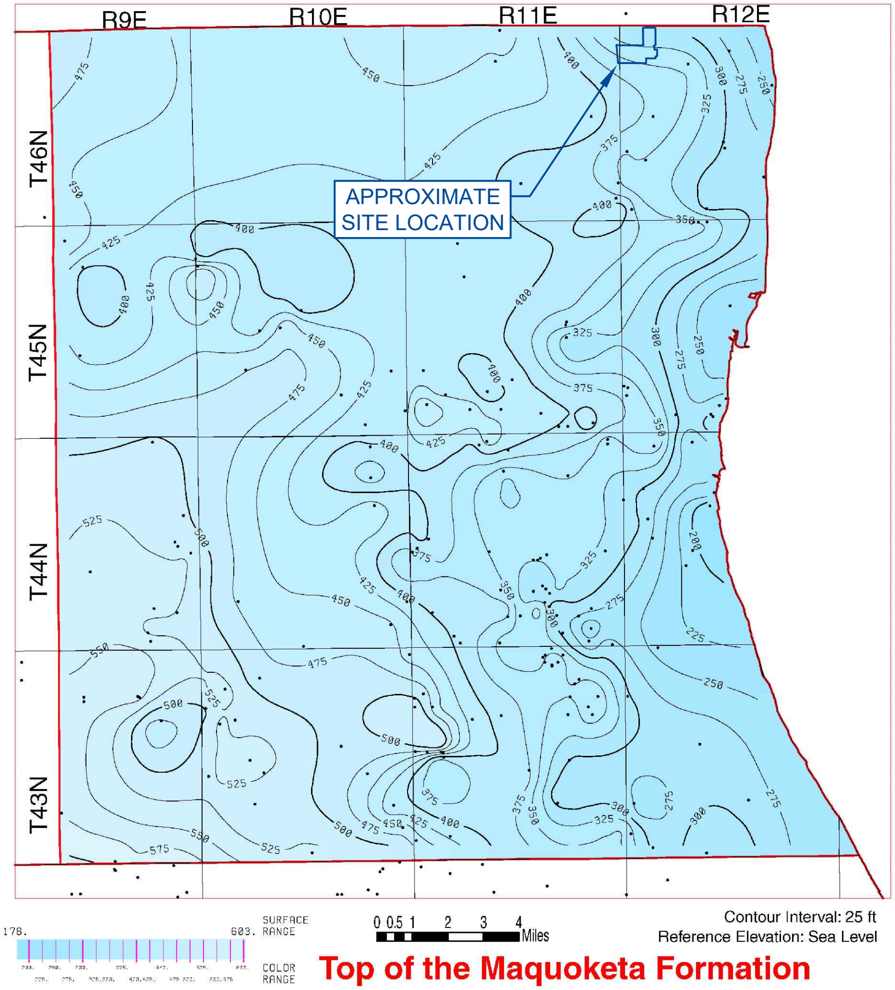
Regional Bedrock Structural Features

The predominant structural features which have influenced the regional bedrock in the northeastern Illinois region, in addition to glacial action and fluvial processes, are the Wisconsin and Kankakee Arches. The Wisconsin Arch is a broad positive area which separates the Michigan Basin on the east from the Forest City Basin on the West. To the southeast, this arch connects with the Kankakee Arch which runs between the Michigan and Illinois Basins (**Figure 2.2-14**).

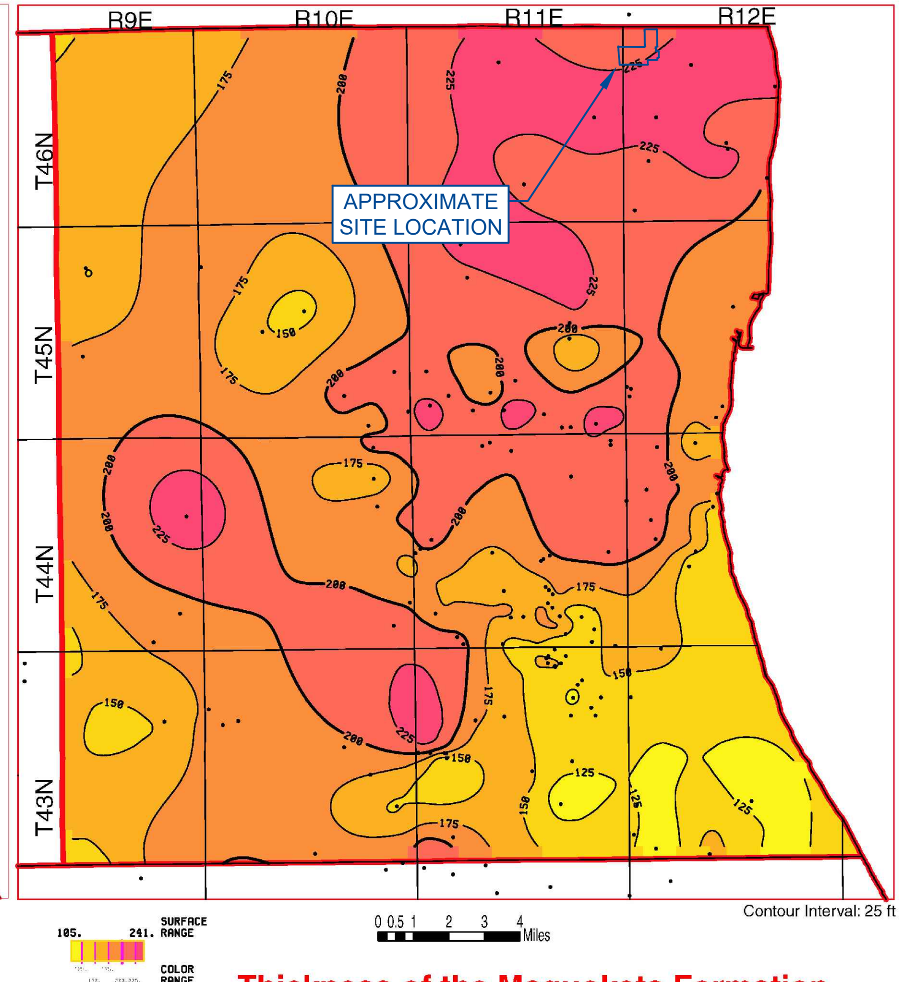
Seismic Risk

Earthquakes are formed when the stresses within the bedrock reach a point at which rupture or breakage of bedrock occurs. This breakage releases a significant amount of energy that is known as an earthquake. The proposed site is located in a low seismic impact zone. Over the last 200 years, the nearest area of major seismic activity is the New Madrid Seismic Zone along the Mississippi River Valley in southeastern Missouri and western Tennessee. In 1811 and 1812, earthquakes with magnitudes of VIII or greater on the Mercalli scale shook the Mississippi Valley. The zone is continuously active with hundreds of tremors recorded each year, however, most are too small to be felt. Away from the zone, epicenters are randomly scattered in a large area. The southern one third of Illinois falls in this area.

The Wabash Valley Fault Zone near southwestern Indiana has experienced structural movement during the post-late Pennsylvanian to pre-Pleistocene time (between 1.6 and 300 million years ago) and the Sandwich Fault zone in northern Illinois has demonstrated significant movement in upper Silurian aged rocks (Nelson, 1995). The youngest rock demonstrably displaced by the Sandwich Fault Zone are of Pennsylvanian age, which ended 286 million years ago.



Top of the Maquoketa Formation



Thickness of the Maquoketa Formation

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ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-11
ELEVATION AND THICKNESS OF THE MAQUOKETA FORMATION IN LAKE COUNTY

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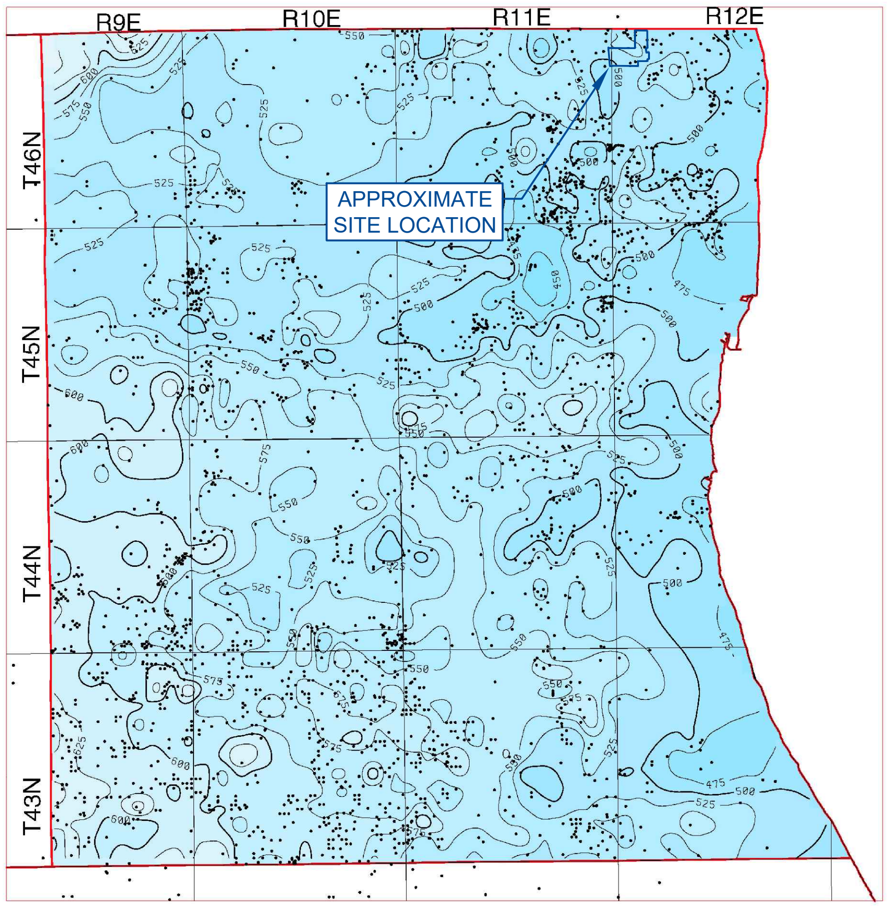
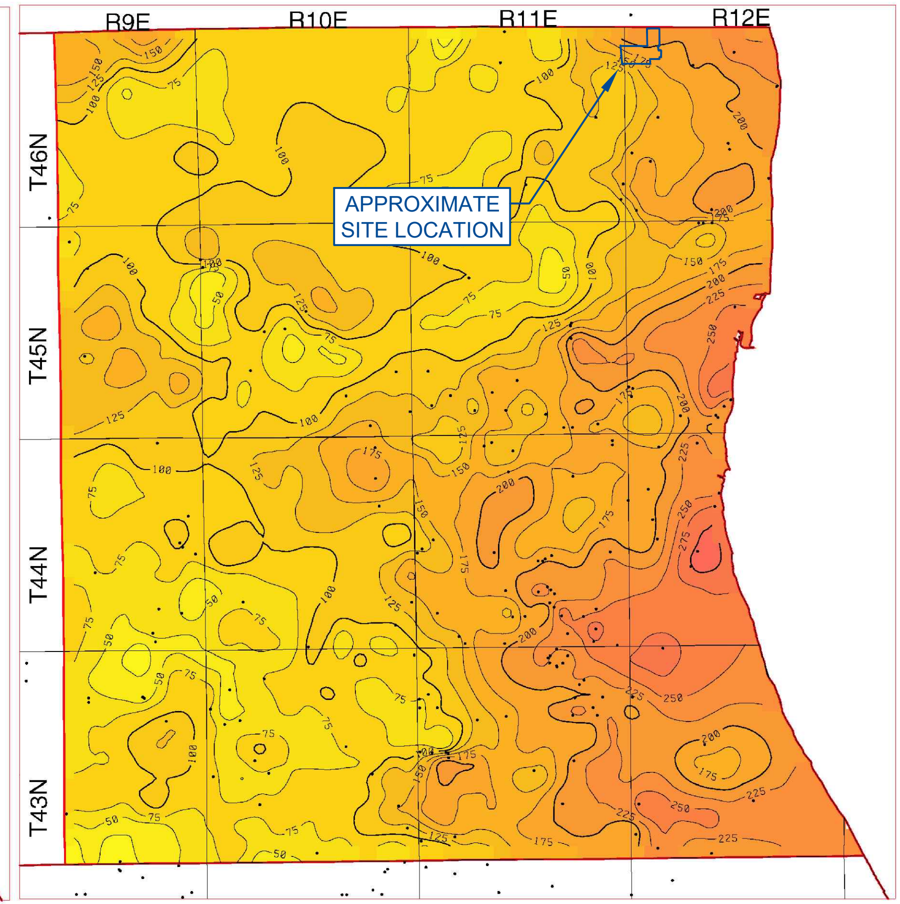


Bedrock Topography



Thickness of Silurian

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FIGURE 2.2-12
BEDROCK TOPOGRAPHY AND THICKNESS OF THE SILURIAN DOLOMITE IN LAKE COUNTY

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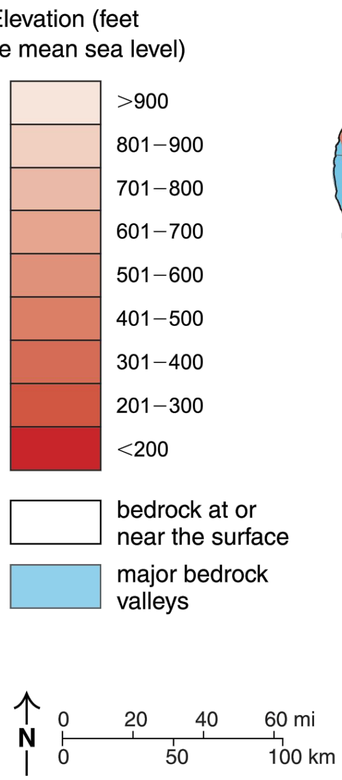
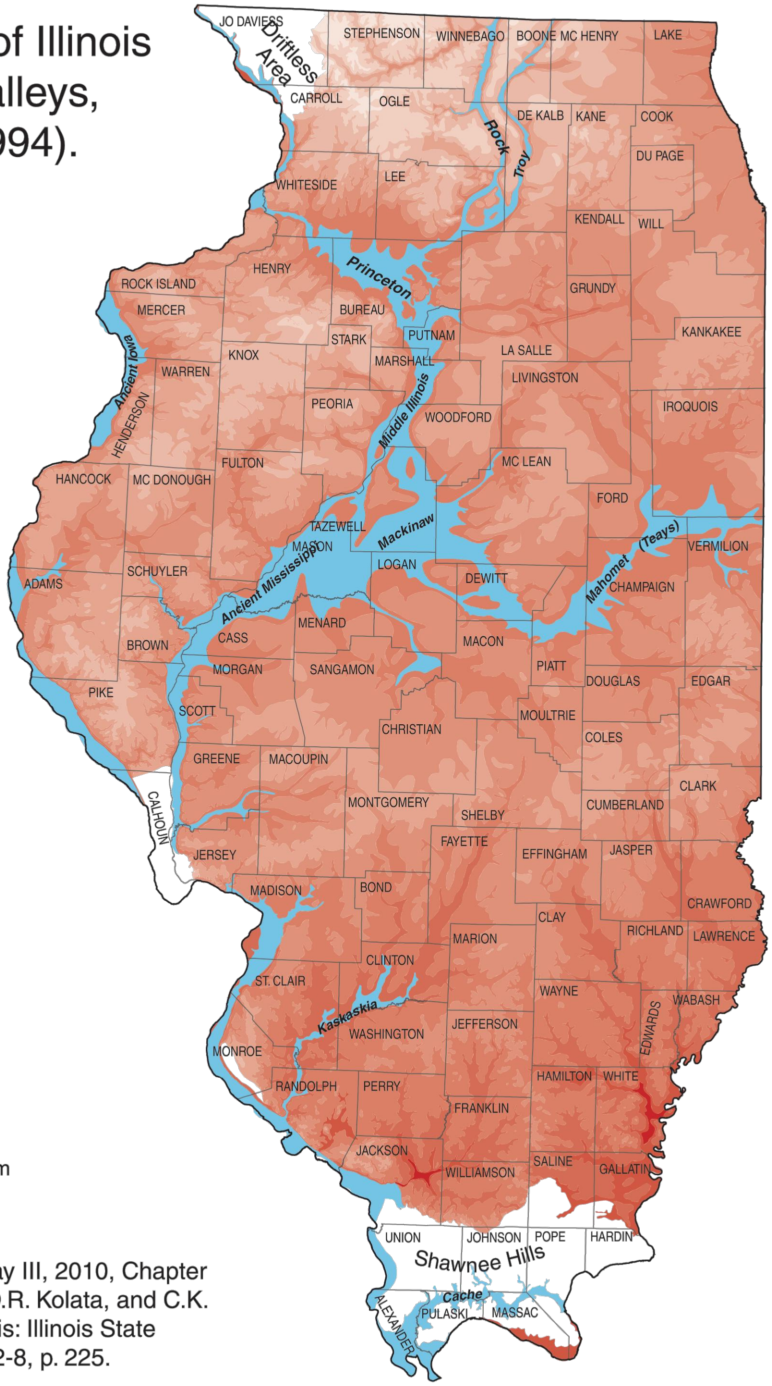
Bedrock topography of Illinois and major bedrock valleys, (from Herzog et al. 1994).





Recommended citation:
Hansel, A.K., and E.D. McKay III, 2010, Chapter 12—Quaternary Period, *in* D.R. Kolata, and C.K. Nimz, eds., Geology of Illinois: Illinois State Geological Survey, Figure 12-8, p. 225.

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FIGURE 2.2-13
BEDROCK TOPOGRAPHY OF ILLINOIS

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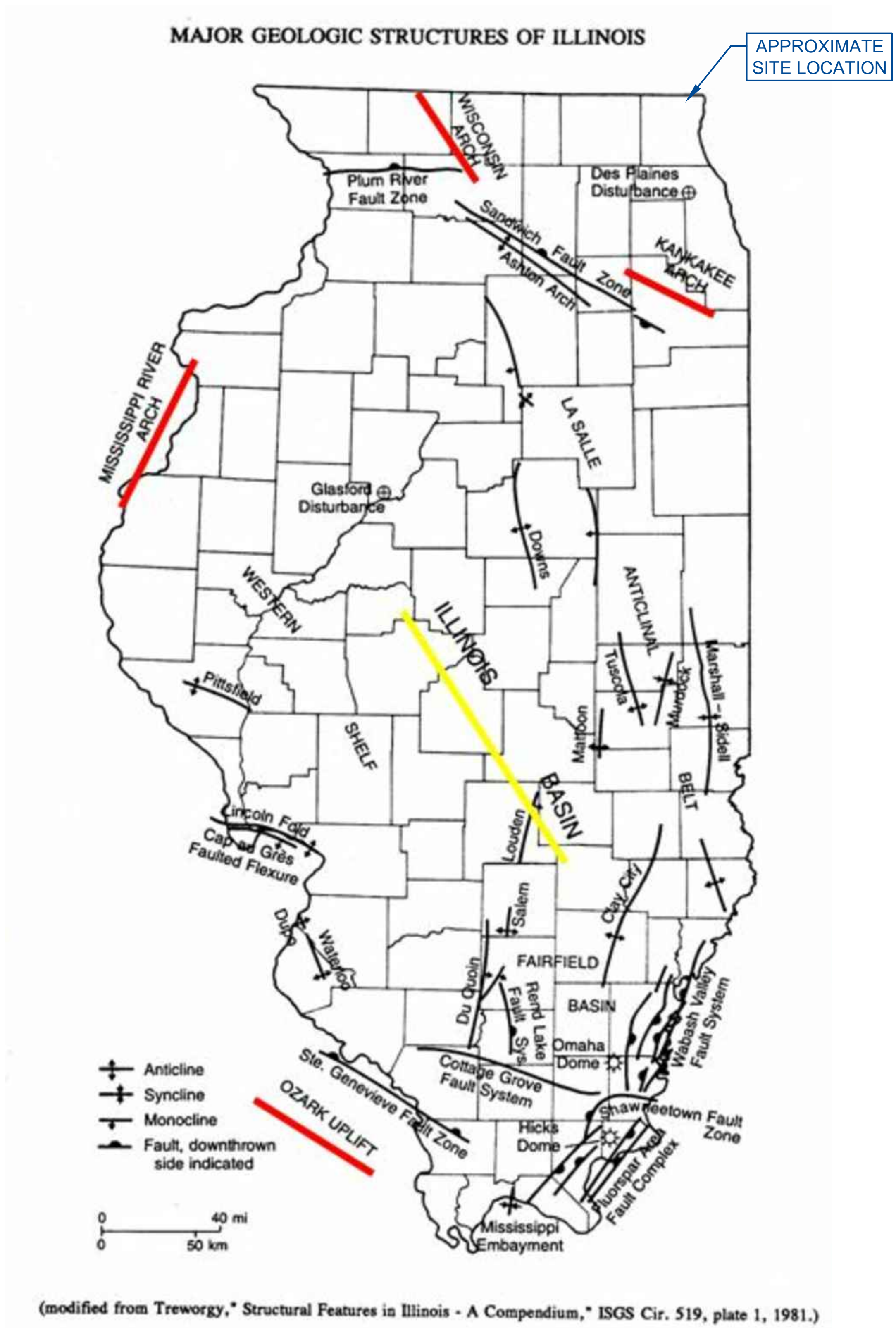

(modified from Treworgy," Structural Features in Illinois - A Compendium," ISGS Cir. 519, plate 1, 1981.)

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FIGURE 2.2-14
STRUCTURAL FEATURES IN ILLINOIS

APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

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As recently as 2008, a moderate earthquake was registered in Wabash County near Mount Carmel. However, this earthquake and others since Silurian time are not strong enough to displace the glacial sediments. The location of the Wabash Valley and Sandwich fault zones are shown on **Figure 2.2-14**.

Southeastern Wisconsin also has some documented faults. As summarized by Kammerer, et al. (1998), faults extend from Wiota (Lafayette County) to Milton (Rock County) and from east of Dodgeville (Iowa County) to Waukesha (Waukesha County). Per an article obtained through the Wisconsin Groundwater Association (Trotta, 2007), the Wiota to Milton Fault has little offset in Lafayette and Green County but about 80 feet of offset near Milton. Kammerer, et al. indicate that the eastern end of the long east-west fault from east of Dodgeville to Waukesha intersects an additional fault which extends to the northwest and one which crosses from the southwest to the northeast. Trotta (2007) further documents those faults and a series of faults associated with the fault running from east of Dodgeville to Waukesha in the area of the Yahara Hills Golf Course which is approximately halfway across the east-west trending fault. The main fault in the Yahara Hills area has about 80 feet of offset but has up to 200 feet of offset south of Fitchburg. Associated faults in the Yahara Hills Area are documented to have up to 400 feet of offset. The Waukesha Fault documented by Kammerer, et al. and Trotta has been more recently studied (Sverdrup, et al., 1997) through the use of gravity data. The results of the gravity survey indicate that the Waukesha fault actually extends from approximately 2 miles south of Eagle northwest at least to Port Washington. Of these Wisconsin faults, the Waukesha Fault is the closest to the proposed expansion site at approximately 40 miles away.

The proposed Site 2 North Expansion is located in an area that has a 90% probability of not exceeding a horizontal acceleration of 0.0461 g in 250 years (Refer to **Figure 2.2-15**). USEPA landfill locational criteria with respect to seismic sensitivity indicates that landfills are not to be located in seismically active zones characterized by an area with a 90% probability of exceeding 0.1 g in 250 years unless all containment structures are designed to withstand the maximum horizontal acceleration for the site. **Figure 2.2-15** indicates that the Facility is not within a defined seismic impact zone. However, a seismic analysis of the Site 2 North Expansion has been conducted, which is discussed in further detail within the Design Report (**Section 2.3**).

Unconsolidated Deposits

The Pleistocene Epoch marked the advance and retreat of 4 major recognized glaciations in Illinois. These glaciations, from oldest to youngest, are known as the Nebraskan, Kansan, Illinoisan, and Wisconsinan. All 4 of the glacial periods have greatly modified the landscape they covered.

Unconsolidated materials and several feet of bedrock were eroded, transported, and redeposited near the ice margins. The Pleistocene deposits in Illinois display a wide range of lithologies, varying from bouldery glacial tills, well sorted silts, and fine grained lacustrine clays. **Figure 2.2-16** shows the distribution of Quaternary deposits in Illinois. **Figure 2.2-17** illustrates the distribution of Quaternary deposits within the vicinity of the site and **Figure 2.2-18** shows a regional cross section depicting the relationships between quaternary deposits and bedrock in the region.

Glacial deposits strongly reflect their mode of origin and source area. Glacial regimen changes based on temperature and thickness of the ice, the rate of ice flow, and the manner in which the ice disappeared (Willman et al., 1975).

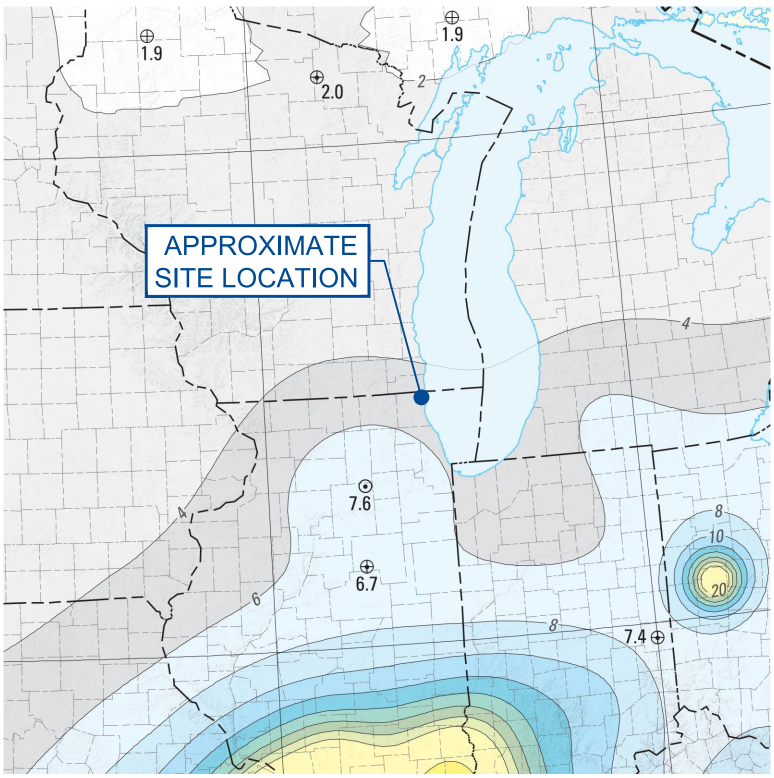
USGS
science for a changing world
Earthquake Hazards Program

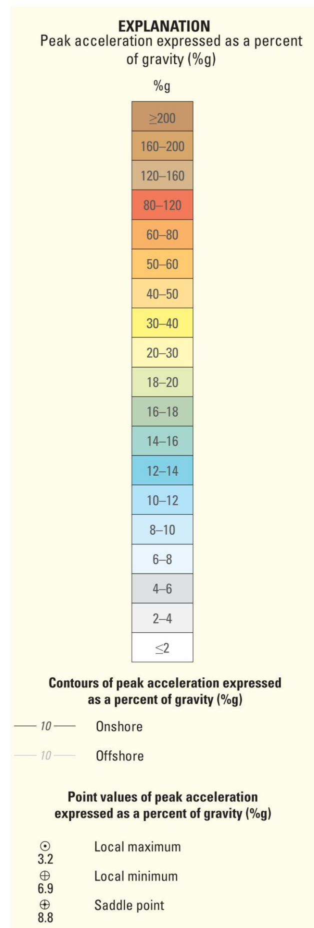
LOCATION 42.490 Lat. -87.867 Long.

The interpolated probabilistic ground motion values, in %g, at the requested point are:

P.E. %	Exp. Time (years)	Ground Motion (g)
2	50	0.0461

U.S. NATIONAL SEISMIC HAZARD MAPS: Peterson, M.D., et al, 2014





Peak Horizontal Acceleration with 2% Probability of Exceedance in 50 Years

NOTES

1. Information obtained from the United States Geological Survey website.
2. 2% P.E. in 50 years is equivalent to 10% in 250 years.

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FIGURE 2.2-15
MAP OF HORIZONTAL ACCELERATION

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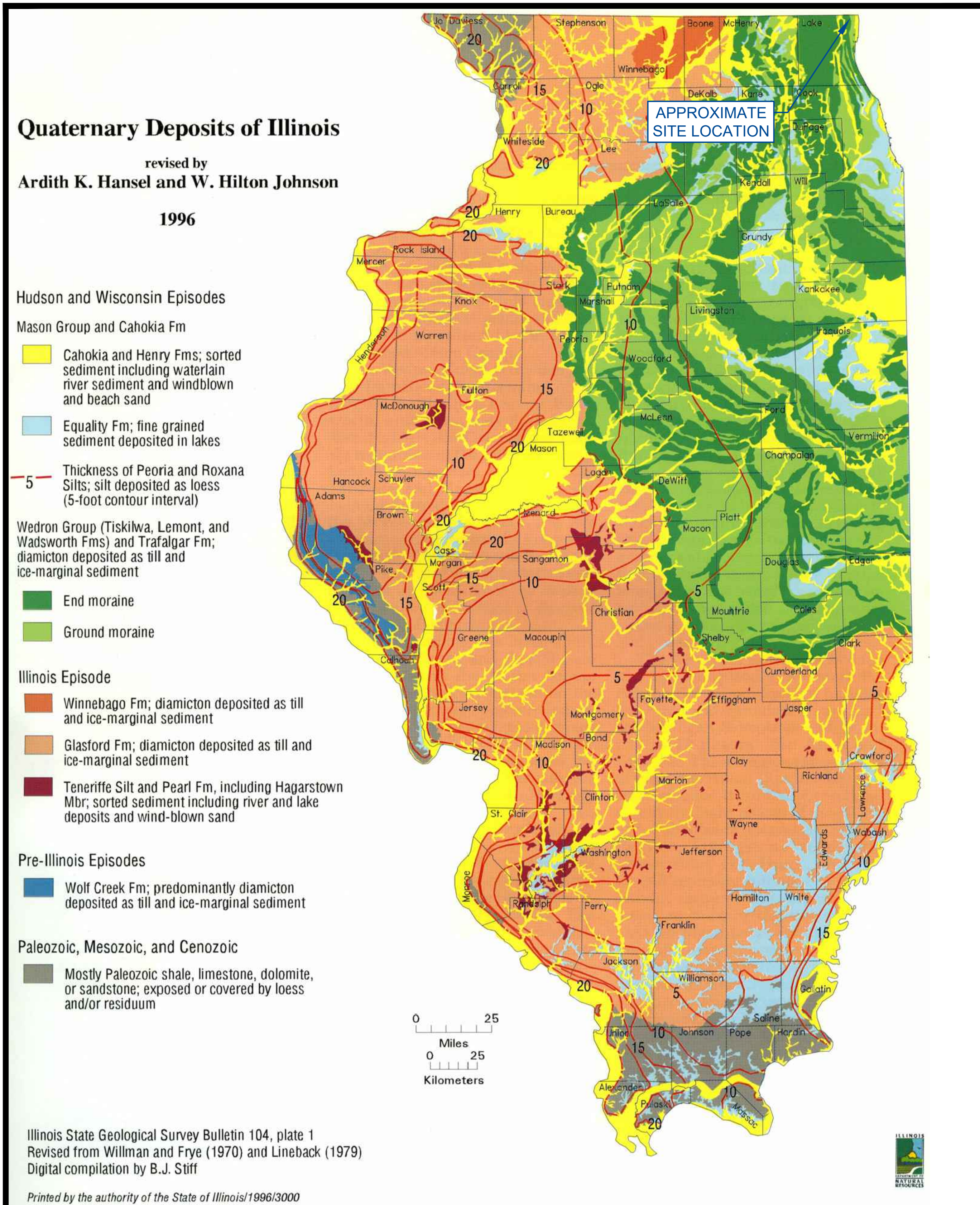

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FIGURE 2.2-16
QUATERNARY DEPOSITS OF ILLINOIS

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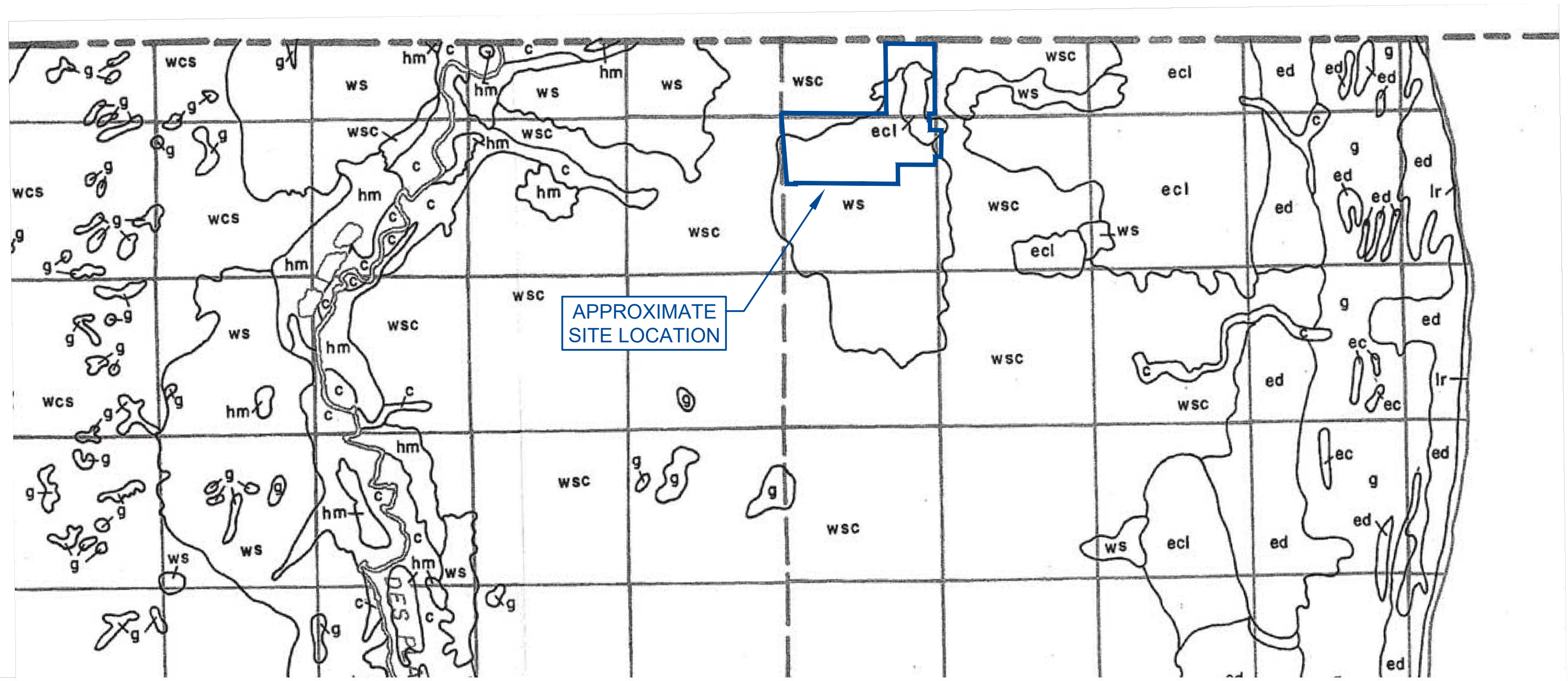

KEY TO MATERIALS

Peat and muck
- g = peat and muck

Clay and silt
- ec = lacustrine silt and clay
- ecl = silt, fine sand, and clay; less than 10 feet thick; over scoured till

Clay, silty, pebbly—glacial till
- wc = clay, pebbly (till), with associated lacustrine deposits; olive brown to gray
- wsc = clay, silty, pebbly (till); olive yellow to olive brown to gray
- wcs = silt, pebbly, locally clayey (till); yellow brown to gray brown
- ws = silt, clayey, pebbly, locally sandy (till); brownish yellow

Silt and sand, poorly sorted
- c = alluvium

Sand, locally containing some clay, silt, and rubbish
- m = made land

Sand, medium-grained, and sandy gravel
- ed = beach and shore deposits
- lr = beach deposits immediately along present Lake Michigan shore

Sand, gravel, and silt
- hw = sand, silt, and some gravel
- hb = gravel, fine-grained; sand; and silt
- hm = sand and gravel in valleys
- hww = sand and gravel in hills

Figure adapted from ISGS Circular 481 - Plate 1 (Larsen, 1973)

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FIGURE 2.2-17
SURFICIAL DEPOSITS IN THE VICINITY OF THE SITE

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Figure 1 Location of cross sections and 7.5-minute quadrangles within Lake County, Illinois.

Cross section units

dg	Disturbed ground
w	Wadsworth Formation
e	Equality Formation
h(m)	Henry Formation Mackinaw Facies
h(p)	Henry Formation Parkland Facies
h(p)gp	Henry Formation Parkland Facies and Grayslake Peat, intermixed
h(r)	Henry Formation Ravina facies
	Silurian bedrock

Cross Section

- Gravel
- Sand and gravel
- Sand, may contain some gravel or silt
- Sand and silt
- Laminated silt and clay
- Diamicton, massive silt, or other fine-grained sediment
- Contact
- Inferred contact

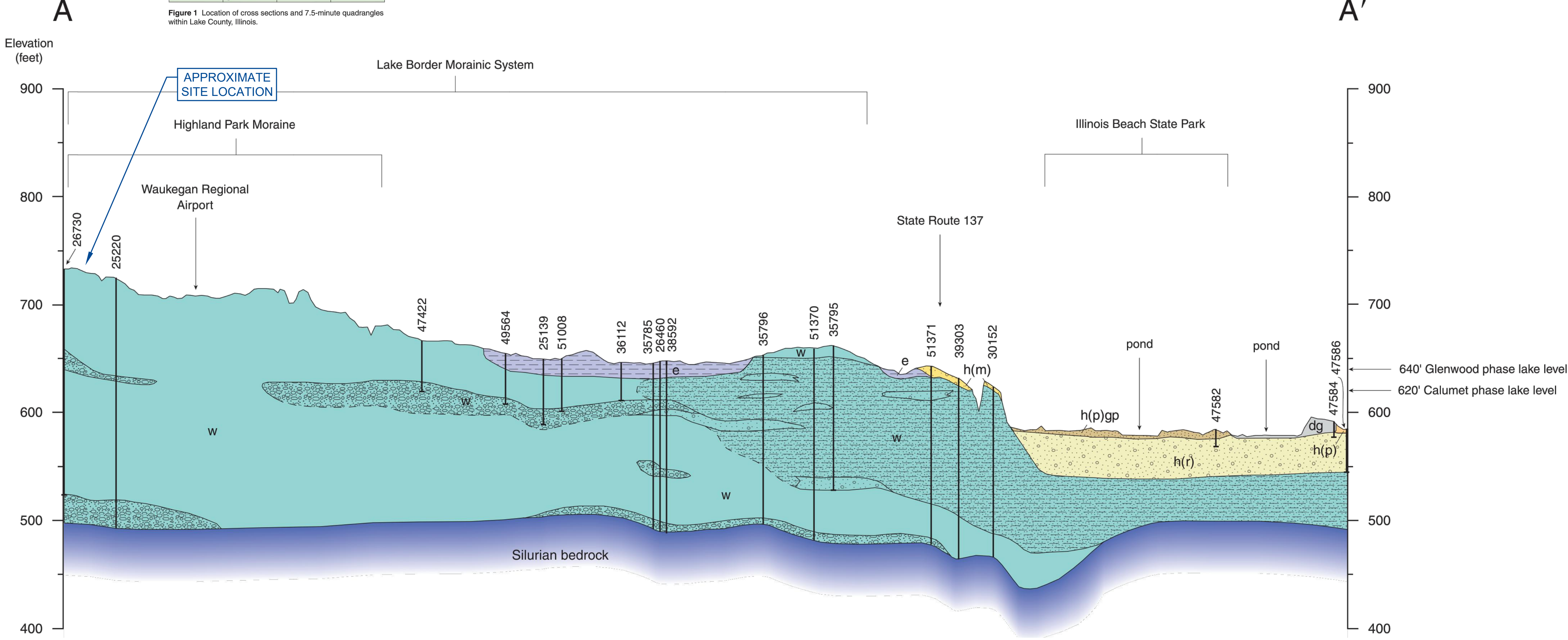

FIGURE ADAPTED FROM BARNHARDT ET AL. (2015).

REV. NO.	DATE	DESCRIPTION

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FIGURE 2.2-18
REGIONAL CROSS SECTION

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Figure 2.2-17 illustrates the location of various glacial units that have been deposited in the vicinity of the site. As illustrated in **Figure 2.2-18**, glacial deposits in the vicinity of the site are generally more than 200 feet thick.

The proposed Site 2 North Expansion is located in an area where several Wisconsin aged moraines merged or overlapped. Although it is likely that many advances and retreats of glacial ice occurred in the vicinity of the site, only evidence of the most recent glacial advances remains. **Figure 2.2-19** illustrates the location of moraines in Lake County.

Multiple minor advances and retreats have been theorized to explain the remaining unconsolidated deposits in the vicinity of the proposed expansion. The silt, sand, and gravel zone which lies directly above the bedrock (Basal Drift) was deposited by the advance and retreat of a glacier which had moved into the area across the bedrock surface. As the glacier retreated, outwash from the melting glacier formed the Basal Drift deposit. After the deposition of the Basal Drift, multiple cycles of glacial ice advance, retreat, and re-advance, resulted in the deposition of the Lower Till above the Basal Drift. Due to the homogeneity of the Lower Till, it is likely that cycling of the same glacier caused deposition of these materials. The presence of lacustrine deposits beneath and within the Lower Till is an indicator that during the periods of glacial retreat, lakes formed between the retreating ice to the east and older moraines to the west. During the last retreat of the glacier which deposited the Lower Till, outwash from the retreating glacier deposited the silt, sand, and gravel deposits above the Lower Till. These deposits are referred to as the Shallow Drift throughout this report.

Advancement of another glacier ultimately deposited the succession of clayey glacial till of the Wadsworth Formation (Wedron Group) above the Shallow Drift. It is likely that some erosion of the Shallow Drift took place during the advancement of this glacial event.

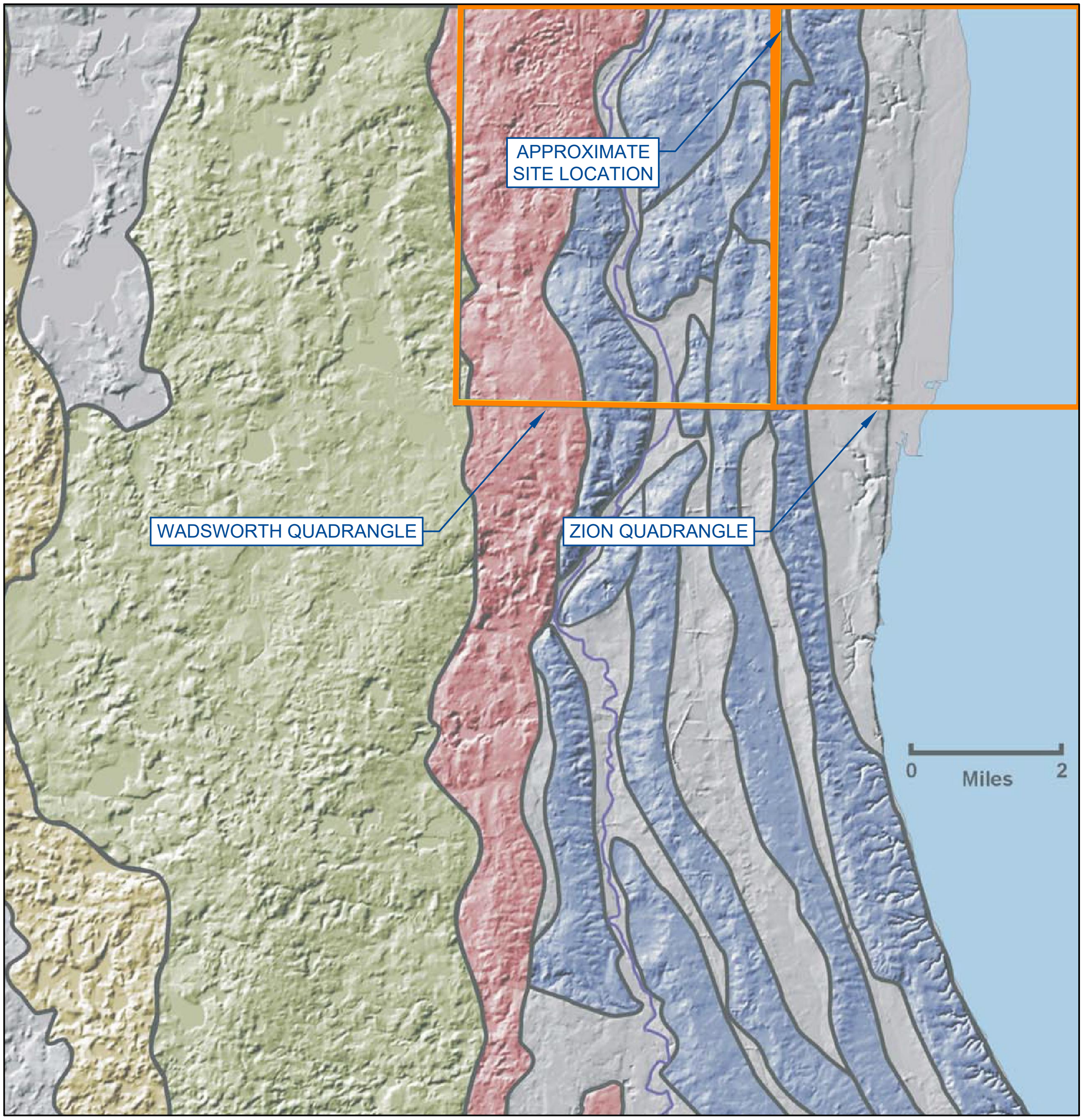
The Wisconsin Age Wadsworth Formation is a distinct gray clay-rich lithostratigraphic unit that consists of calcareous, gray, fine textured diamicton that contains lenses of sorted and stratified sediment (Hansel and Johnson, 1996).

Holocene, or recent deposits overlie all of the above mentioned deposits in the vicinity of the site. The Holocene deposits which are mainly referred to as the Peoria Silt consist of a light yellow tan to gray silt that grades from sandy silt to clayey silt. In some areas it may contain beds of well-sorted (eolian) sand, fossil and snail shells, organic debris, wood, and rarely clay layers (Hansel and Johnson, 1996). Other Holocene deposits, including beach and shore deposits of the Lake Michigan Member (Ravinia Formation) along Lake Michigan and areas of peat and muck of the Grayslake Peat Formation can be found throughout the region (**Figure 2.2-17**).

Other Wisconsin age glacial units are present in Lake County including the Haeger Member of the Lemont Formation on the far western edge of the County and the glacial outwash deposits of the Henry Formation which are found at the ground surface in the northeastern portion of the County and inter-tongued with the Wisconsin age glacial tills.

Regional Groundwater Resources

There are 4 aquigroups identified in the vicinity of the site. They are the Basal Bedrock, Midwest Bedrock, Upper Bedrock, and Prairie Aquigroups, as illustrated in **Figure 2.2-5**. The Basal Bedrock Aquigroup is composed of the Elmhurst-Mt. Simon Aquifer. The Midwest Bedrock Aquigroup contains the Ironton-Galesville, Ancell, and Galena-Platteville dolomite aquifers.



ADAPTED FROM BARNHARDT ET AL. (2001)

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FIGURE 2.2-19
MORAINES IN LAKE COUNTY

APPROVED BY: TFA	PROJ. NO.: 631020105	DATE: MAY 2022

The Upper Bedrock Aquigroup is composed of the Silurian Dolomite, and the Prairie Aquigroup contains aquifers composed of glacial outwash silt, sand, and gravel deposits (Visocky, et al., 1985).

Bedrock Groundwater Resources

The Elmhurst - Mt. Simon Aquifer includes productive sandstone aquifers below thick, regionally extensive shale of the Eau Claire Formation. The sandstone of the Elmhurst - Mt. Simon Aquifer lies unconformably above Cambrian granite and can be greater than 1,500 feet thick in Lake County (Visocky, et al., 1985).

The Ironton-Galesville Aquifer system is comprised of the Galesville and Ironton Sandstones. The Galesville Sandstone is fine-grained, well-sorted sandstone, essentially free from shale and glauconite, whereas the Ironton is medium-grained, generally poorly-sorted, dolomitic sandstone. Both sandstones occur throughout the northern half of Illinois and lie above the shale deposits of the Eau Claire Formation and beneath the glauconitic, argillaceous sandstone of the Fraconia Formation. The aquifer system is approximately 150 ft. thick in the vicinity of the proposed expansion (Visocky, et al., 1985).

The St. Peter Sandstone of the Ancell Aquifer is a relatively pure and very fine to coarse grained sandstone present across most of Illinois. The St. Peter Sandstone is situated directly beneath the Platteville Group carbonates and unconformably overlying uneroded carbonates of the Knox Megagroup. Where tapped to provide water supplies, the Ancell Group typically provides small to moderate quantities of potable water (Visocky, et al., 1985).

Groundwater obtained from the Galena-Platteville dolomite is less highly mineralized than in deeper bedrock formations. However, the transmissivity of this formation is dependent upon the degree of interconnectedness of fractures through which groundwater migrates. As the nature of the fractures is highly variable from one location to another, the quantity of water obtainable from this formation is variable.

The Silurian Dolomite is commonly tapped for domestic water wells in the vicinity of the proposed expansion. It is separated from the underlying Galena-Platteville dolomite by the Maquoketa Shale. The upper third of the Silurian Dolomite is the most productive part because numerous fractures, crevices, and solution cavities occur there. The greater the number of such openings intersected in the well bore, the higher the well yield. However, it should be noted that in many areas, the water quality of the Silurian dolomite is adversely affected by the presence of naturally occurring gas, oil, and hydrogen sulfide.

The Elmhurst-Mt. Simon Sandstone, the Ironton-Galesville Sandstone, and the St. Peter Sandstone (Ancell Group) are found throughout northeastern Illinois and furnish large quantities of water to the cities, villages, and industries of this region. In Lake County, domestic water wells which tap bedrock are primarily screened within the St. Peter Sandstone, Galena-Platteville Dolomite, or Silurian age dolomites. In the vicinity of the site, most wells which utilize the bedrock for a drinking water source are screened within the Silurian dolomite. This is primarily due to its relatively shallow depth and lower well installation and maintenance costs. The deeper aquifers are used only for larger municipal and industrial water supplies because construction and maintenance costs are high (Berg, et al., 1984).

Surficial and Glacial Deposit Groundwater Resources

The other major sources of water supply in the vicinity of the site are surface water bodies (Lake Michigan), aquifers located within the glacial drift (Shallow Drift Aquifer), and aquifers located below the glacial drift connected with the bedrock surface (Basal Drift Aquifer).

Lake Michigan serves as the primary surficial source of community water supply within the site region. The City of Zion, began purchasing water from the Zion-Benton Treatment Plant (Lake County Public Water District) in 1957.

The silt, sand, and gravel deposits of the Shallow Drift and Basal Drift Aquifers (Prairie Aquigroup) are utilized by many of those nearby residents that do not have access to water from the Lake County Public Water District (Lake Michigan). Generally these shallow deposits are preferable over deeper bedrock formations in the area due to the lower cost of construction and lower mineralization than the deeper aquifers. Approximately 100 of the 334 well logs that were obtained from within 1 mile of the proposed expansion appear to indicate that the well is screened within the Shallow Drift Aquifer. It should be noted that in some cases, those wells that pull water from the Basal Drift Aquifer also utilize the upper portion of the Silurian Dolomite.

Discontinuous silt, sand, and gravel deposits which are contained within the Wadsworth Formation and Lower Till (Intra-Till Sediments) do not generally exhibit sufficient yield to serve as a water source. This is clearly illustrated through site-specific data that has been collected and through analysis of regional water well logs. Only 6 of the 334 well logs that were obtained from within 1 mile of the proposed expansion appear to indicate that the well is screened within a zone above the Shallow Drift Aquifer.

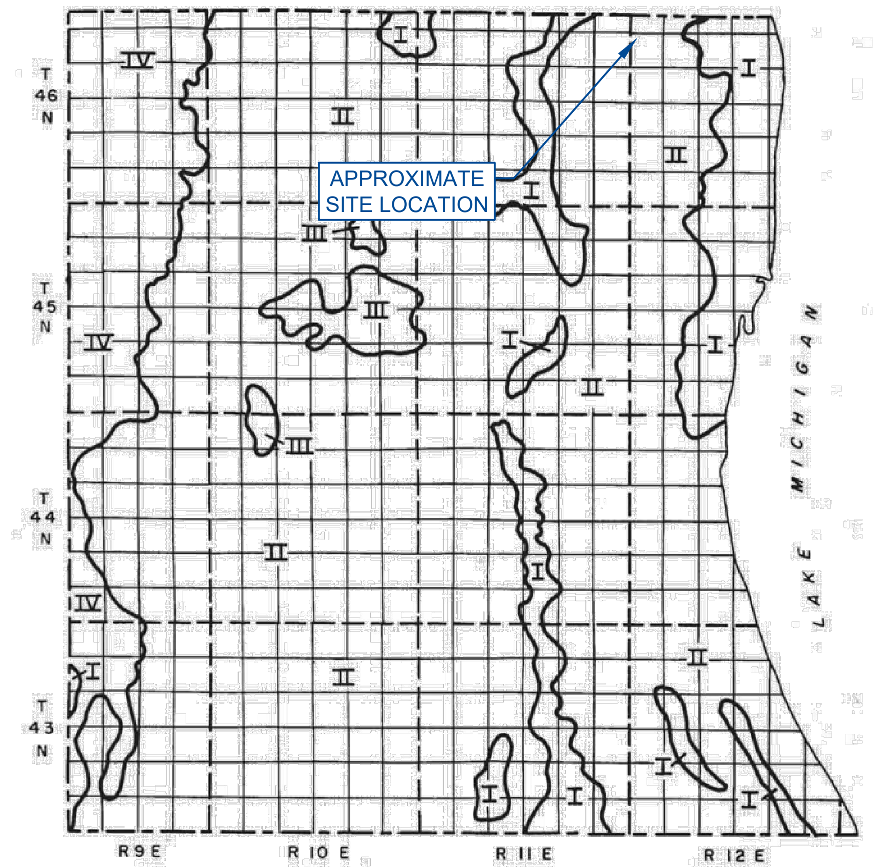
Aquifer Sensitivity

The publication "Geology for Planning in Lake County, Illinois," (Larsen, 1973) states that, "Suitable sites for solid-waste disposal may be found in the widespread morainic uplands of the county, where the relatively fine-grained surficial materials naturally re-strict the movement of pollutants." **Figure 2.2-20** illustrates the surficial materials of Lake County which have been differentiated by Larsen on the basis of their properties related to waste disposal. As can be seen in **Figure 2.2-20**, the proposed expansion site is located in an area which has been identified as being geologically optimal for the development of a landfill within Lake County.

In 1984, Berg et. al. created another map which classified the area on a basis of potential for contamination of shallow aquifers from land burial of municipal wastes. The map created by Berg et al., (Refer to **Figure 2.2-21**) indicates that the site is in an area designated as category "E", exhibiting the lowest potential for aquifer contamination within Lake County. It should also be noted that Category "E" indicates that the area is one of the best locations in the State of Illinois for land burial of municipal waste.

Coal Mining

The Directory of Coal Mines in Illinois and the Wisconsin Department of Natural Resources (WDNR) were reviewed for coal mines in or near Lake County. Per the Directory, zero known coal mines were identified to have operated in Lake County. In fact, the nearest coal mine is located over 75 miles south south-west of the proposed landfill site. Due to the distances



I	Surficial sand, gravel, or silt, 5–40 ft. thick, overlying till
II	Silty clay till, silt till, or clay till, 40–80 ft. thick
III	Fine sand, silt, or gravel, 10–20 ft. thick, overlying till
IV	Coarse sand and gravel, some silt, up to 100 ft. thick

Area I "Dissolved substances migrating from landfills tend to move laterally with the ground water through the relatively permeable surficial deposits and downward into the underlying till."

Area II "In most of Area II, there is not much danger of polluting groundwater resources by the disposal of solid wastes although pollution of surface water is possible. Most of the dissolved substances migrating into the subsurface would move downward through the till and be attenuated to low levels within short distances."

Area III "Dissolved substances migrating from waste disposal activities could move with the ground water through the relatively permeable surficial materials into lakes or swamps. Hydrogeologic conditions are such that caution is necessary in planning waste-disposal activities."

Area IV "The potential to pollute groundwater through waste-disposal activities is greater in Area IV than in any other in Lake County. Waste disposal activities in Area IV should therefore be planned with care."

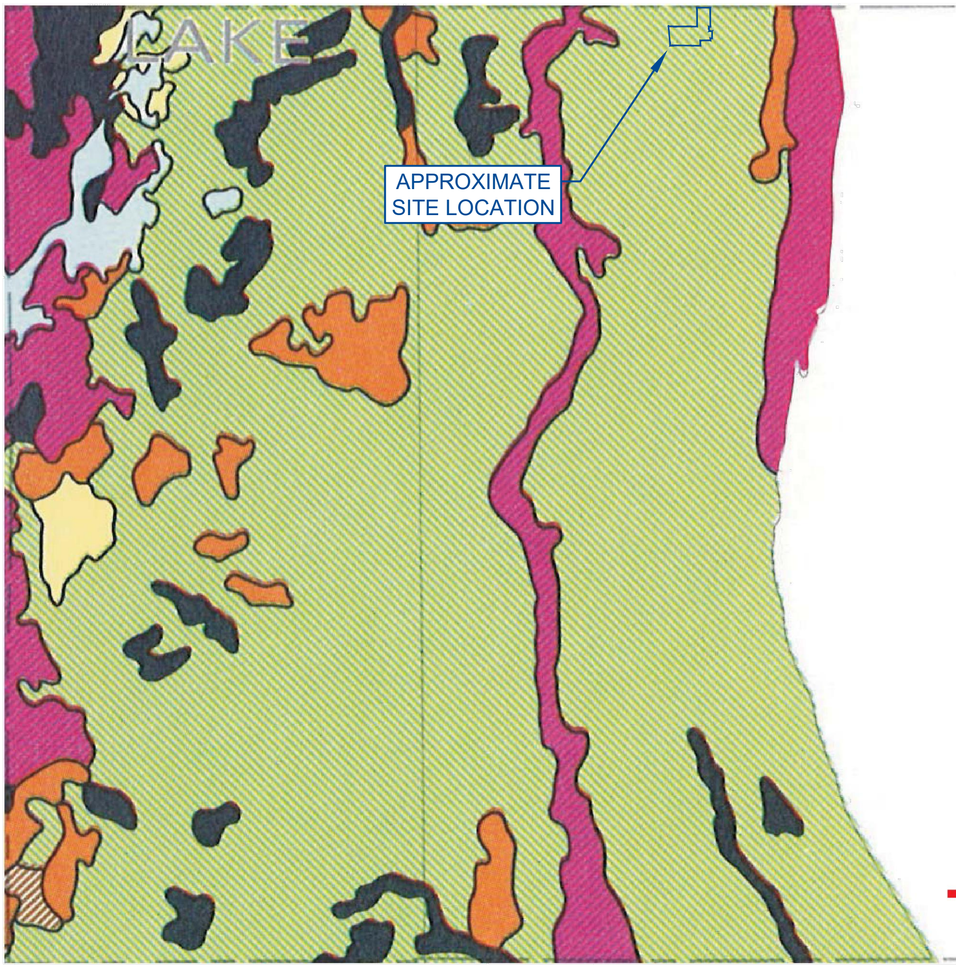
Figure adapted from Larsen 1973.

APTIM Environmental & Infrastructure, LLC

ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-20
SURFICIAL MATERIALS OF LAKE COUNTY DIFFERENTIATED ON BASIS OF PROPERTIES RELATED TO WASTE DISPOSAL

APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022



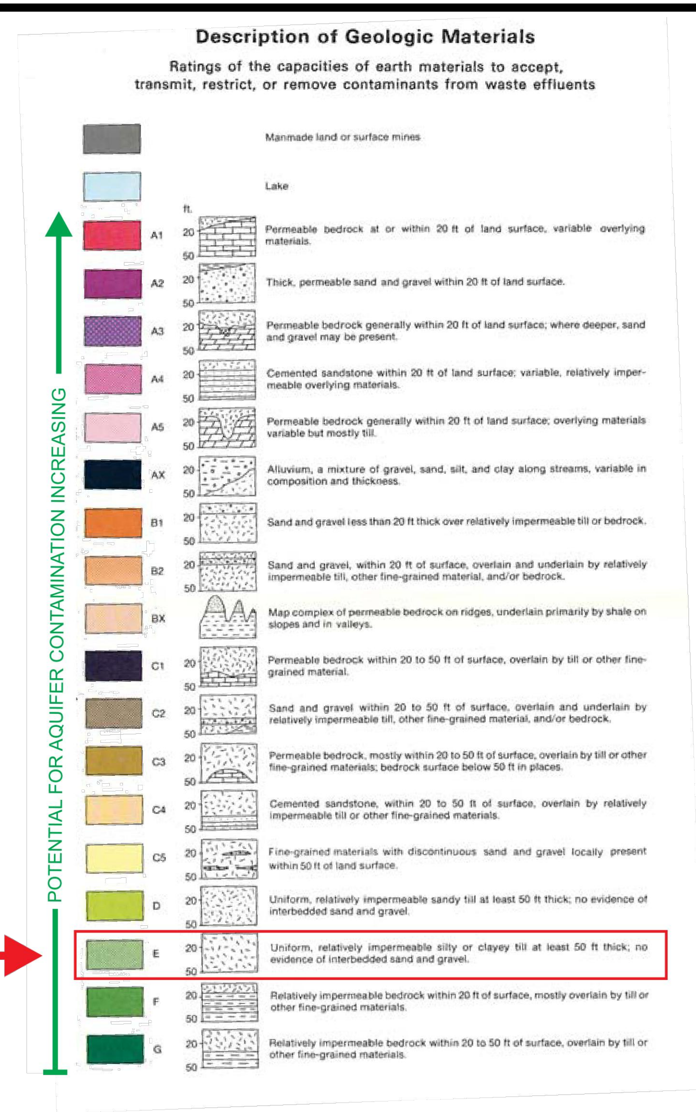


Figure modified from Plate 1 within ISGS Circular 532 (Berg and Kempton, 1984)

APTIM Environmental & Infrastructure, LLC

ZION LANDFILL SITE 2 NORTH EXPANSION HYDROGEOLOGIC INVESTIGATION

FIGURE 2.2-21
POTENTIAL FOR CONTAMINATION OF SHALLOW AQUIFERS FROM LAND BURIAL OF MUNICIPAL WASTES

DRAWN BY:	NV	APPROVED BY:	TFA	PROJ. NO.:	631020105	DATE:	MAY 2022

away from the proposed site, the stability of the proposed landfill will not be affected by the potential presence of coal mines.

Site Specific Hydrogeologic Investigation

Throughout the history of the Zion Landfill, numerous subsurface investigations have been performed within, and surrounding the various landfill units. All site-specific geological conditions discussed herein were derived from the evaluation of continuously sampled borings located within or near the vertical and horizontal expansion area.

Table 2.2-2 lists the continuously sampled borings that were used for construction of new geologic cross-sections through the proposed Site 2 North Expansion area and the adjacent existing landfill area to the south, surface and isopach maps of the Wadsworth Formation, a surface contour map of the Shallow Drift, and/or to obtain other information presented within this discussion of site-specific geology. The location of these borings are illustrated on **Drawing No. G3**.

As shown in **Table 2.2-2**, 15 of the continuously sampled borings (B-01-18 through B-15-18) were advanced during the most recent hydrogeological investigation. This investigation was performed in order to supplement previously collected information and to characterize the geologic and hydrogeologic characteristics beneath the proposed horizontal expansion area. In addition to boring advancement, 23 new piezometers were installed as summarized in **Table 2.2-2**. **Table 2.2-2** also summarizes additional piezometers or monitoring wells which currently exist at the continuously sampled boring locations within, or near, the expansion site. The locations of all monitoring wells, piezometers, and gas probes (current and former) at the site are illustrated on **Drawing No. G4**. All boring logs are located in **Appendix G**.

All new borings have a -18 designation. The other borings from previous investigations which were used in the characterization of geology beneath and adjacent to the proposed horizontal and vertical expansion areas have either a -07 designation or an EB or GK prefix.

The -07 borings (B-1-07 through B-10-07) were advanced by APTIM (then Shaw E&I) in March through May of 2007. Six piezometers/monitoring wells had been previously or were installed in 2007 at 6 of the boring locations, including P-8 at B-2-07, G178 at B-7-07, MW-3-07 at B-3-07, MW-6-07-D at B-6-07, MW-8-07 at B-8-07, and MW-9-07 at B-9-07.

The EB- borings (EB-6 through EB-8 and EB-10 through EB-15) were advanced by Testing Service Corporation in March of 1986. Initially, 4 piezometers were installed at these boring locations, including: P-8, EP-10S, EP-10I, and EP-10D. The S piezometer screens glacial till, the I-piezometer screens Intra-Till Sediments, and the D-piezometer screens the Shallow Drift Aquifer. Shallow piezometer EP-10S was ultimately replaced with EP-10S(R) due to an ineffective seal.

Boring logs and as-built diagrams from previously advanced borings and installed piezometers and monitoring wells located to the south of the proposed Site 2 North Expansion area are included in **Appendix G**, along with boring logs and piezometer as-built diagrams from the most recent investigation. The procedures used to conduct the most recent investigation activities are discussed in detail within the following sections.

TABLE 2.2-2
SUMMARY OF BORING AND MONITORING WELL/PIEZOMETER INFORMATION WITHIN OR NEAR THE PROPOSED VERTICAL AND HORIZONTAL EXPANSION AREA

Previously Advanced Boring	Existing Monitoring Well(s) / Piezometer(s) at Location Prior to the Most Recent Investigation	New Boring Advanced	New Piezometer Installed
EB-6	-	-	-
EB-7	-	-	-
B-1-07	-	-	-
EB-8/B-2-07	P-8	-	-
B-3-07	MW-3-07	-	-
B-4-07/TB-1	-	-	-
EB-10/B-6-07	EP-10SR, EP-10D, EP-10I, and MW-6-07-D	-	-
EB-11/B-5-07	-	-	-
EB-12	-	-	-
EB-14	-	-	-
EB-15	-	-	-
GK9D/G172	-	-	-
B-7-07	G178	-	-
B-8-07	MW-8-07	-	-
B-9-07	MW-9-07	-	-
B-10-07	-	-	-
-	-	B-01-18	P-01-18IT and P-1-18SD
-	-	B-02-18	-
-	-	B-03-18	P-03-18SD
-	-	B-04-18	P-04-18USD and P-04-18LSD
-	-	B-05-18	P-05-18SD
-	-	B-06-18	P-06-18USD and P-06-18LSD
-	-	B-07-18	P-07-18USD and P-07-18LSD
-	-	B-08-18	P-08-18IT, P-08-18SD, and P-08-18D
-	-	B-09-18	P-09-18SD
-	-	B-10-18	P-10-18SD
-	-	B-11-18	P-11-18SD
-	-	B-12-18	P-12-18IT and P-12-18SD
-	-	B-13-18	P-13-18IT and P-13-18SD
-	-	B-14-18	P-14-18IT and P-14-18SD
-	-	B-15-18	P-15-18SD

Hydrogeologic Investigation Methodology

A description of each of the procedures used to perform the hydrogeologic component of this investigation is discussed in detail in the following subsections.

Drilling and Field Procedures

All field exploration was performed by a team of experienced geologists and engineers under the direction of a Licensed Professional Engineer and Licensed Professional Geologist. The borings were drilled by an experienced drilling crew using rotary drill rigs mounted on a truck or an all-terrain vehicle (ATV) (refer to **Photograph 2.2-1**). The field geologists and engineers maintained daily drilling records, logged the soil samples and rock samples, selected representative samples for laboratory testing, performed field hydraulic conductivity testing, and supervised the installation of the piezometers. The boring logs, as-built diagrams, IDPH well construction reports, and required borehole sealing forms are provided in **Appendix G**. The unconsolidated samples were described in general conformance with the Unified Soil Classification System (ASTM D 2487) along with locally adapted soil description terminology, both of which are presented with the boring logs in **Appendix G**. The results of the hydraulic conductivity and laboratory testing are located in **Appendices H and I**, respectively.

Photograph 2.2-1
All-Terrain Drill Rig

Soil Sampling

The investigation included the advancement of 15 borings (B-01-18 through B-15-18) with the installation of 23 piezometers. As previously indicated, all of the borings and piezometers confirmed the general stratigraphy reported in the available regional information and previous hydrogeologic investigations at the adjacent permitted landfill. Additionally, the investigation included aquifer testing and geotechnical laboratory analyses. The boring locations are shown on **Drawing No. G3**.

Boreholes were advanced through unconsolidated deposits using a set of 6 inch O.D. solid flight augers (0 to 10 feet) and a 3 7/8 inch tri-cone roller bit beyond 10 feet. Continuous sampling was conducted at each of the 15 boring locations advanced during the most recent investigation.

Soil samples were obtained using one of the following methods: 1) driving a 2-inch O.D. standard penetration test split spoon sampler in accordance with ASTM D1586 or 2) pushing a thin-walled 3-inch diameter Shelby tube in accordance with ASTM D1587. The sample method, soil type, location, and recovery for each sample interval are shown on the boring logs located in **Appendix G**.

Penetrometer tests were performed in the field on cohesive samples using a calibrated pocket penetrometer. The test results serve as a general measure of consistency and to estimate unconfined compressive strengths. **Photograph 2.2-2** illustrates a penetrometer test being performed on a sample of glacial till which was obtained using a 2-inch O.D. split spoon.

Photograph 2.2-2
A penetrometer test being performed on a sample of glacial till

Representative soil samples obtained using the standard penetration method were placed into 8-ounce clear glass jars sealed with air-tight screw top lids. When a break in the soil stratigraphy was logged within a split spoon sample interval, the sample was split and samples above and below the break were collected. After sealing the jars, individual samples were labeled and packaged for transport.

In addition, representative samples obtained using Shelby tubes were sealed with paraffin wax, and then capped at the ends. All thin-walled Shelby tube samples were carefully transported in a vertical position to Midland Standard Engineering and Testing of East Dundee, Illinois for extrusion, logging, and/or testing. **Photograph 2.2-3** shows a 3-inch diameter Shelby tube that has been sealed and is ready for transport to a geotechnical laboratory for analysis.

Photograph 2.2-3
Shelby tube used for collection of sample for geotechnical laboratory analysis

Piezometer Installation

A total of 23 open standpipe piezometers were installed (either individually or within clusters) at 14 of the 15 continuously sampled boring locations to depths ranging from approximately 42 to 206 feet below ground surface (bgs). The additional borings for the clustered piezometers were advanced through unconsolidated deposits using a set of 6 inch O.D. solid flight augers (0 to 10 feet) and a 3 7/8 inch tri-cone roller bit beyond 10 feet and were not continuously sampled since the stratigraphy at the clustered piezometer location can be assumed to be the same as the adjacent continuously sampled borings (all wells were installed within 10 feet of the continuously sampled boring). The locations of the piezometers are illustrated on **Drawing No. G3**. The piezometers were installed to allow aquifer testing, on-going water level measurements, and the ability for water quality sample collection within the various units encountered.

All piezometer boreholes were thoroughly flushed with fresh potable water prior to construction of the piezometer. The boreholes were flushed until the return water was clear of suspended fines.

The piezometers consisted of a 2-inch diameter Schedule 40 PVC pipe with 5 or 10 foot long, 0.010-inch slotted type Schedule 40 PVC well screens. A sand filter pack was installed from the bottom of the boring to at least 2 feet above the top of the slotted well screen. The depth of the screen elevation and the top of the sand pack was measured and recorded in the field by the project engineer/geologist. **Photograph 2.2-4** shows the sand filter pack being placed.

Photograph 2.2-4
Installation of sand filter pack

A bentonite pellet seal was installed at the top of the sandpack (refer to **Photograph 2.2-5** which shows the bentonite pellet seal being placed) and the annular space above the seal was tremie grouted to approximately 2 feet below the ground surface using cement/bentonite grout. **Photograph 2.2-6** shows a batch of grout being mixed. At all well locations concrete was placed above the grout seal after allowing time for the seal to settle. The concrete extends from an average depth of 2 feet bgs to the ground surface and the well locations were finished with a 4-inch circular steel protective outer casing which was secured with a padlock. Finally, protective bumper posts were placed around all completed piezometers.

All piezometers were installed in accordance with 35 Ill. Admin. Code, Sections 811.318 and the USEPA Handbook of Suggested Practices for the Design and Installation of Ground-Water Monitoring Wells. **Table 2.2-3** summarizes the location and depths of the newly installed piezometers. **Appendix G** includes copies of all boring logs, piezometer as-built diagrams, and IDPH well construction reports. **Photograph 2.2-7** illustrates a typical finished piezometer cluster. After completion, well construction reports were sent to the LCHD and IDPH on IDPH well construction report forms.

Piezometer Development

Development of the piezometers was accomplished by pumping the wells dry or until a minimum of 5 well volumes were removed and the water was clear of suspended fines. This process included surging of the piezometers to loosen any material at the base of the screened intervals and/or within the filter pack. Water quality field measurements (pH, conductivity, temperature, and turbidity) were taken during development and are recorded in development forms for each location.

Photograph 2.2-5
Installation of bentonite pellet seal

Photograph 2.2-6
Mixing bentonite grout prior to pumping it into piezometer annulus

TABLE 2.2-3 SUMMARY OF PIEZOMETER INFORMATION					
Monitoring Well	**Northing**	**Easting**	**Unit**	**Top of Sandpack Elevation (FT-MSL)**	**Bottom of Sandpack Elevation (FT-MSL)**
P-01-18IT	13134.36	11523.09	Intra-Till Sediments	697.39	689.99
P-01-18SD	13129.54	11523.43	Shallow Drift	652.99	639.99
P-03-18SD	13129.62	13038.69	Shallow Drift	655.58	641.22
P-04-18USD	13621.32	11614.82	Shallow Drift	649.62	637.02
P-04-18LSD	13615.89	11614.85	Shallow Drift	626.59	619.11
P-05-18SD	13628.73	12391.54	Shallow Drift	630.29	622.51
P-06-18USD	13638.28	13043.36	Shallow Drift	647.18	634.70
P-06-18LSD	13632.77	13043.36	Shallow Drift	625.46	612.87
P-07-18USD	14157.94	11602.38	Shallow Drift	656.65	644.25
P-07-18LSD	14162.75	11601.29	Shallow Drift	629.19	616.46
P-08-18IT	14142.76	12384.45	Intra-Till Sediments	692.43	684.74
P-08-18SD	14139.31	12384.45	Shallow Drift	628.30	620.88
P-08-18D	14134.91	12383.73	Basal Drift	549.34	536.86
P-09-18SD	14125.68	13178.87	Shallow Drift	647.27	634.05
P-10-18SD	14652.76	11602.68	Shallow Drift	645.64	633.01
P-11-18SD	14545.78	12392.18	Shallow Drift	626.26	614.05
P-12-18IT	14641.44	13210.12	Intra-Till Sediments	694.90	682.23
P-12-18SD	14646.46	13209.63	Shallow Drift	647.77	635.04
P-13-18IT	15107.46	11532.14	Intra-Till Sediments	688.04	680.45
P-13-18SD	15112.47	11532.48	Shallow Drift	646.81	634.29
P-14-18IT	15154.47	12399.61	Intra-Till Sediments	662.75	654.98
P-14-18SD	15154.58	12405.35	Shallow Drift	624.80	617.06
P-15-18SD	15144.13	13213.52	Shallow Drift	650.70	636.54

Photograph 2.2-7
Finished piezometer cluster at boring location B-14-18

All piezometers were developed in accordance with the USEPA Handbook of Suggested Practices for the Design and Installation of Ground-Water Monitoring Wells. **Photograph 2.2-8** shows the setup for development of a typical piezometer. Refer to **Appendix G** for development forms.

Photograph 2.2-8
Well development at completed piezometer

Surveying

The locations and elevations of all the borings and piezometers installed during this investigation were determined by conventional surveying procedures. All surveying work was performed under the direction of a Registered Land Surveyor. The locations are shown on **Drawing No. G3**. All surveying was conducted in accordance with national map accuracy standards. Horizontal locations are accurate to ±0.5 feet. Ground surface elevations are accurate to ±0.1 feet. Well casing elevations are accurate to ±0.01 feet.

Borehole Abandonment Procedures

Boreholes which were not converted into piezometers were abandoned and sealed in accordance with the applicable IDPH regulations and 35 Ill. Admin. Code, Section 811.316. The boreholes were tremie grouted from the bottom of the borehole to the ground surface with cement/bentonite grout. The geologist or engineer documented the abandonment and sealing and prepared the required IDPH well abandonment forms. The forms were submitted to the LCHD and IDPH. Copies of the forms are enclosed in **Appendix G**.

Water Level Measurements

Water level measurements were obtained from on-site piezometers and monitoring wells on quarterly from 1st Quarter 2019 through 1st Quarter 2021. The depth to water from the top of the riser was measured using an electronic water level indicator. The water levels were converted to MSL elevations using the surveyed top of PVC riser elevations for each piezometer or monitoring well. The water level elevations are summarized in **AppendixG.**

In-Situ Hydraulic Conductivity Testing

Slug tests were performed in all 23 piezometers using falling and rising head tests in order to determine the in-situ hydraulic conductivities of the geologic units at the site. Each test measures the hydraulic conductivity of the zones into which the screens and the sand packs were installed.

Falling and rising head tests were performed by lowering or retrieving a solid PVC slug of known volume into or from the static water (See **Diagram 2.2-1**). An In-Situ Level Troll 700

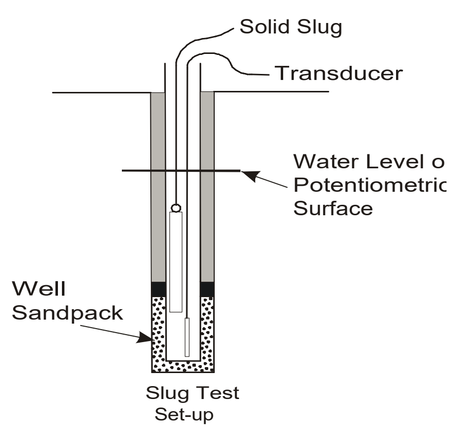


Diagram 2.2-1
Simplified Diagram of Slug Test Assembly

data logger/pressure transducer was used to record the water level versus time following the removal/insertion of the slug or water.

The potentiometric data for various geologic formations encountered at the subject site indicate that the Intra-Till Sediments, Shallow Drift Aquifer, and the Basal Drift Aquifer are under confined conditions. Therefore, a confined aquifer solution proposed by Bouwer and Rice (1976) was selected for analyzing the slug test data from these wells. This method, as described by Freeze and Cherry (1979), is appropriate for analyzing slug test data under confined conditions. This method assumes that the aquifer is homogeneous and isotropic $(k_h/k_v)=1$, of uniform thickness, and flow to the well is horizontal. It should be noted that piezometer P-13-18SD exhibited an oscillatory slug test response and was therefore analyzed using the Butler solution for confined aquifers. The Butler solution accounts for inertial effects in the well and oscillatory slug test response in high hydraulic conductivity aquifers (Butler, 1998).

The analyses were performed using the AQTESOLV™ for Windows™ program. The results of the analysis, including the data plots and a summary table, are provided in **Appendix H**. Also included in **Appendix H** is a summary table of slug test results for previously installed piezometers and monitoring wells at the existing landfill and expansion property.

Laboratory Soil Testing

Laboratory testing was utilized to characterize the properties of the unconsolidated materials encountered for geotechnical purposes, and to aid in characterizing different hydrogeologic units present in the investigation area.

Testing was performed in general accordance with the American Society of Testing Materials (ASTM) standard procedures as applicable. The results of all laboratory tests performed during the investigation are presented in **Appendix I**. A brief description of tests and their purpose are provided below:

- Moisture Content Tests (ASTM D2216) - Moisture Content tests were performed on selected cohesive soil samples. Moisture contents indicate the state of the soils relative to the soil plasticity and density, i.e., whether the soils are generally dry, moist, wet, or saturated. Further, if the soils are saturated, the moisture content is indicative of the soil's total porosity.

- Atterberg Limit Tests (ASTM D 4318) - Atterberg limit tests were performed on samples to evaluate plasticity characteristics. Soil plasticity values near the moisture content suggest that the soils should be relatively easy to compact. Liquid limit values near the soil moisture content suggest that the soils will need to be worked/dried prior to compaction.

- Grain Size Analyses (ASTM D 422) - Combined hydrometer grain size analyses were performed to evaluate grain size distribution (percentages of gravel, sand, silt, and clay). This test provides a basis for classifying the soil profile constituents.

- Specific Gravity (ASTM D 854) - Specific gravity analyses were performed in order to calculate the relative volume of solids to water and air in a given volume of soil.

- Hydraulic Conductivity Tests (ASTM D 5084) - Hydraulic conductivity tests were performed on undisturbed soil samples to evaluate hydraulic conductivity in the vertical direction.

  Soil samples were placed in the triaxial cell and were backpressure saturated under confining stress. After saturation was verified, the specimen was subjected to an effective confining stress approximately equal to the anticipated field conditions and thus reducing the possibility of excessive confining pressure.

  Pore-water was then forced through the specimen from the bottom to the top. Flow quantity was monitored using a calibrated manometer system. The test continued until steady state conditions were reached. Due to the orientation of the specimens during the tests, the values obtained reflect the soil matrix permeability in the vertical direction.

- Density, void ratio, degree of saturation, and porosity were reported with the Hydraulic Conductivity Test results.

- Triaxial Shear Tests - (ASTM D4767 and D2850) Cohesive soil samples were collected and submitted for triaxial shear testing in order to determine the strength of the samples under stress for both consolidated and unconsolidated soil conditions (CU and UU Tests). During the test, confining pressure is applied to the sides of the sample as it is loaded until failure. The stress/strain curves generated from these tests were used to determine the shear strength of each sample.

- One-Dimensional Consolidation Tests (ASTM D2435) - Cohesive soil samples were submitted for consolidation testing in order to predict soil consolidation settlement in the underlying soil units. The rate of consolidation was recorded as a normal load was applied to the soil. The slope of the consolidation plot reflects the compressibility of the soil.

- Laboratory Compaction Characteristics (ASTM D698) – Bulk samples of soil were submitted for evaluation of compaction characteristics. Laboratory compaction tests provide the basis for determining the percent compaction and molding water content needed to achieve the required engineering properties for fill and liner construction, and for allowing for controlled construction to assure that the required compaction and water contents are achieved.

Site Geology

Review of the results of the recent hydrogeologic investigation for the proposed horizontal expansion site and of information collected during previous investigations conducted within the expansion area indicate that the uppermost geology consists of glacial till and other unconsolidated deposits overlying dolomite bedrock. Geologic cross sections are provided in Drawing Nos. G5 through G16 and each geologic unit is discussed in detail in the following sections.

Bedrock

Across and adjacent to the expansion site, there are 3 borings which have been advanced into the top of the bedrock surface. Two borings, B-4-07/TB-1 and EB-10/B-6-07, were advanced approximately 10.9 feet and 4.8 feet, respectively into the bedrock surface during previous investigations. During the most recent investigation, boring B-08-18 was advanced approximately 3.6 feet into the bedrock surface (206 feet below ground surface).

Although no core samples of the bedrock were collected during the advancement of any of these borings, observation of drill cuttings and rock collected in split spoon samplers identified the rock as the Silurian dolomite. This is consistent with regional publications which indicate that the Silurian dolomite underlies the unconsolidated deposits across Lake County[2]. The bedrock and its relationship to the overlying glacial till, outwash deposits, and other unconsolidated deposits (referred to collectively as overburden) beneath the horizontal expansion area and adjacent permitted landfill to the south is illustrated in the geologic cross sections (**Drawing Nos. G14 through G16**).

Unconsolidated Deposits

Unconsolidated deposits overlie the bedrock and extend upward to the ground surface. From the surface downward, the deposits are the Peoria Silt, Wadsworth Formation, Intra-Till Sediments (within the Wadsworth Formation), Shallow Drift, Lower Till, lacustrine deposits inter-tongued with the Lower Till, and Basal Drift. The thickness of these unconsolidated deposits (using three continuously sampled borings in **Table 2.2-2**) ranges from approximately 215.3 feet (EB-10/B-6-07) to 202.4 feet (B-08-18). The individual geologic units which comprise the unconsolidated deposits are discussed below.

Peoria Silt. The Wisconsinan Age Peoria Silt was identified at the surface across the proposed horizontal expansion as a rich, modern topsoil. Across and adjacent to the proposed expansion site, the Peoria Silt ranges from approximately 0.0 to 4.6 feet thick with an average thickness of 1.07 feet, where present. In some locations, the Peoria Silt had been removed and was replaced with fill material.

Wadsworth Formation. The Wisconsinan Age Wadsworth Formation is a succession of fine grained, gray diamicton units located immediately beneath the Peoria Silt at the site and was encountered in all 31 of the continuously sampled borings which have been advanced within and near the expansion site. The top of the Wadsworth Formation was encountered at depths ranging from the ground surface (EB-10/B-6-07, B-10-18, and B-12-18) to 10.0 feet bgs (B-7-07/G178 where fill was encountered at the surface), with the top present at an average depth of 1.38 feet bgs. The top of the Wadsworth Formation was found at elevations ranging from 745.0 feet above MSL (B-06-18) to 724.8 feet above MSL (B-7-07/G178) with an average surface elevation of 737.1 feet above MSL. **Drawing No. G17** illustrates the top elevation of the Wadsworth Formation beneath the expansion area. **Photograph 2.2-9** illustrates a typical sample of the Wadsworth Formation.

The Wadsworth Formation was found to exhibit maximum and minimum thicknesses of 95.0 feet and 71.9 feet, respectively across the expansion site. The average thickness of this unit was calculated to be approximately 83.5 feet, which includes a weathered zone with brown to grayish brown or olive brown coloring that ranges in thickness from approximately 3.30 feet to 16.2 feet with an average thickness of approximately 9.08 feet. The weathered zone was determined in the field through visual observation of coloring/oxidation. **Drawing No. G18** illustrates an isopach of the Wadsworth Formation in the expansion area assuming pre-landfill conditions.

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[2]Csallany and Walton (1963), Horberg (1950), Larsen (1973), Leetaru et al. (2003), Thwaites (1927), Visocky, et al. (1985), Willman, et al. (1975), and Willman (1971).

Photograph 2.2-9
Typical section of the Wadsworth Till

Within the Wadsworth Formation, and consistent with regional publications[3], discontinuous lenses of silt, sand, and gravel were also identified. These discontinuous lenses of sediment (Intra-till Sediments) are not sufficiently saturated to serve as a water source. This is clearly illustrated through site-specific data (refer to geologic cross sections on **Drawing Nos. G5 through G16**) and observation during construction of the existing landfill units.

The laboratory test results from the proposed expansion area are provided in **Appendix I**. Also included in **Appendix I** are summary tables of geotechnical testing at the existing Zion Landfill. Soil testing results for the Wadsworth Formation indicates that it is generally classified by USCS standards as a silty clay (CL or CL-ML) based on grain size. The grain size analysis from the most recent investigation yielded an average of 2.1 percent gravel, 17.0 percent sand, 53.0 percent silt, and 28.0 percent clay. Based on 27 samples, the average liquid limit and plastic limit are 26.6 and 13.2, respectively. The soil exhibits an average plasticity index of 13.4. The average specific gravity of the Wadsworth Formation was found to be 2.75. The average dry density and porosity of the Wadsworth Formation were found to be 118.4 pounds per cubic foot and 0.31, respectively.

The laboratory measured vertical hydraulic conductivity of the Wadsworth Formation in the proposed expansion area from 6 samples collected during the most recent investigation ranges from 9.84×10^{-8} cm/sec to 2.38×10^{-8} cm/sec, with a geometric mean value of 4.85×10^{-8} cm/sec. The laboratory measured vertical hydraulic conductivity of the Wadsworth Formation at the existing Zion Landfill from samples collected during previous investigations

[3] Frye and Willman (1975), Hansel and Johnson (1996), Johnson, et al. (1985), and Larsen (1973).

was reported to range from 4.10 x 10^{-7} cm/sec to 3.63 x 10^{-8} cm/sec, with a geometric mean value of 1.04 x 10^{-7} cm/sec.

Horizontal hydraulic conductivity determined from previous slug testing of wells within the Wadsworth Formation at the existing Zion Landfill during previous investigations ranges from 9.53 x 10^{-8} cm/sec to 6.07 x 10^{-9} cm/sec with a geometric mean hydraulic conductivity of 1.85 x 10^{-8} cm/sec. A summary table of the slug testing performed at the existing Zion Landfill is provided in **Appendix H**.

As previously discussed, a weathered zone with an average thickness of 9.08 feet was identified within the Wadsworth Formation directly below the Peoria Silt, or at ground surface where the Peoria Silt is absent. Fracturing in the weathered zone was not identified at the site during the most recent hydrogeologic investigation. Additionally, the proposed excavation for the expansion (approximately 60 to 70 feet) will remove this weathered zone.

Discontinuous lenses of silt, sand, and gravel were also identified throughout the Wadsworth Formation beneath the site (Intra-Till Sediments). In most cases, these deposits exhibit a very similar color to the Wadsworth formation, but have also been identified in varying shades of brown. The thickness of these lenses at the site range from a fraction of a foot to as much as 8.5 feet.

Soil testing results for the Intra-Till Sediments during previous investigations indicate that it is generally classified by USCS standards as a silty sand (SM). The grain size analysis yielded an average of 24.3 percent gravel, 52.0 percent sand, 16.9 percent silt, and 6.9 percent clay for the discontinuous deposits. The results of these tests are provided in **Appendix I**.

A total of 10 Slug tests, including 5 falling head tests and 5 rising head tests, were performed at 5 piezometers screened within the Intra-Till Sediments in the subsurface below the horizontal expansion site during the most recent investigation. The horizontal hydraulic conductivities of the deposits obtained from slug testing range from 3.46 x 10^{-3} cm/sec to 2.66 x 10^{-6} cm/sec with a geometric mean of 1.26 x 10^{-4} cm/sec. The horizontal hydraulic conductivities of the deposits obtained from the previous investigation at the adjacent Site 2 East Expansion had a geometric mean of 2.85 x 10^{-5} cm/sec. The results of the slug test analysis, including a summary table and the data plots, are provided in **Appendix H**.

<u>Shallow Drift</u> The Shallow Drift underlies the Wadsworth Formation and is recognized as a zone of inter-tongued gray to dark gray silt, sand, and gravel deposits. The top of the Shallow Drift was encountered at depths ranging from 72.7 feet bgs (B-07-18) to 95.0 feet bgs (EB-10/B-6-07) with an average depth of 84.98 feet bgs and at elevations ranging from 659.9 feet above MSL (B-07-18) to 643.8 feet above MSL (B-13-18) with an average surface elevation of 653.6 feet above MSL. **Drawing No. G19** illustrates the top elevation of the Shallow Drift beneath the expansion area. **Photograph 2.2-10** illustrates a typical sample of the Shallow Drift.

Photograph 2.2-10
Typical section of the Shallow Drift

Soil testing results for the Shallow Drift from 3 samples submitted during the most recent investigation indicate that it is generally classified by USCS standards as a silty sand (SM) where tested. The grain size analysis yielded an average of 1.5 percent gravel, 80.7 percent sand, 16.4 percent silt, and 1.5 percent clay. The results of these tests are provided in **Appendix I**. Also included in **Appendix I** is a summary table of geotechnical testing for the existing Zion Landfill.

A total of 42 Slug tests, including 21 falling head tests and 21 rising head tests, were performed at 17 piezometers screened within the Shallow Drift Aquifer in the subsurface below the horizontal expansion site during the most recent investigation. A second round of tests were run at 4 locations that exhibited quick recoveries. The horizontal hydraulic conductivities of the deposits obtained from slug testing range from 9.72×10^{-2} cm/sec to 1.87×10^{-6} cm/sec with a geometric mean of 3.57×10^{-4} cm/sec. The horizontal hydraulic conductivities of the deposits obtained from the previous investigation at the adjacent Site 2 East Expansion had a geometric mean of 1.77×10^{-4} cm/sec. The results of the slug test analysis, including a summary table and the data plots, are provided in **Appendix H**.

<u>Lower Till</u> Beneath the Shallow Drift is another till unit which has been identified within this report as the Lower Till. The Lower Till is identified at the site as being similar to the Wadsworth Formation in that it is a succession of fine grained, gray diamicton units (refer to **Photograph 2.2-11**). The Lower Till ranges in thickness from 72.8 feet to 100.5 feet with an average thickness of approximately 84.5 feet across the site. It should be noted that the thickness of the Lower Till includes inter-tonguing silty clay lacustrine deposits. These lake deposits are identified at the site as gray silty clay deposits that are predominantly fine grained and are distinguishable from the more massive till units by distinct bedding structures. The top of the Lower Till was encountered at depths ranging from 133.6 feet bgs (EB-10/B-6-07) to 101.5 feet bgs (B-4-07/TB-1) with an average depth of 117.0 feet bgs and at

elevations ranging from 636.7 feet above MSL (B-4-07/TB-1) to 610.5 feet above MSL (EB-10/B-6-07) with an average surface elevation of 620.9 feet above MSL.

Photograph 2.2-11
Typical section of the Lower Till

Soil testing results from 5 samples of the Lower Till deposits tested during previous investigations indicate that they are generally classified by USCS standards as a silty clay (CL or CL-ML). The grain size analysis yielded an average of 4.6 percent gravel, 28.3 percent sand, 42.5 percent silt, and 24.6 percent clay based on geotechnical testing of 4 samples. The average liquid limit and plastic limit are 21.0 and 13.3, respectively. The soil exhibits an average plasticity index of 7.7. The dry density and porosity of the Lower Till were measured to be 126.7 and 0.26, respectively. The results of the laboratory testing are provided in **Appendix I**.

<u>Basal Drift</u> The Basal Drift underlies the Lower Till and lacustrine deposits, and overlies bedrock. It is recognized as a zone of inter-tongued gray to dark gray silt, sand, and gravel deposits (refer to **Photograph 2.2-12**). The top of the Basal Drift was encountered at depths ranging from 195.0 feet bgs (B-08-18) to 213.7 feet bgs (EB-10/B-6-07) with an average depth of 203.6 feet bgs and at elevations ranging from 547.9 feet above MSL (B-08-18) to 530.4 feet above MSL (EB-10B-6-07) with an average surface elevation of 538.2 feet above MSL.

Falling and rising head slug tests performed on 1 well installed within the Basal Drift during the most recent investigation indicated a geometric mean hydraulic conductivity of 2.51×10^{-4} cm/sec. Slug testing and data printouts from the recent tests are provided in **Appendix H**.

Photograph 2.2-12
Typical section of the Basal Drift

Site Hydrogeology

This section presents a discussion of the hydrogeology associated with the Shallow Drift Aquifer, which is the predominant water bearing geologic unit below the proposed expansion area. Potentiometric maps were created for the Shallow Drift Aquifer (Uppermost Aquifer) utilizing groundwater data collected from 1st Quarter 2019 through 1st Quarter 2021. **Drawing Nos. G21 through G42** depict the potentiometric surface of the Shallow Drift Aquifer across the eastern portion of the facility, including the expansion area. Monitoring wells G168, G169, G191, and G201 through G206 are screened in the upper portion of the Shallow Drift and have therefore been included in the creation of the potentiometric maps of the Upper Shallow Drift (**Drawing Nos. G21, G23, G25, G27, G29, G31, G33, G35, G37, G39, and G41**). **Drawing Nos. G5 through G16** show geological cross-sectional relationships across the horizontal expansion site. Boring logs and well construction diagrams for all monitoring wells and piezometers used in the creation of potentiometric maps are contained in **Appendix G.**

Analysis of the potentiometric maps developed for the upper portion of the Shallow Drift Aquifer across the site indicates that groundwater flow within this unit is generally to the east and to the north (with an additional slight southerly component) with an average easterly horizontal gradient of approximately 0.000386 measured across the lower portion of the horizontal expansion site and an average northerly horizontal gradient of approximately 0.002401 measured across the upper portion of the horizontal expansion site (refer to **Appendix G**). The maximum and minimum horizontal gradients for the upper portion of the Shallow Drift Aquifer were measured to be 0.00410 and 0.0000462, respectively.

Analysis of the potentiometric maps developed for the lower portion of the Shallow Drift Aquifer across the site indicates that groundwater flow within this unit is generally to the east with an average easterly horizontal gradient of approximately 0.00292 measured across the central portion of the horizontal expansion site (refer to **Appendix G**). The maximum and minimum horizontal gradients for the lower portion of the Shallow Drift was measured to be 0.00523 and 0.00184, respectively.

By multiplying the average measured horizontal gradient in the upper portion of the Shallow Drift Aquifer in its predominant easterly flow direction (0.000386) by the geometric mean hydraulic conductivity of the Shallow Drift Aquifer (3.57×10^{-4} cm/sec), a Darcy Velocity of 0.04 m/yr (0.13 ft/yr) was calculated. The effective porosity value for the Shallow Drift Aquifer was 0.367. This was derived by converting laboratory data for total porosity of the Shallow Drift Aquifer deposits (provided in Appendix I of this application) to effective porosity based on empirical data provided by Sara (1994) as shown in Appendix P. Using this porosity, the seepage velocity in the Shallow Drift Aquifer is approximately 1.09 m/yr (3.57 ft/yr) (refer to **Appendix G**).

At several locations, it was possible to calculate a vertical groundwater flow gradient through the Wadsworth Till and the Shallow Drift (refer to **Appendix G** for a summary of vertical gradient calculations). The average calculated vertical gradient through the Wadsworth Till is 0.83 with minimum and maximum calculated gradients of 0.25 and 1.20, respectively. The average calculated vertical gradient through the Shallow Drift is 0.40 with minimum and maximum calculated gradients of 0.14 and 0.83, respectively. At each of these locations, the gradient was in the downward direction. A vertical gradient was also calculated through the Lower Till using piezometers P-08-18D and P-08-18SD. The calculated vertical gradient through the Lower Till at this location is 0.006 in the downward direction (refer to **Appendix G**).

Uppermost Aquifer

The uppermost aquifer at the site was identified per IEPA definitions. 35 Ill. Admin. Code, Section 810.103 defines an aquifer as:

> *“Aquifer” means saturated (with groundwater) soils and geologic materials which are sufficiently permeable to yield economically useful quantities of water to wells, springs, or streams under ordinary hydraulic gradients and whose boundaries can be identified and mapped from hydrogeologic data.*

The same regulation defines the uppermost aquifer as the following:

> *"Uppermost aquifer" means the first geologic formation above or below the bottom elevation of a constructed liner or wastes, where no liner is present, which is an aquifer, and includes any lower aquifer that is hydraulically connected with this aquifer within the facility’s permit area.*

Due to its hydrogeologic properties, fairly continuous nature, location below the base of the proposed liner, and use as a potable water source in the vicinity of the site, the Uppermost Aquifer below the proposed expansion has been determined to be the Shallow Drift Aquifer. It should be noted that the Shallow Drift Aquifer is also the Uppermost Aquifer at the existing site.

620 Groundwater Classification Evaluation

Groundwater classifications and standards are established in 35 Ill. Adm. Code, Part 620 Groundwater Quality. This section defines the groundwater classification for each of the geologic units identified at the proposed site, using the criteria specified in the regulations.

By default, and per regulation, groundwater is classified as Class I (Potable Resource Groundwater) unless it can be classified Class II (General Resource Groundwater), Class III (Special Resource Groundwater), of Class IV (Other Groundwater). As a result, all of the geologic units identified in the Geology portion of this section were assumed to be Class I and were subsequently evaluated to determine whether they can be otherwise classified. The results of the evaluation found that groundwater within the Intra-till Sediments, Lower Till, and Basal Drift are classified as Class I and groundwater within the Peoria Silt, Wadsworth Till, and Lower Till are classified as Class II. None of the groundwater could currently be classified as Class III or Class IV. **Table 2.2-4** summarizes this evaluation.

TABLE 2.2-4
SUMMARY OF EVALUATION OF GROUNDWATER CLASSIFICATION

	Section 620.210 Criteria					
Geologic Unit	**(a)1**	**(a)2**	**(a)3**	**(a)4 A & B**	**(b)**	**Result**
Peoria Silt	Within 10 feet of the ground surface					Class II
Wadsworth Till	No	No	No	NE, No	NA	Class II
Intra-till Sediments	No	Yes	No	NE, Yes	NA	Class I
Shallow Drift	No	Yes	No	NE, Yes	NA	Class I
Lower Till	No	No	No	NE, No	NA	Class II
Basal Drift	No	Yes	No	NE, Yes	NA	Class I

Notes:
NE = Not evaluated; NA = Not Applicable

Geologic and Hydrogeologic Conclusions

Based on the findings of this investigation, the geologic and hydrogeologic site conditions present at the proposed site are suitable for the development of a landfill and will serve to protect the public health, safety, and welfare. The following conclusions can be made concerning the geologic and hydrogeologic conditions at the site.

- A significant amount of hydrogeolgic investigation activities have been conducted at the existing landfill prior to the most recent investigation. Data collected during the previous hydrogeologic investigation activities was obtained through the advancement of over 260 borings (over 110 of which were continuously sampled) and the installation of over 200 monitoring wells.

- The most recent site investigation included a review of previous site investigations and the advancement of an additional 15 borings. The geology beneath the Site 2 North Expansion, as characterized by the 15 borings, is consistent with the geology encountered beneath the existing facility and the geologic setting which is described in regional publications[4], providing additional support to the findings of this investigation. The continuity observed from boring to boring demonstrates that the investigation activities were adequate in extent to verify the geologic and hydrogeolgic features beneath the site. Fourteen of the 15 borings were converted to piezometers and 9 additional nested piezometers were installed to supplement the hydrogeologic information for the site.

- A low-permeability cohesive soil (Wadsworth Formation) is present across the proposed site which will separate the footprint of the proposed Site 2 North Expansion from the uppermost aquifer. This low permeability cohesive soil (clayey till) has an average thickness of approximately 83.5 feet in the expansion area with maximum and minimum thicknesses of 95.0 feet and 71.9 feet, respectively. Field and laboratory test results and field observations indicated that this soil will effectively restrict vertical and horizontal movement of groundwater and will serve as an additional environmental safeguard at the proposed expansion. The average thickness of the Wadsworth Formation includes discontinuous lenses of silt, sand, and gravel (Intra-Till Sediments) which are contained within the till.

- As discussed in the design report, the engineered liner system beneath the expansion area will include 5 feet of recompacted clay and a high-density polyethylene (HDPE) liner. Such a liner exceeds the requirements of the U.S. EPA and has been accepted by the Illinois Environmental Protection Agency (IEPA) and other experts in the landfill field as providing a high level of environmental safety. The natural clay that is present on the site below the liner system will act as a second, natural liner system for the landfill expansion.

- In addition to following the requirements of the City of Zion Pollution Control Facility Siting Ordinance, the investigation report was created in general accordance with the requirements contained in 35 Ill. Admin. Code, Section 811.315, 812.314, and 812.315. These regulations specify the necessary content of a hydrogeologic investigation report submitted to the IEPA as part of an application for a landfill expansion permit.

- The proposed Site 2 North Expansion is located in an area that is classified by Berg and Kempton (1984) as Map Unit E (low aquifer sensitivity with respect to land burial of municipal solid waste) with uniform, relatively impermeable silty or clayey till at least 50 feet thick. The site is also located in an area that has been classified by Larson (1973) as being geologically optimal for the development of a landfill within Lake County.

- Based on discussions with the site operator and CQA Engineer, the geologic interpretations that have been established within this report are consistent with the conditions observed during the development of large-scale excavations at the existing

[4]Csallany and Walton (1963), Frye and Willman (1975), Hansel and Johnson (1996), Horberg (1950), Johnson, et al. (1985), Kammerer, et al. (1998), Larsen (1973), Leetaru et al. (2003), Piskin, et al. (1975), Thwaites (1927), Visocky, et al. (1985), Willman, et al. (1975), and Willman (1971).

facility. The site-specific observations verify the thickness of the clayey till and discontinuous nature of the intra-till sediments as described within this analysis. IEPA review and approval of construction documentation reports supports this as well.

- The hydrogeologic conditions at the site will allow a comprehensive groundwater monitoring system to be implemented which will be able to adequately verify groundwater resources are being protected.

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SECTION 2.3

Design

2.3 DESIGN

Introduction

Zion Landfill, Inc. owns and operates the Zion Landfill (Facility) in the City of Zion, Illinois. Capacity of the existing Site 2 Landfill (Landfill) is projected to be depleted around the year 2028. To provide continued, uninterrupted operation of the Landfill, Zion Landfill, Inc. is proposing to expand the Landfill to the North (Site 2 North Expansion or Expansion).

This text provides an overview of key design features and evaluations of the proposed Site 2 North Expansion and is supplemented by the referenced design drawings, appendices, and associated text sections within this application to the IEPA.

Site 2 Landfill

The existing Facility consists of two older units that have ceased acceptance of waste and are closed (Site 1 Phase A and Site 1 Phase B), as well as the currently active unit referred to as the Site 2 Landfill (Landfill). The currently active Site 2 Landfill, which is proposed to be expanded as described in this application, includes an older, closed section (Old Site 2), as well as two prior expansion areas constituting the open, operating portion of the Facility. The proposed Site 2 North Expansion that is the subject of this application will be the third expansion of the Site 2 Landfill. The Landfill is permitted by the Illinois IEPA (Site No. 0978020002).

The original area of the Site 2 Landfill, referred to as Old Site 2, is a non-hazardous solid waste unit that was regulated under 35 IAC, Part 807. Old Site 2 commenced landfilling operations on December 23, 1981, pursuant to IEPA Permit No. 1980-24-DE. In 1993, a final cover system was constructed over the site. Siting approval for the first Site 2 Expansion (initially identified as Site 3 at that time) was granted by the Zion City Council on April 17, 1995 which approved a new landfill unit east of Old Site 2 including a “piggyback” onto the eastern portion of Old Site 2. The Site 2 Expansion was originally permitted under 35 IAC, Part 812, Subparts A and C, and is now regulated under 35 IAC, Part 811 regulations, which meet or exceed Subtitle D Federal landfill regulations.

A second expansion, referred to as the Site 2 East Expansion, included vertical and an approximate 26.5-acre horizontal expansion to the east of the previous Site 2 Expansion footprint. The initial phase of the Site 2 East vertical expansion was permitted on June 3, 2011, with the remainder of the expansion approved for development on June 13, 2014. The Site 2 East Expansion is regulated under 35 IAC, Part 811 regulations.

Site 2 North Expansion

The proposed Site 2 North Expansion includes a horizontal and vertical component. The proposed horizontal Expansion will advance the existing Landfill to the north, expanding the waste unit boundary of the existing Landfill by 65.6 acres and increasing the overall facility boundary 124 acres to the north. The proposed vertical Expansion will tie into the Site 2 East Expansion portion of the existing Landfill by vertically expanding over its north sideslopes. **Figure 2.3-1** provides a plan-view representation of the Expanded Landfill. **Figure 2.3-2** provides a cross-section representation of the Expanded Landfill.

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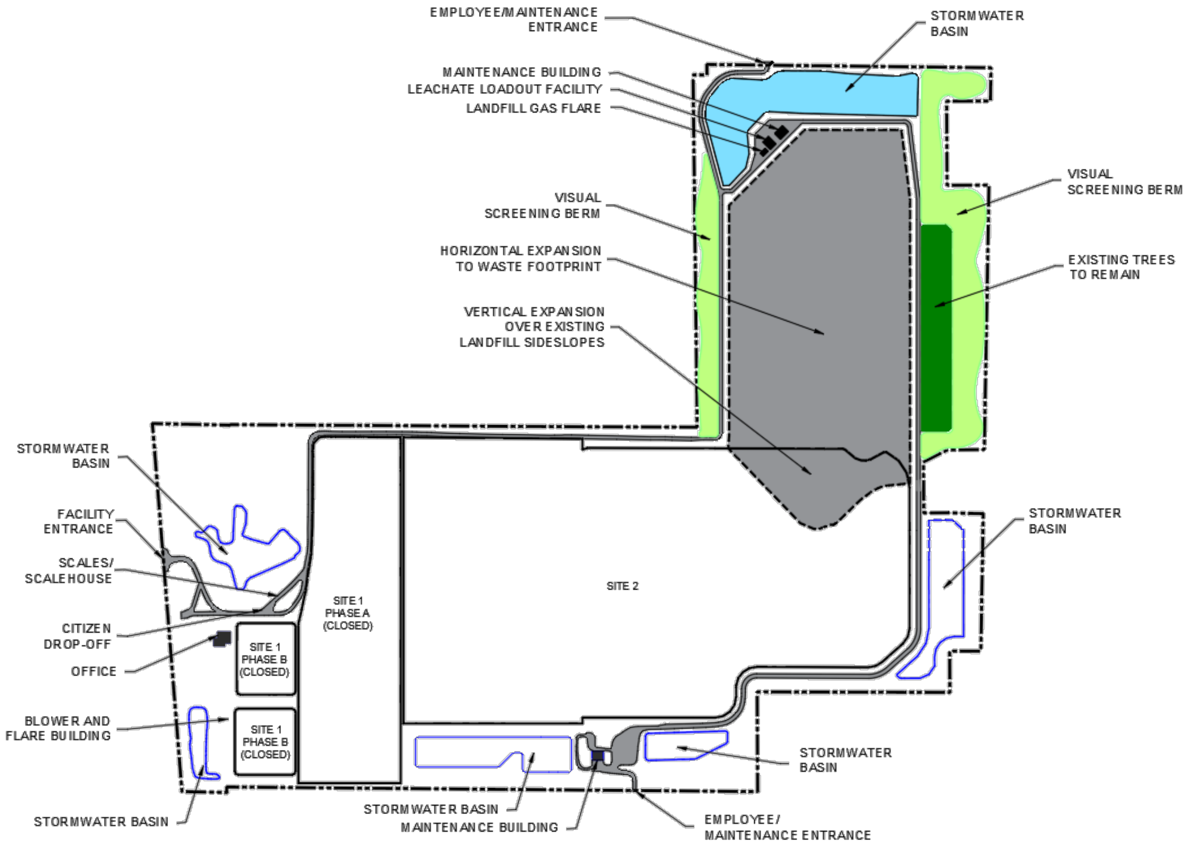


Figure 2.3-1
Proposed Expansion Facility Layout

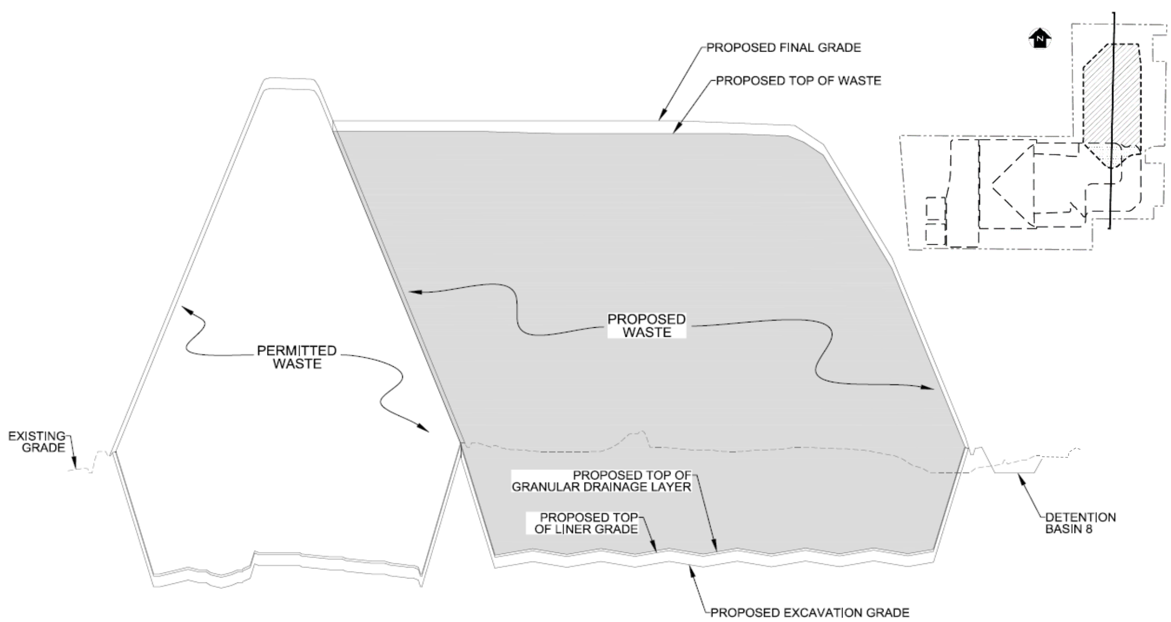


Figure 2.3-2
Cross Section of Expanded Landfill

The Expansion will add approximately 12.7 million airspace cubic yards of waste disposal capacity (approximately 14 million tons) to the existing Landfill, which is anticipated to extend the life of the existing Landfill into 2044 assuming historical annual disposal volume and projected growth in annual disposal volumes is unchanged.

Most of the existing infrastructure supporting the landfill will remain in place as part of the expansion, including the landfill entrance, citizen drop-off area, administrative buildings, landfill gas processing area, maintenance shop, etc. A new leachate tank, landfill gas flare, and maintenance shop will be constructed to support the expansion. The leachate collection system, landfill gas collection and control system, and stormwater management system will be expanded to capture the footprint of the expanded landfill. Each of these features are further described in subsequent text.

Proposed Landfill Design Overview

The proposed Expansion design incorporates numerous extensive environmental safeguards. The design has been modeled based on site-specific conditions to ensure that it works in conjunction with its geologic and hydrogeologic conditions and facility location.

This proposed design includes modern landfill design features, including a composite liner system, a leachate collection and removal system, and a composite final cover. These design features have been successfully used at the existing Zion Landfill and many other modern landfills, have been well studied, and are known to be protective of the public health, safety, and welfare. A brief summary of each is described below:

1. *Composite Liner System*. The Expansion will utilize a composite liner system consisting of a minimum 5-foot-thick compacted cohesive soil liner with a maximum permeability 1×10^{-7} cm/sec and a 60-mil high density polyethylene (HDPE) geomembrane.

   This liner thickness significantly exceeds the regulatory standard of a 3-foot compacted clay liner system. In addition, though not required by regulations, the Landfill's composite liner will be further enhanced in the leachate collection sump areas. The composite liner system in these areas will include a geosynthetic clay liner (GCL) between the 60-mil HDPE geomembrane and 5-foot-thick compacted clay liner, in addition to a double-sided geocomposite drainage layer and 60-mil HDPE geomembrane below the compacted clay liner (see **Drawing D16**). This design significantly exceeds the federal and state regulations, which require only one 60-mil HDPE geomembrane.

   The composite liner system will effectively prevent the release of potential hazards from the Landfill. The liner system has been computer modeled, and the computer analysis demonstrates that the proposed Landfill will not impact existing or future groundwater quality (see **Section 2.7**).

2. *Leachate Collection System.* The Expansion design incorporates a leachate collection system consisting of a one-foot-thick permeable granular drainage layer placed above the composite liner on the Landfill floor and sideslopes. The leachate collection layer drains to collection points located along the perimeter of the waste boundary. Leachate will be removed from these collection points and properly managed.

3. *Final Cover System*. The final cover system of the Expansion consists of a low-permeability layer to inhibit precipitation from entering the Landfill and a protective soil layer used to maintain the long-term integrity of the cap. The low-permeability layer will include a 40-mil linear low-density polyethylene (LLDPE) geomembrane.

   The geomembrane will be underlain by a 2-foot-thick compacted cohesive soil layer with a maximum constructed permeability of 1 x 10^{-5} cm/sec. A double-sided geocomposite drainage net will overlay the geomembrane to drain infiltrated water away from the low-permeability layer. A protective soil cover layer will be placed over the geocomposite and will include a minimum of 2.5 feet of protective cover soil and six inches of vegetative cover soil. The Site 2 North Expansion will have a maximum slope of 4H:1V. In order to minimize the potential for erosion, the final slopes of the Landfill will be vegetated.

4. *Landfill Gas Collection System*. The Expansion will have an active landfill gas management system to collect and control gases generated through the natural decomposition of waste. The collected landfill gas will be flared or beneficially used once a sufficient amount of landfill gas is available.

Location of Landfill Design

Prior to developing the Expansion design, the property was reviewed with respect to location standards to determine whether the area was suitable for landfill development. As detailed in **Section 2.1** of this application, Illinois landfill regulations contain standards that restrict where landfills may be developed (35 IAC, Sections 811.102 and 811.302). Federal regulations and statutes also contain location requirements. The collective purpose of each of these location standards and requirements is to protect public health, safety, and welfare; the environment; and the structural integrity of the engineered landfill.

The selected location of the proposed Expansion will comply with all applicable federal, state, and local site location standards. **Section 2.1** provides a detailed description of each location standard and a demonstration that the standard is met. **Drawing D2** and **Drawing G2** shows the location of the proposed facility and demonstrates that the facility falls outside the applicable setback distances. **Appendix F** supplements these drawings when other maps, such as floodplain maps, are more appropriate to display setback compliance.

Designed Integration with Existing Facility

Existing Infrastructure

The existing scalehouse, haul roads, office, maintenance building, detention basins, leachate storage tanks, facility entrances, and other facilities will continue to be used as part of the facility Expansion. Additional infrastructure will be added as part of the proposed Expansion and will include:

- ❑ An additional maintenance building;
- ❑ An additional secondary entrance for employee and ancillary vehicles;
- ❑ Parking;
- ❑ Additional perimeter roads;
- ❑ Leachate storage and loadout facilities;
- ❑ Stormwater management basins;
- ❑ Landfill gas processing facilities; and

- ❑ Staging areas for equipment and supply storage.

See **Design Drawings** for the location of all structures associated with the Expansion.

Utilities

Utilities used to manage the facility will include, at a minimum:

- ❑ Electrical service to office/maintenance building, leachate/condensate pumps, landfill gas flare station, and scalehouse.
- ❑ Phone service to office/maintenance building and scalehouse.
- ❑ Two-way radio or cellular communication between supervising equipment operator(s), General Manager, and office.
- ❑ Water supply to the office and maintenance buildings.
- ❑ Sanitary service to the office and maintenance buildings.

Utilities will be provided and maintained at the site during the operating and post-closure care periods of the landfill for safety and compliance with the requirements of 35 IAC 811.

Physical Connection to Existing Landfill

The proposed Expansion will build vertically over a portion of the permitted Site 2 East Expansion and expand the waste footprint horizontally to the north of the Site 2 East Expansion.

A continuous composite liner and leachate collection system (both described in subsequent text) will be developed between the constructed Landfill and Expansion area, such that all areas of Landfill development have these underlying environmental controls and design features. Refer to **Drawing D17** for details depicting transitions between the existing Landfill and the Horizontal Expansion Area.

Hydrogeologic Considerations in Landfill Design

The design of the Expansion is supplemented by existing geologic and hydrogeologic features to provide a high level of environmental safety. An extensive site investigation was completed at the facility prior to developing the Landfill Expansion design in order to characterize both the geology and hydrology of the subsurface geologic units. This investigation included both an examination of regional geology and hydrogeology, as well as a site-specific exploration program. The exploration program included detailed logging of soil and rock samples, geotechnical laboratory testing, installation of monitoring wells, performance of field hydraulic conductivity tests, a coal mine reconnaissance, water level collection, and data evaluation.

The Wadsworth Formation, a low-permeability cohesive soil that has existed for over 10,000 years, is present across the proposed Site and will separate the footprint of the proposed Landfill Expansion from the uppermost aquifer. Field and laboratory test results and field observations indicate that this soil will effectively restrict vertical and horizontal movement of groundwater and will serve as an additional environmental safeguard at the proposed Expansion. The Wadsworth Formation contains a weathered portion directly below the Peoria Silt that has the potential to exhibit fractures within the upper 20 feet, although no fractures were identified at the site during the most recent investigation. The proposed excavation for the Expansion (approximately 60 feet) will remove this weathered zone. Additionally, loading stress caused by the Landfill will close any fractures within this zone. Thus, Wadsworth formation will provide a geologic barrier between the landfill and the uppermost aquifer that will provide very long-term protection of the environment.

Refer to **Section 2.2** for a complete description of geologic setting and to **Section 2.7** for the results of contaminant transport modeling for the Expanded Landfill. The Environmental Monitoring Program is described within **Section 2.8**.

Landfill Composite Liner System

An engineered composite liner system will be present in the proposed Expansion. The composite liner system will be constructed at the bottom and sides of the Expansion to contain the waste materials and prevent contaminants from leaving the Expansion and impacting groundwater. The composite liner will consist of a compacted cohesive soil liner overlain by a geomembrane (plastic) liner. The soil liner will consist of a minimum 5-foot-thick layer of recompacted cohesive soil with a maximum permeability of 1 x 10^{-7} cm/sec. The geomembrane will be a 60-mil HDPE liner. Additionally, a geocomposite clay liner will be installed in critical areas in the Expansion, namely the leachate collection sumps.

The liner system of both the Site 2 Landfill Expansion liner system and subsequent Site 2 East expansion have been permitted and constructed utilizing the same design. It is noted that the recompacted soil liner thickness exceeds the typical three-foot liner thickness used at other landfill facilities within Illinois.

The proposed liner system for the Expansion has been designed to function for the entire design period, pursuant to Section 811.306(c). The low-permeability component of the proposed liner system consists of low permeability till soils and are generally clayey soils that have survived for thousands of years. Long-term laboratory testing of HDPE geomembranes indicate that the service life of geomembranes is several hundred years (see **Appendix K**). In addition, **Appendices J** and **K** provide a demonstration that the proposed liner system will be stable (i.e. will function) under both short-term and long-term conditions. **Appendix K** includes a demonstration that the composite liner system will perform better than a five-foot clay liner system.

Low-Permeability Earth Liner

The low-permeability earth liner for the Expansion will meet regulatory requirements by providing a minimum 5-foot layer of compacted cohesive soil with a maximum hydraulic conductivity of 1 x 10^{-7} cm/sec. The earth liner thickness exceeds typical three-foot liners as an additional environmental safeguard.

It is anticipated that the low-permeability earth liner will be constructed of Wadsworth formation soils due to the favorable physical properties for construction and low hydraulic conductivity. As discussed in **Section 2.2** of this Application, the native soils have permeabilities that are less than the 1 x 10^{-7} cm/sec requirement.

Roots, boulders, debris, and other deleterious material will be removed from the soil prior to compaction. Frozen soil will not be used for construction and liner material will not be placed on frozen ground. Each soil layer will be worked sufficiently to break down oversized clods, and obtain acceptable moisture and density requirements, as defined by the CQA Plan. Earth Liner material, placement, and compaction standards are provided in the CQA Plan located in **Appendix O**.

Geomembrane

The geomembrane will be installed above the Earth Liner by personnel experienced in liner installation. The geomembrane liner will consist of panels of 60-mil textured HDPE. Geomembrane materials, installation, seaming, and testing will be performed in accordance with the CQA Plan located in **Appendix O**.

The geomembrane panels will be arranged to minimize the number of field seams. It is assumed that the geomembrane panels will be 22.5 feet wide by 400 feet long (panel lengths and widths may vary by manufacturer's specifications at the time of construction). **Drawing D9** provides a conceptual geomembrane panel layout for the Landfill. The actual constructed layout of the geomembrane panels will be provided with each cell construction certification report. Penetrations through the geomembrane liner system are not proposed or anticipated.

The geomembrane liner subgrade will be prepared to be smooth and free of rocks, stones, roots, sharp objects or other undesirable debris. In order to maintain stable side slopes, the geomembrane liners will be anchored beyond the limits of the waste into the anchor trenches as shown on **Drawing D15**.

The geomembrane liner will also be protected from sharp items in the waste by the granular drainage blanket which will serve as part of the leachate collection system on the Landfill floor and sideslopes.

Based on current technology, a dual fusion wedge weld is generally the preferred seaming method to join panels and will generally be used for areas except at sumps, corners, or other irregular areas where an extrusion weld is necessary. Extrusion welds are also highly effective welds and are anticipated to be used to repair destructive sample locations, and any repair areas.

The geomembrane will have sufficient strength and durability to function for the design period under the maximum expected loading imposed by the waste and equipment and stresses imposed by settlement, temperature, construction, and operation, pursuant to Section 811.306(e). Calculations demonstrating the strength and durability of the HDPE liner are provided in **Appendix J**. Demonstration that HDPE is compatible with the Landfill environment is provided in **Appendix K**.

Geosynthetic Clay Liners (GCLs)

Within each leachate collection sump, a GCL will be beneath the 60-mil HDPE geocmembrane and placed on top of the 5-foot thick recompacted cohesive soil liner as shown on **Drawing D18**. GCL materials and installation will comply with the CQA Plan in **Appendix O**.

CQA Documentation

Liner construction, documentation, and certification will be performed in accordance with the CQA Plan contained in **Appendix O** of this Application. A CQA Officer will supervise and be responsible for all inspections and testing. The CQA Officer will be an independent licensed Professional Engineer. A construction acceptance report will be prepared under the direct supervision of the CQA Officer and submitted to the IEPA after completion of each major phase of construction.

Leachate Management

Origin of Leachate

Leachate is any liquid that has contacted waste. Leachate can come from several sources, including the biological breakdown of waste or the movement of infiltrated moisture, such as rainwater, through the waste. Leachate generation will vary depending on the composition and moisture content of the incoming waste (i.e., dry waste will absorb more water than wet waste). Most of the leachate in a conventional landfill stems from precipitation that falls on the active area of the landfill, or from precipitation that percolates through daily/intermediate cover. The low permeability final cover employed at the Expansion will essentially eliminate long-term leachate generation on sections of the landfill that have been capped.

The rate of leachate generation and the composition of the leachate are influenced principally by the following factors:

1. The availability and potential for infiltration or seepage of water into the landfill.

2. The physical and chemical characteristics of the waste (i.e. the moisture content, absorptive capacity, and solubility of the waste).

3. The environment in which the biological decomposition process takes place (i.e. pH, availability of oxygen and temperature).

Municipal solid waste landfill leachate typically contains the following chemicals in order of decreasing concentrations: 1) dissolved and suspended solids including salts (i.e. sodium chloride), sulfates, and sodium bicarbonate; 2) metals (principally iron and zinc); and, 3) organic compounds. The waste decomposition process will also yield methane, carbon dioxide, and traces of other gases. Some heat will be generated as the waste decomposes.

The rate of decomposition in a landfill depends on the type of waste and the landfill environment in which the waste is present, with moisture content being one of the primary factors. Food wastes typically decompose first, followed by paper, wood, textiles, and discarded un-stabilized plastics. Microbes that are initially present in the waste or introduced with the materials used as daily cover will initiate the aerobic portion of the decomposition process. Inert materials (soils, coal combustion byproducts, grit, some plastics, and some construction/demolition debris) which do not readily degrade will essentially remain unchanged by the decomposition process.

Overview of Leachate System

The Expansion will include a leachate collection system to collect and remove leachate for treatment and disposal. The vertical expansion area will be underlain by the currently permitted leachate collection system at the facility. The existing leachate collection system has been evaluated for adequacy for the vertical expansion (see **Appendix K**). This system will be expanded to incorporate the horizontal expansion Area as cell development progresses within the Expansion. Though the Facility has historically been permitted for leachate recirculation, leachate will not be recirculated within the expanded landfill and is therefore not included in the Expansion design.

Throughout the Landfill, the leachate collection system will consist of a highly permeable leachate drainage layer overlaying the entire base of the Landfill and a system of leachate collection pipes, collection sumps, collection risers and cleanout risers. The drainage layer material will have a minimum hydraulic conductivity of 1.0 x 10^{-1} cm/sec, which will facilitate the flow of leachate across the base of the Landfill.

A nonwoven geotextile will be installed above the entire drainage layer. The purpose of this geotextile is to serve as a filter to the leachate as it enters the drainage layer. This geotextile minimizes the potential for clogging within the drainage layer. The geotextile seams will be overlapped, heat bonded, and/or field sewn as required by the CQA Plan (see **Appendix O**).

Once leachate passes through the geotextile filter, it will flow by gravity through the granular drainage material, which is anticipated to be coarse sand or pea gravel. Leachate collection lines consist of perforated HDPE pipe situated in a gravel or stone envelope. The base composite liner for each cell in the expansion is designed to slope at a minimum of 2.0 percent toward the leachate collection pipe. The maximum horizontal distance from the leachate drainage divide to the collection point is approximately 192 feet.

Once leachate reaches the collection pipe, the collection pipe is designed to flow by gravity to sumps (collection points) located at the base of the landfill sidewalls. The leachate collection pipes will be sloped at a minimum of 1.0 percent to promote drainage within the pipes to the leachate header pipes and leachate collection sumps.

Access to the sumps will be provided by dual risers which will be placed on the landfill sidewalls and will extend beyond the waste boundary. The riser pipes will extend from the collection sumps to the edge of the waste footprint, where the point of extraction is accessible. Pumps will be placed within the risers to remove leachate from the landfill and will be equipped with a leachate level detection system for monitoring leachate levels. A force main will be used to convey leachate from the sumps to the leachate storage tank. All leachate piping outside of the waste limits will be dual-contained.

The location and details of the components of the leachate collection system are shown on **Drawings D10**, **D15**, **D16**, **D17**, **D18**, and **D19**. Material and installation specifications for the various components are provided in the CQA Plan in **Appendix O**.

Safeguards of the Leachate Collection System

The leachate collection system for the proposed Expansion is appropriately designed and provides the following design safeguards:

1. The highly permeable granular drainage layer will have a minimum hydraulic conductivity of 1.0 x 10^{-1} cm/sec and be a minimum of 12-inches thick across the floor of the Landfill. This drainage layer will promote flow to the collection pipes, minimizing the leachate head above the HDPE composite liner system.

2. The collection pipes are capable of handling volumes far exceeding the maximum estimated leachate flow volumes for the Expansion.

3. The leachate collection cleanout risers will allow access to all points along the collection lines for cleaning out the pipes and back-flushing, if necessary.

4. The granular pipe envelope will serve as a conduit to other collection points in the unlikely event that a temporary clog or localized pipe failure occurs.

5. All of the components of the leachate collection system will be constructed of materials that are chemically resistant to the anticipated composition of leachate.

Maintaining the Leachate Collection System

The leachate collection system of the Expansion has been designed to efficiently collect leachate throughout the operating life, post-closure care period, and beyond. The system is designed to handle leachate quantities determined by computer modeling and consistent with rates at similar facilities. The drainage layer has been designed to maintain laminar flow and will be constructed of materials that are chemically resistant to leachate. The CQA Plan in **Appendix O** requires testing (ASTM D2488 and ASTM D3042) to verify that the granular materials will be compatible with the expected leachate at the landfill.

The leachate management system has been designed to safely handle leachate during routine maintenance and repair activities. To facilitate cleanout, each collection pipe will be connected to a cleanout riser. The proposed cleanout riser locations are shown on **Drawing D10**. The leachate collection pipes will typically be cleaned by hydraulic jetting or flushing, which requires access from only one end of the pipe. The leachate forcemain will also be cleaned by jetting. Hydraulic flushing or jetting typically uses a 1-inch hose connected to a 3-inch diameter nozzle assembly to deliver high-pressure water to remove obstructions. The hose and nozzle will fit through the 6-inch diameter leachate collection pipe. The 3-inch diameter nozzle can produce approximately 3,000 psi of hydraulic pressure, allowing it to easily breakup any obstructions.

Any liquid or debris resulting from the cleaning of the leachate collection line will be properly handled and disposed. All liquid will be treated as leachate, and any solid debris will be returned to the active face of the Landfill or hauled by a properly licensed truck to another permitted disposal facility.

The leachate collection pipes will be cleaned and maintained as necessary. The cleanout system has been designed so that all work can be performed at the ground surface. The leachate collection and management system will be routinely inspected for evidence of clogging or general system repair. Areas specifically targeted for maintenance inspections and monitoring include collection pipes (leachate levels), extraction points, leachate forcemains, leachate storage tanks, and leachate containment structures. Any observed damage or deficiencies will be quickly repaired following detection.

Leachate Collection and Disposal

As leachate collects in the sumps of the Expansion, it will be extracted using submersible pumps. The type of pumps used in the sumps will depend on the actual quantity and quality of leachate generated for each cell and is anticipated to vary over the life of the Landfill. Pumps will be installed with an automated leachate-level activated switch to pump leachate from the collection system when the leachate level within each sump rises to the level of the lowest leachate collection pipe entering the sump. The leachate drainage and collection system will not be used for the purpose of storing leachate. Any leachate system piping outside the waste boundary will be dual-contained. Once collected and removed, the leachate will be conveyed to either a publicly owned treatment works (POTW) facility or a privately owned treatment works facility for treatment and disposal or temporarily stored in a leachate tank.

Leachate Storage Tank and Secondary Containment System

35 IAC Section 811.309(d) requires that sufficient storage capacity is provided to contain the volume of leachate that is generated assuming the maximum daily leachate generation rate calculated in accordance with 35 IAC Section 811.307. In accordance with regulatory requirements, it is assumed that five days of storage capacity will be required, given that containment of leachate within onsite storage tanks are the only approved storage option. Calculation of the maximum daily leachate generation rate and required 5-day storage capacity is provided in **Appendix K.9**, resulting in a calculated storage requirement of 3,881 gallons under closed conditions.

The Facility currently operates two 32,000-gallon leachate storage tanks on the south side of the Facility and a 165,000-gallon leachate storage tank on the north side of the Facility which are permitted by IEPA to provide needed storage capacity for the existing Landfill. The 165,000-gallon tank is located within the proposed Expansion footprint, and therefore will be removed prior to construction of the first cell of the Expansion and relocated to the northwest corner of the proposed Expansion footprint. These tanks will continue to be used to serve the Expansion.

All on-site storage structures and secondary containment facilities comply with the conditions and specifications required by 35 IAC Section 811.309. The storage tanks will incorporate secondary containment equivalent to the protection provided by a 2-foot-thick clay liner having a permeability no greater than 1×10^{-7} cm/s. The primary tank shells will be coated steel or other material that is compatible with leachate.

Leachate Monitoring

Leachate will be sampled in accordance with 35 IAC Section 811.309(g). Sampling will be conducted as long as the leachate collection system is in operation. Test results will be submitted to the IEPA. The schedule for the leachate monitoring program is discussed in further detail in **Section 2.8** of this Application.

Evaluations of the Leachate Collection System

The leachate collection system has been evaluated to ensure that its design is appropriate for use at the Expanded Landfill. Calculations provided in **Appendix K.8** demonstrate that the leachate collection system is appropriately sized to convey the maximum estimated leachate flow volumes expected for the Landfill. The proposed design also exceeds the IEPA performance requirements by maintaining less than the maximum allowable one foot of leachate head across the liner floor during steady-state conditions.

In addition, the following key findings are summarized, as further presented in **Appendices J and K:**

1. The leachate collection system is capable of supporting the weight of the overlying landfill, including operating equipment (see **Appendices K.3 and K.4**).

2. The potential for differential settlement of the underlying compressible Wadsworth Till soils due to the weight of the landfill has been evaluated to ensure that the leachate collection pipes will continue to function as intended after settlement. The differential settlement was found to be nominal; the leachate collection pipe slope is appropriate for development (see **Appendix J.3-B**).

3. The maximum leachate head in the granular drainage blanket was calculated based on the estimated leachate generation rates, the hydraulic conductivity of the drainage layer and the leachate collection system design. The analysis indicates that the maximum leachate head in the granular drainage blanket will not exceed 12 inches, as required by regulations (see **Appendix K.6**).

4. The efficiency of the leachate collection pipes to collect and transport the maximum estimated leachate volume was assessed. The analysis indicates that the existing 6-inch diameter pipes beneath the vertical expansion area and the proposed 6-inch diameter pipes beneath the horizontal expansion area are appropriately sized to transport the peak percolation rate (see **Appendix K.8**).

Final Cover System

The Landfill will be covered with an engineered final cover system which will meet or exceed all federal, state, and local requirements. The final cover will be used to: 1) minimize the infiltration of precipitation, 2) prevent the release of landfill gas to the atmosphere, 3) support vegetation, and, 4) eliminate accessibility to the waste by vectors. The proposed final cover system is a multi-layer system consisting of:

1. A 12-inch-thick intermediate cover layer (foundation soils)

2. A 24-inch-thick low permeability compacted cohesive soil liner (maximum constructed hydraulic conductivity of 1 x 10^{-5} cm/sec)

3. A 40-mil double-sided textured LLDPE geomembrane liner

4. A geocomposite drainage layer

5. A minimum three-foot-thick protective layer overlaying the low permeability layer, with the uppermost six inches consisting of soil suitable for vegetation.

6. Vegetation consisting of grass or similar shallow-rooting vegetation

The final cover system will cover the entire Landfill and connect with the bottom liner system. A typical cross section of the proposed final cover is shown in **Drawing D15**, and the contours of the final landform are shown on **Drawing D11**. As shown on **Drawing D15**, the low permeability layer of the final cover will connect with the bottom liner system. The constructed slope of the final cover will be a minimum of 10 percent, with typical sideslopes of 4H:1V. The following text provides a more detailed description of each layer within the Landfill final cover system.

Low Permeability Layer

The 24-inch low permeability soil layer will have a constructed hydraulic conductivity of 1 x 10^{-5} cm/sec or less. The low permeability soil layer will be placed and compacted in lifts. Each soil layer will be uniformly placed with roots, cobbles, debris, organic, and other deleterious material removed prior to compaction. Additionally, the final surface will be inspected prior to geomembrane installation to ensure that no rocks, roots, or other objectionable items are exposed on the cover surface. All construction will be conducted and documented in accordance with the procedures outlined in the CQA Plan located in **Appendix O** of this application.

Geomembrane Layer

A 40-mil linear low-density polyethylene (LLDPE) geomembrane will be included in the composite final cover system for the facility. The material specifications for the 40-mil geomembrane liner material are included in **Appendix O** of this application. The geomembrane layer will serve as an impermeable barrier against infiltration of moisture through the final cover into the Landfill as well as a barrier preventing landfill gas from migrating out of the Landfill.

Geocomposite Drainage Layer

Overlaying the geomembrane layer is a geocomposite drainage layer. The geocomposite drainage layer consists of a geonet (drainage net) sandwiched by two non-woven needle-punched geotextiles. The geocomposite drainage layer will discharge at the toe of the Landfill final cover. The end of the geocomposite drainage layer will be protected, as shown on **Drawing D15**, and will discharge into a gravel envelope with drainage pipes installed with a nominal separation of 200 feet. The purpose of these outlets is to release hydraulic pressure and provide a discharge path into the perimeter stormwater channels. The material specifications for the geocomposite material are included in **Appendix O** of this Application.

The geocomposite drainage layer will serve three purposes:

1. Lowers the hydraulic head acting on the final cover, which improves the slope stability of the final cover;

2. Removes water from the final cover, reducing the potential for it to infiltrate into the waste mass; and

3. Provides a cushion layer between the geomembrane and the protective layer, reducing the potential for puncture of the geomembrane.

The geocomposite will be installed and tested in accordance with the requirements of the CQA Plan detailed in **Appendix O** of this Application.

Protective Layer

A protective layer consisting of a minimum of 36 inches of soil will be placed over the geocomposite drainage layer to protect the underlying layers from frost, desiccation, erosion, and penetration by roots or vectors. On-site material will be supplied for use in constructing the protective layer. The uppermost six inches of the material will consist of soil capable of supporting vegetation. The protective layer will be tested and placed in accordance with the requirements detailed in the CQA Plan, **Appendix O** of this Application.

Vegetative Cover

The vegetative cover planned for the Landfill is intended to protect the final cover from wind and water erosion, as well as to minimize run-off and maximize evapotranspiration. The vegetative cover will be placed after completion of the protective layer at the appropriate time for successful germination and growth.

The vegetative cover will consist of a variety of grasses that will: 1) protect the soil surface against erosion; 2) not interfere with the integrity of the geocomposite drainage layer or low permeable layer; 3) increase evapotranspiration thereby minimizing infiltration into the Landfill; 4) provide for sufficient stormwater management; and 5) improve the appearance of the final land surface. The vegetative cover will be established in accordance with the CQA Plan provided in **Appendix O**.

Time of planting is a critical factor in successful establishment of plants from seeds. Seed will be planted at the appropriate time for successful germination and growth based on soil temperature and precipitation, to be determined each year at the time of planting. Generally, seed will be planted in the spring or late summer/early autumn. Mulch and/or erosion control blankets will be applied as needed to control erosion and enhance vegetation establishment.

Final Cover Construction and Maintenance

The final cover will be constructed in accordance with the Specifications and Construction Quality Assurance guidelines outlined in the comprehensive CQA Plan (**Appendix O** of this Application). The low permeability layer of the final cover system will be constructed no later than 60 days after placement of the final lift of solid waste. The final protective layer will be placed as soon as possible after placement of the low permeability layer to prevent desiccation, cracking, freezing or other damage to the low permeability layer. The final protective layer will be 36-inches thick, which exceeds to frost penetration anticipated at the facility (approximately 20-24 inches). Thus, the final protective layer is sufficiently thick to prevent frost penetration into the underlying low permeability layer. Cover maintenance will be performed as necessary to maintain the final cover to meet the design objectives.

Cover Percolation

After placement of final cover, virtually all of the precipitation which falls on the Landfill will be diverted into the stormwater management system. Controlled runoff, evaporation, evapotranspiration, and barrier layers will minimize percolation through the final cover system.

Final Landform

Suitable grasses will be used for the vegetative cover, which will provide erosion protection. The grass seed mixture that is selected will be amenable to the soil quality/thickness, slopes and moisture/climatological conditions that exist and will not require significant maintenance. The seed mixture will be selected to protect the low permeability liner system from root penetration. Generally, a protective layer that is 450 mm (17.7 in.) to 600 mm (23.6 in.) is adequate to protect against root penetration. Since the protective layer will be 36-inches thick and the grass seed mixture will be carefully selected, the protective layer is deemed more than adequate to prevent root penetration from occurring in the geocomposite drainage layer or low permeability layer. Long-term management of grassed areas will require regular mowing. Fertilizer, lime, and mulch will be used at rates necessary to establish proper growth of the seed.

The maximum elevation of the Landfill in the horizontal expansion will be approximately 896 feet above MSL and in the vertical expansion it will be approximately 898 ft MSL. The gentle slopes of the Landfill top are proposed to be constructed no flatter than 10 percent to promote drainage from the top of the landform, allowing for differential settlement. The Landfill will have maximum slopes of 4H:1V on the sideslopes.

Terrace ditches and lined terrace downslope ditches and/or letdown culverts will be incorporated into the final slopes to further minimize erosion, as described in the Stormwater Management Plan in **Section 2.4** of this application.

Stormwater Management

The existing Landfill has a detailed stormwater management system that has been reviewed and permitted by the IEPA. Stormwater that falls on the Landfill is intercepted by the terrace benches and is directed to downslope ditches (also referred to as downchutes) or letdown pipes. The downslope ditches and letdown pipes convey water into ditches that follow the perimeter of the Landfill.

All existing stormwater controls that are not in the footprint of the proposed Expansion will continue to be utilized based on their proven performance. The Vertical Expansion Area will build upon a portion of the existing Landfill. As such, a portion of the stormwater that falls on the Vertical Expansion will utilize stormwater controls of the existing Landfill.

The Horizontal Expansion Area will be developed to the north of the existing Site 2 East Expansion area. The western and northern ditches around the horizontal expansion drain to the Detention Basin 8 system. The eastern ditches of the horizontal expansion drain to Detention Basin 5R.

The proposed Landfill will largely be developed with similar controls as the existing Landfill based on their proven performance, although it is noted that some of these features' dimensions have been modified as appropriate for the new development. However, the overall conveyance strategy remains similar.

All stormwater modeling has been completed that exceeds state, federal, and local requirements. Analyses indicate that stormwater will be discharged at a controlled rate for all modeled storm events, including the 100-year storm. Please refer to **Section 2.4** of this application for a description off the stormwater management plan and **Appendix M** for a demonstration that all controls are appropriate for this Landfill.

Landfill Gas Management

Landfill gas is a natural byproduct of the decomposition of waste in a landfill. Landfill gas contains methane, carbon dioxide, nitrogen, and other trace constituents. When captured for reuse, landfill gas is an important source of renewable energy. The Landfill includes systems to monitor and manage landfill gas.

Both below grade and above grade air monitoring will be provided at the facility. The Landfill gas monitoring probes and detection devices will be constructed/installed in accordance with all applicable federal and state requirements. A detail of a typical monitoring probe is included on **Drawing D20** and the proposed conceptual landfill gas management system is shown on **Drawing D14**.

The low permeability composite bottom liner and final cover systems minimize the potential for landfill gas to migrate from the waste boundary. Landfill gas will typically migrate through the most permeable zones within the landfill waste and will be less likely to migrate through the low permeable liner and cover systems. The landfill gas will typically migrate through pathways in the waste, flowing toward a landfill gas extraction well.

An active gas collection system already exists at the permitted Landfill and will be expanded to withdraw landfill gas from the Expansion area. The proposed gas system will collect gas and destroy methane and other constituents, reducing the potential for odors and greenhouse gas emissions. The existing perimeter odor misting system will also be expanded as the Expansion develops to neutralize odors, should they occur. The landfill gas is planned to be flared or may be recovered for reuse as energy at an onsite gas-to-energy facility or for other beneficial use. A detail of a typical vertical landfill gas extraction well and typical caisson landfill gas extraction well is shown on **Drawing D26**. Landfill gas extraction wells will be fitted with a pump to remove leachate as necessary to ensure adequate landfill gas extraction.

Landfill Gas Composition

Landfill gas quality is an important determinant of the end use for collected landfill gas. Landfill gas results from the decomposition of the waste, and therefore the quality of the landfill gas produced depends almost exclusively on the decomposition process. Landfill gas quality is different at each landfill and will also vary at different stages during the design life of a given landfill. In order to more fully appreciate how landfill gas quality will vary, it is important to understand the waste decomposition process.

The biological and chemical decomposition of solid waste results generally in the formation of heat, leachate, and landfill gas. Decomposition will begin soon after the waste material is placed in the landfill. The rate of decomposition will be affected by the availability of moisture, the physical and chemical characteristics of the waste, and the availability of oxygen. Waste decomposition passes through three phases, beginning with aerobic decomposition and proceeding to a two-phase anaerobic decomposition.

Food wastes typically decompose first, followed by paper, wood, textiles, and discarded un-stabilized plastics. Microbes that are initially present in the waste or introduced with the materials used as daily cover will initiate the aerobic portion of the decomposition process. Inert materials (soils, coal combustion byproducts, grit, some plastics, and some construction/demolition debris) which do not readily degrade will essentially remain unchanged by the decomposition process. The waste decomposition process will also yield methane, carbon dioxide, and traces of other gases. Some heat will be generated as the waste decomposes.

Initially, aerobic decomposition will take place with the principal by-products being carbon dioxide, leachate, and heat. Aerobic decomposition requires oxygen to continue. Modern landfills are designed to keep oxygen out as a method of fire control. Therefore, as the finite amount of oxygen within the waste is depleted, anaerobic decomposition will begin to take place. During the first phase of anaerobic decomposition, carbon dioxide and hydrogen are the principal by-products. Once the first phase of anaerobic decomposition is completed, the second phase of anaerobic decomposition begins. This decomposition results in the generation of methane (CH_4) and carbon dioxide (CO_2). Trace amounts of nitrogen, hydrogen sulfide, and other non-methanogenic organic compounds (NMOCs) are also present in the second phase of anaerobic decomposition. The typical composition of landfill gas generated at a conventional sanitary landfill during this second phase is summarized in **Table 2.3-1**.

Table 2.3-1 Typical Composition of Landfill Gas	
Landfill Gas	**Component Percentage* (Dry Volume Basis)**
Methane (CH_4)	50% to 55%
Carbon Dioxide (CO_2)	45% to 50%
Other gases (oxygen (O_2), nitrogen (N_2), sulfides, etc.)	2% to 5%
Source: U.S. EPA, *Landfill Gas Energy Basics*, LFG Energy Project Development Handbook, June 2017.	

Quantity of Landfill Gas

The rate of landfill gas generation is dependent upon the waste decomposition process, which is controlled by many factors including moisture availability, waste composition and availability of oxygen. Diversion of paper, aluminum, plastics, and landscape waste may also have an effect on the generation of methane. The total quantity of landfill gas that will be generated can be estimated based on measurements of gas quantities at existing conventional landfills. Actual monitoring of the landfill gas at the Landfill will verify the quantity and quality of the landfill gas.

The quantity of landfill gas that is generated also depends on the quantity of waste being decomposed. The rate of waste decomposition and landfill gas production is primarily controlled by the moisture content of the waste. The most significant landfill gas generation rates occur when moisture in the form of leachate flows through the waste, transporting the bacteria and nutrients necessary for decomposition. This movement of leachate through the waste occurs only when the moisture content of the waste is above field capacity or when infiltrated moisture passes through preferential pathways that may exist in the waste. The final cover of the Landfill has been designed to minimize the infiltration of moisture into the waste after closure.

Typically, generation of significant quantities of landfill gas occurs for a period of thirty to forty years after placement. Gas generation rates are calculated in **Appendix L** for the existing Landfill; these calculations will be updated as Landfill development (including development in the Expansion area) proceeds.

Landfill Gas Collection

Landfill gas generated will be controlled in accordance with all applicable current and future regulations, including applicable Clean Air Act New Source Performance Standards (NSPS) and 35 Ill. Admin. Code requirements. The current *NSPS Landfill Gas Collection and Control System Design Plan* for the Landfill is contained in **Appendix L** and will be periodically updated as Landfill development proceeds. The gas collection system and all associated equipment will be part of the facility. Under no circumstance shall the gas collection system compromise the integrity of the liner, leachate collection system, or final cover system.

The gas collection system will be designed and constructed to function for the entire design period and be able to accommodate changing gas flow rates or compositions. **Drawing D14** illustrates conceptual extraction well locations for the Expansion and **Drawing D26** shows a typical extraction well from such a system. The gas collection system shall be operated until the waste has stabilized enough to no longer produce methane quantities that exceed allowable concentrations in 35 IAC Section 811.311(a)(1-3).

Multiple gas extraction devices will allow gas to be efficiently extracted from the Landfill during all stages of development. During cell construction, caisson vertical extraction wells will be constructed overlying the leachate collection layer. The caisson wells will consist of perforated piping surrounded by coarse aggregate within the caisson. The landfill gas collection piping will be vertically extended as cell filling progresses, and the caisson will be raised during each extension until final grades are achieved. This will enable collection of landfill gas soon after waste placement and provide direct drainage of leachate and gas condensate to the leachate collection system. Horizontal gas collection piping will supplement the vertical gas extraction wells.

A vertical collection well spacing with a radius of influence of 125-150 feet within the center landfill area and 125' along the perimeter, consistent with the currently utilized landfill gas collection system, is currently anticipated unless a larger well spacing can be demonstrated in accordance with state and federal guidelines. Extraction wells will be interconnected through a wellhead piping system. This landfill gas extraction network will transport the landfill gas to a central location for processing at a landfill gas flare, gas-to-energy facility or other approved method of processing depending on the landfill gas quality. A minimum 6" solid HDPE pipe will be used. However, header pipes will be properly sized to accommodate the landfill gas quantity. The gas collection system shall be operated until the waste has stabilized enough to no longer produce methane quantities that exceed allowable concentrations in Section 811.311(a)(1-3).

The landfill gas collection piping system will be composed of HDPE or other material capable of resisting corrosion due to the landfill material and gas composition. HDPE and other materials also offer strength and flexibility which will withstand the effects of settlement to the system. Landfill gas piping may be installed above or below the final cover geomembrane, with initial installation typically occurring below the geomembrane and future replacement, if needed, occurring above the geomembrane. The well head assembly will be equipped to allow the monitoring and adjustment of landfill gas flow and the collection of landfill gas samples.

The gas header pipes will be sloped to drain condensate to either condensate driplegs within the Landfill waste or to condensate sumps located outside the waste boundaries and part of the perimeter gas header. Condensate sumps will be single-walled HDPE structures with the sump portion wrapped in GCL. Collected condensate will be pumped to the leachate tank through underground double-walled transmission piping. A sufficient number and locations of condensate sumps and driplegs will be established to ensure condensate management. Condensate that is collected will be stored and managed as leachate. Gas will not be directly discharged to the atmosphere without treatment or burning, in accordance with a permit issued pursuant to 35 Ill. Adm. 200-45.

Settlement will occur due to decomposition of the refuse. The design of the GCCS components include several features to account for this settlement. As detailed on **Drawing D26**, the extraction well heads will be connected to the LFG transmission piping via a flexible pipe or hose connection. This allows the LFG piping to accommodate changes in the orientation of the LFG transmission piping or LFG extraction well. Additionally, the LFG transmission piping within the Landfill waste boundary will be sloped at sufficient grades (at a minimum slope of six percent) so that reasonable amounts of differential and total settlement may occur without causing pipe breakage or disrupting the overall flow gradient of the LFG transmission piping. These slopes exceed the maximum differential settlement values determined in **Appendix J**.

Compliance with Siting Ordinance Conditions

In accordance with the conditions of the Siting Ordinance, the landfill owner/operator commits to installation of the landfill gas collection system, as permitted, in each cell, within the first three years of waste acceptance in any cell, or as otherwise needed to maintain BMPs at the landfill, whichever occurs first. The landfill gas collection system shall, at a minimum, follow BMPs for construction, installation, repair or alteration, and monitoring, at the time such activities take place. For example, current BMPs may include, but are not limited to:

1. Landfill gas collection on leachate sumps for odor control;
2. Early collection of landfill gas through horizontal or caisson wells;
3. Precision flow meter or equivalent at well head;
4. Surface emission monitoring; and
5. Liquid removal from vertical landfill gas wells, as necessary.

Necessary repairs to or replacement of any gas collection header piping that remains below the final cover geomembrane upon construction of the final cover will be performed by abandoning the affected piping in-place and installing replacement piping above the final cover geomembrane.

Geotechnical Analyses

Geotechnical analyses have been performed for the proposed design in order to verify that the liner and final cover will be stable during construction, operation, and following closure of the Landfill. The analyses demonstrate that the Landfill slopes will be stable and that the structural integrity of the bottom liner and final cover will be maintained over the life of the Landfill and beyond. Specifically, the following evaluations have been completed:

1. *Shear Strength Evaluation*. The stability of the proposed final cover system and bottom and sideslope liner and leachate collection system were evaluated to ensure the minimum factors of safety against failure (1.5 for static conditions and 1.3 for seismic conditions) are achieved.

2. *Foundation Evaluation*. Foundation evaluations analyzing the maximum foundation settlement, hydrostatic uplift, and foundation bearing capacity failure potential were conducted for the proposed Landfill.

3. *Liner / Leachate Collection System Evaluation.* This evaluation includes calculations analyzing the anchor trench design, wheel loading, and puncture resistance. These evaluations consider additional geosynthetic material considerations as to whether the proposed materials will function as required over the life of the proposed Landfill.

4. *Final Cover Evaluation.* This evaluation contains analyses which determine the maximum differential settlement of the waste, whether the geomembrane has the required strength to withstand the normal stresses imposed by the waste stabilization process, whether the final cover geocomposite and toe drains will remain free-draining, and the factor of safety against slope failure of the terrace berms on the final cover for static and seismic conditions.

5. *Additional Geosynthetic Strength and Protection Considerations*. These analyses include several calculations such as geomembrane strain, leachate pipe

deflection and crushing, wheel loading, puncture resistance, and final cover geocomposite transmissivity. These evaluations consider additional geosynthetic material considerations as to whether the proposed materials will function as required over the life of each Landfill design option.

Supporting documentation and calculations are provided in **Appendix J**. Geotechnical analyses have been performed under the direct supervision of a licensed professional engineer experienced in geotechnical engineering.

Geotechnical Analyses Design Parameters Summary

A summary of the material unit weights and shear strength values for the Landfill layers and geologic units is presented in **Table 2.3-2**. These values were calculated from laboratory test results that were completed as part of the hydrogeological investigation. These values are used in the geotechnical calculations in **Appendix J**.

Table 2.3-2
Zion Landfill – Site 2 North Expansion
Summary of Material Unit Weights and Shear Strength

Layer Description	Dry Unit Weight "γ_{dry}" (pcf)	Total Unit Weight "γ_{total}" (pcf)	Saturated Unit Weight "$\gamma_{saturated}$" (pcf)	Shear Strength Short-Term Conditions[1] Cohesion c (psf)	Shear Strength Short-Term Conditions[1] Friction Angle φ' (degrees)	Shear Strength Long-Term Conditions[2] Cohesion c' (psf)	Shear Strength Long-Term Conditions[2] Friction Angle φ' (degrees)
In Situ / Foundation Soils Beneath Landfill & Outside Landfill Footprint							
Wadsworth Till	118.4	136.6	137.8	1,465	11.8	1,000	14.3
Shallow Drift Aquifer[3]	104.8	123.3	129.8	-	-	-	-
Landfill Layers:							
Final Cover Soils[4]	106.7	121.5	130.3	1,465	11.8	0	34.3
Waste Fill[5]	75.0	75.0	75.0	0	33	0	30
LCS Granular Drainage Layer[6]	125.0	126.0	130.0	0	30	0	30
Low Permeable Earth Liner[4]	112.6	128.2	134.1	1,465	11.8	0	34.3

Notes:

1. Shear strength values for short-term conditions of the Wadsworth Till, Final Cover Soils, Low Permeable Earth Liner are derived from the unconsolidated-undrained triaxial shear strength Mohr circles (see attached figures). It is assumed these conditions occur during initial landfill cell development and interim waste fill heights / active landfill cell phase. A summary of the test results are presented in the attached Tables and the complete laboratory test results are provided in **Appendix I**. The Mohr circles are also provided in the attached pages.
2. Shear strength values for long-term conditions of the Final Cover Soils and Low Permeable Earth Liner are conservatively derived from the Mohr circles of the effective stress, consolidated-undrained triaxial shear strength tests. The long-term shear strength value assumed for the Wadsworth Till is derived from the Mohr circles of the total stress, consolidated-undrained triaxial shear strength tests. A summary of the test results are presented in the attached Tables and the complete laboratory test results are provided in **Appendix I**. The Mohr circles are also provided in the attached pages.
3. The Shallow Drift Aquifer, Lower Till, Basal Drift, and Bedrock units are significantly lower than the proposed landfill base and therefore were not considered in the geotechnical analyses.
4. The unit weights of the Final Cover Soils and the Low Permeable Earth Liner are derived from the results of Standard Proctor tests performed on the Wadsworth Till soils. It was assumed that the Final Cover Soils and Earth Liner Soils will be compacted to 90% and 95% of the Standard Proctor test results, respectively, and therefore the unit weights of the Final Cover Soils and Low Permeable Earth Liner are based on these corresponding values. A summary of the Standard Proctor test results are presented on Table 3 in the attached pages. The complete Standard Proctor laboratory test reports are provided in **Appendix I**. Modified Proctor testing was not analyzed for hydraulic conductivity testing or shear strength evaluations. In the event that Modified Proctors are intended to be used during construction, shear strength values should be evaluated prior to use to ensure that the strength parameters fall within the acceptable ranges identified within this Appendix. While direct comparisons cannot be made, 90% and 95% of the Standard Proctor (for the Final Cover Soil and Low Permeable Earth Liner, respectively) generally produce similar compactive efforts as 85% and 90% of the Modified Proctors, respectively. It should be noted that the unit weights of the Final Cover Soil and Low Permeable Earth Liner in the Site 2 North Expansion are as conservative as or more conservative than the values used for the Zion Site 2 East Expansion completed in 2011 by Weaver Boos Consultants, Inc.
5. The unit weight of the waste fill is based on an average value reported in published technical literature (see attached pages). The shear strength values of the waste fill are based on bilinear shear strength envelopes results for MSW waste in conventional landfills described in Eid et. al (2000). It should be noted that the unit weights and shear strength values of the Waste for the Site 2 North Expansion are as conservative as or more conservative than the values used for the Zion Site 2 East Expansion completed in 2011 by Weaver Boos Consultants, Inc.
6. The unit weights of the LCS Granular Drainage Layer are based on the unit weights of similar materials used at other landfills. It should be noted that the unit weights and shear strength values of the LCS granular drainage layer for the Site 2 North Expansion are as conservative as or more conservative than the values used for the Zion Site 2 East Expansion completed in 2011 by Weaver Boos Consultants, Inc.

Additionally, for all geotechnical analyses a seismic coefficient of 0.0461g for the Landfill site area was used. This value was obtained from the United States Geologic Survey (USGS)

Earthquake Hazards Program – National Seismic Hazard Mapping website. It represents a 10% or greater probability that the maximum horizontal acceleration in lithified earth material, will exceed 0.10g in 250 years.

Shear Strength Evaluations

Stability analyses were performed for the final cover and bottom liner and leachate collection systems in order to determine if the geometry and material properties of the proposed Landfill design are appropriate and will remain stable during static and seismic conditions.

Final Cover Stability

A final cover stability analysis was conducted to determine the range of acceptable peak shear strength parameters for the final cover system. Multiple combinations of friction angles and adhesions were evaluated to determine the minimum acceptable peak interface shear strength envelope to achieve stability of the final cover. The results of the analysis yielded factors of safety greater than 1.5 for static conditions and greater than 1.3 for seismic conditions. The supporting calculations are provided in **Appendix J.2-A**.

Bottom Liner and Leachate Collection System Stability Prior to Waste Placement

A liner and leachate collection system stability analysis was conducted to determine the range of acceptable shear strength parameters that provide a factor of safety against slope failure prior to waste placement. Multiple combinations of friction angles and adhesions were evaluated to determine the minimum acceptable interface shear strength envelope to achieve stability of the liner and leachate collection system prior to waste placement. The results of the analysis yielded factors of safety greater than 1.3 for static conditions and greater than 1.0 for seismic conditions. The supporting calculations are provided in **Appendix J.2-B**.

Bottom Liner and Leachate Collection System Stability After Waste Placement

A pseudo-seismic analysis was performed to determine the range of acceptable liner and leachate collection system shear strength parameters that provide a factor of safety against slope failure during construction/operation and closure periods and during seismic events.

Landfill Stages Analyzed and Modes of Failure

The stability of the Landfill was analyzed for two different landfill stages: complete landfill build-out / final landform and intermediate/operational buildout. The two landfill stages were analyzed using two modes of failure within the computer model SLIDE (a 2D Limit Equilibrium Slope Stability software program by Rocscience, Inc.) - translational (non-circular / block) failure and rotational (circular) failure. The translational failure mode was used to analyze the stability of the liner system along critical (weak) interfaces; and the rotational failure mode was used to analyze the stability of the waste mass and the foundation.

The stability analyses were performed for both short-term (unconsolidated / undrained) and long-term shear strength (consolidated / undrained) under static and seismic loading conditions. Long-term shear strength conditions will most likely occur following the complete build-out of the Landfill.

Results of the stability analyses are summarized in **Table 2.3-3**. The following results demonstrate that the Landfill design meets the requirements of 35 Ill. Admin. Code (35 IAC) 811.304, which states that all final slopes must achieve a minimum factor of safety of 1.5 for static conditions and a minimum factor of safety of 1.3 for seismic conditions. A more detailed discussion is provided in **Appendix J.2-C** that includes a discussion of the critical cross sections selected for analysis, the scenarios / conditions modeled for each cross section, and supporting model output files.

Table 2.3-3 Zion Landfill – Site 2 North Expansion Slope Stability Summary				
Analysis	Factors of Safety			
	Short-Term Shear Strength		Long-Term Shear Strength	
	Static	Seismic	Static	Seismic
Stability Cross Section A-A' – Horizontal Expansion (northern slope): Intermediate Buildout				
NonCircular / Liner Block Search	1.523	1.300		
Circular / Grid Search	1.709	1.332		
Stability Cross Section A-A' – Horizontal Expansion (northern slope) : Complete Buildout				
NonCircular / Liner Block Search	2.127	1.738	1.984	1.628
Circular / Grid Search	2.658	1.914	2.337	1.949
Stability Cross Section B-B' – Horizontal Expansion (eastern slope) : Intermediate Buildout				
NonCircular / Liner Block Search	1.549	1.320		
Circular / Grid Search	1.850	1.532		
Stability Cross Section B-B' – Horizontal Expansion (west slope) : Complete Buildout				
NonCircular / Liner Block Search	2.188	1.790	2.040	1.676
Circular / Grid Search	2.711	2.117	2.339	1.951
Stability Cross Section B-B' – Horizontal Expansion (east slope) : Complete Buildout				
NonCircular / Liner Block Search	2.128	1.742	1.982	1.629
Circular / Grid Search	2.623	2.042	2.340	1.953

Evaluation of Wadsworth Till During Rapid Drawdown of Detention Basin

Rapid drawdown conditions arise when submerged slopes experience a rapid reduction in water level. The reduction in water level removes the stabilizing force from the weight of the water and the pore pressure of the basin foundation material (Wadsworth Till) will be slow to dissipate. These scenarios will reduce the slope stability of the basin. This calculation is developed to identify the lowest factor of safety assuming that rapid drawdown of the detention basin occurs with the force of the fully constructed landfill behind it (worst case scenario).

Landfill Stages Analyzed and Modes of Failure

Stability of the landfill was analyzed during final buildout (following final cover placement) conditions and during rapid drawdown conditions of the detention basin. There are three methods of rapid drawdown analyses in SLIDE with two of the methods having different interpolation methods which relate the undrained strength of the soil (after drawdown) to the pre-drawdown strength.

The stability of the waste mass and foundation after rapid drawdown was evaluated within the SLIDE model using the rotational (circular) failure. This uses a grid search to find the most critical circular failure surfaces within the waste mass and foundation. The grid search was performed in an iterative manner by the SLIDE model user. Each time the user adjusted / fine-tuned the grid to the point where the model generated the absolute lowest factor of safety.

Results of the stability analyses are summarized in **Table 2.3-4**. A more detailed discussion is provided in **Appendix J.2-D** that includes a discussion of the critical cross section selected for analysis, the scenarios / condition modeled for cross section, and supporting model output files.

Table 2.3-4 Zion Landfill – Site 2 North Expansion Rapid Drawdown Conditions		
Analysis (Interpolation Method)	**Factors of Safety**	
	Short-Term Shear Strength	
	Seismic (>1.3)	**Static (>1.5)**
Stability Cross Section A-A' – Horizontal Expansion (northern slope) : Complete Buildout		
Duncan, Wright and Wong (VandenBerge, Wright)	1.951	2.657
Duncan, Wright and Wong (Duncan, Wright and Wong)	1.950	2.655
Lowe and Karafiath (VandenBerge, Wright)	2.076	2.771
Lowe and Karafiath (Duncan, Wright and Wong)	2.084	2.728
Army Corp of Engineers (NA)	1.862	2.584

Landfill Foundation Evaluations

Foundation evaluation calculations were performed for the proposed Landfill. These analyses verify the Landfill foundation is will remain stable during excavation, capable of supporting the weight of overlying operating equipment and waste, will maintain stability in seismic situations, and that the leachate collection system will continue to function as intended with foundation settlement.

Hydrostatic Uplift

The stability against hydrostatic uplift of the excavation during construction activities was estimated. The potentiometric levels of the Wadsworth Till were assumed to be 5-feet below the existing ground surface and in contact with the top of the granular drainage layer along the liner base and side slopes. This represents the worst-case scenario for groundwater at the site. The maximum excavation depth will occur in Cell 11 and be approximately 60 feet.

The hydrostatic uplift under these conditions was determined to be 3,744 psf. Based on the worst anticipated conditions at the site and a minimum factor of safety of 1.2, it was determined that hydrostatic uplift will be counteracted once waste is placed in the horizontal expansion to an initial height of approximately 49.2 feet. Before the waste reaches this height, stability will be achieved by dewatering of the Wadsworth Till using the gradient control system. See **Appendix J.3-A** for the calculation.

Foundation Settlement

As the Landfill is constructed, the weight of the waste will cause the low permeable earth liner and the Wadsworth Till foundation to consolidate slightly. Consolidation is the settlement due to the reduction of void space. Differential settlement calculations were performed to verify that the leachate collection system will still drain after the Landfill foundation settles (refer to **Appendix J.3-B**).

It was determined that the slopes of the leachate collection system pipes exceed the maximum anticipated differential settlement that will occur, allowing the pipes to remain free-draining. Although the slope of the proposed leachate collection system may change over time due to settlement, the resulting slopes will continue to allow for drainage and meet performance requirements.

Bearing Capacity Foundation Analysis

Bearing capacity analyses were performed to demonstrate that the foundation materials beneath the proposed Landfill exhibit sufficient strength to support anticipated loads. The most critical location across the Landfill base was analyzed at the maximum waste height for the proposed Landfill, which was found to be in Cell 7 in the vertical expansion area. Terzaghi's bearing capacity equation was used to calculate the ultimate bearing capacity using engineering properties of the geologic and engineered fill materials. The factor of safety is the ratio of the ultimate bearing capacity to the overburden pressures expected to act on the foundation.

The results of the analysis yielded factors of safety greater than or equal to 2.0 under static conditions and greater than 1.8 under seismic conditions. The supporting calculations are provided in **Appendix J.3-C**. The calculations contained in **Appendix J.3-C** also demonstrate that the bedding materials of the leachate collection system possess the structural strength to support the maximum loads imposed by the overlying materials and landfill equipment.

Liner/Leachate Collection System Evaluations

Liner/leachate collection system evaluations were performed for the proposed Landfill design to ensure the geosynthetic materials will continue to function as required over the life of the Landfill design.

Anchor Trench Design

The geosynthetics to be used as part of the proposed Landfill design provide sufficient friction angles that they are anticipated to hold themselves in place after installation. However, anchor trenches are proposed to be used along the perimeter of the waste boundary to bury the edge of geosynthetic materials, in order to protect the edges and provide protection from wind uplift. The anchor trench design was evaluated based on the strength properties of the geomembranes.

It was found that the depth of the anchor trench should not exceed 5.2-feet in order to provide holding capacity against the self-weight of the geomembrane, while allowing pull-out of the geomembrane at loads approaching the ultimate material strength of the geomembrane, which minimizes the potential for tearing. The proposed design depth for each anchor trench is 3-feet and therefore the anchor trench design is considered appropriate. See **Appendix J.4-A** for detailed calculations.

Wheel Loading on Geomembrane

The wheel loading due to construction and compaction equipment operating on the initial lift of waste and acting on the geomembrane was evaluated. The wheel loading was analyzed using the Caterpillar 836K Compactor and the product information of a 60-mil HDPE geomembrane. A resulting factor of safety of 56.6 was determined, which indicates that the geomembrane can withstand the wheel loading of the construction equipment without degradation in material quality. See **Appendix J.4-B** for supporting calculations.

Puncture Resistance of Geosynthetics

The geosynthetics in the composite liner and leachate collection systems (consisting of the 60-mil HDPE geomembrane, 10-oz/yd^2 non-woven geotextile filter, and 12-oz/yd^2 non-woven geotextile cushion) were analyzed to demonstrate they are an appropriate thickness to resist puncture from the adjacent aggregate material in the horizontal expansion. The geosynthetics were analyzed at the maximum waste thickness of approximately 198 feet in the horizontal expansion area, based on an aggregate shape being sub-rounded to sub-angular and an assumed safety factor of 2.0.

Based on these parameters, the maximum acceptable average diameter for aggregate to resist puncture of the geotextiles and the aggregate material diameters specified in the CQA Plan (see **Appendix O**) is as follows:

1. For the 10-oz/yd^2 geotextile filter overlying the granular drainage layer: 2.25 inches. This is greater than the assumed maximum granular drainage layer particle diameter of 1.0 inches.

2. For the 10-oz/yd^2 geotextile overlying the leachate collection system trench coarse aggregate in the leachate collection trenches: 1.74 inches. This is greater than the assumed maximum leachate collection system coarse aggregate diameter of 1.5 inches.

3. For the 12-oz/yd^2 geotextile underlying the granular drainage layer across the base of the horizontal expansion: 2.40 inches. This is greater than the assumed maximum granular drainage layer particle diameter of 1.0 inches.

4. For the 12-oz/yd^2 geotextile overlying the 60-mil HDPE geomembrane and underlying the leachate collection system trench coarse aggregate in the leachate collection trenches: 1.86 inches. This is greater than the assumed maximum leachate collection system coarse aggregate diameter of 1.5 inches.

See **Appendix J.4-C** for supporting calculations.

To demonstrate puncture resistance of the geomembrane underlying the leachate collection system in the proposed horizontal expansion and in the existing constructed areas over which the vertical expansion will be constructed, a series of laboratory (including bench-scale and large-scale) evaluations were conducted using the same material configuration as what is proposed. The laboratory evaluation was originally conducted for the Orchard Hills Landfill, located in Davis Junction, Illinois, to replicate the puncture resistance of the in-place LCS geosynthetics[1]. Therefore, the laboratory report is being used to demonstrate that the proposed LCS pipe trench configuration in the proposed horizontal and vertical expansions of Zion Landfill will not puncture the 60-mil textured geomembrane. This approach was utilized because necessary coefficients to complete the calculation were not available in

[1] Zion Landfill, Inc. and Orchard Hills Landfill were historically under the common ownership of Advanced Disposal Services until October 29, 2020. The prior evaluation conducted for Orchard Hills Landfill was completed in advance of that date and provided to the Landfill team for use in this application.

source literature for the specific combination of aggregate materials and geotextiles that will be constructed in the horizontal expansion and have been constructed at the existing Landfill. The laboratory evaluations demonstrated that materials utilized in construction of the existing leachate collection system did not result in puncture under the loading conditions that will be present in the proposed horizontal and vertical expansions. The laboratory evaluations are provided in **Appendix J.4-C**.

Final Cover Evaluations

The final cover was evaluated to ensure adequate drainage will be maintained and that the geomembrane and terrace berms will have the appropriate strength and geometry to support stability throughout the life of the Landfill.

Waste Settlement

This calculation determines the maximum settlement that is anticipated to occur within the waste mass at multiple locations to ensure that the plateau area of the final cover will maintain positive drainage after settlement occurs. For the purpose of this analysis, the maximum differential settlement is determined for the plateau (top) of the Landfill, as it is designed with the minimum slope of all final cover areas. The maximum potential differential settlement within the waste mass is added to the calculated differential settlement within the foundation to determine whether the slopes of the final cover are appropriate.

Five analysis points were determined to provide the maximum potential settlement along the plateau in the horizontal and vertical expansions. These five points represent the maximum and minimum waste thickness along the plateau in the horizontal and vertical expansions, and the maximum waste thickness over an LCS pipe along the plateau in the vertical expansion. The maximum waste thickness for the proposed Landfill design will exist in Cell 7 with an approximate material thickness of 206 feet. The minimum thickness will exist between Cells 7 and 1 and Cells 9 and 1 with an approximate waste thickness of 136.7 feet.

The maximum differential settlement across the proposed Landfill plateau is calculated to be approximately 5.04 percent (4.15 percent + 0.89 percent from the foundation soil settlement). The design slope of the plateau is 10H:1V (approximately 5.71 degrees). Therefore, the resulting slope after differential settlement is anticipated to be approximately 0.67 degrees (approximately 1.17 percent). This slope is acceptable, as the final cover will maintain positive drainage. See **Appendix J.5-A** for additional information.

As an additional safeguard, the Landfill final cover will be periodically monitored, and maintenance will be performed as necessary. Final cover inspection and maintenance will be performed in accordance with the facility's post-closure care plan contained in **Section 2.9** of this Application.

Final Cover Geomembrane Strain

The final cover geomembrane was evaluated to see if it possesses the required strength to withstand the normal stresses imposed by the waste stabilization process. A textured LLDPE geomembrane is analyzed, which will be utilized in all areas with slopes greater than 10H:1V. The allowable strain for the final cover geomembrane was determined to be 30 percent, which is based on manufacturer's specifications.

AutoCAD Civil3D 2018 (AutoCAD) was used to determine the maximum differential settlement dimensions that occur based on the initial design final cover slopes and maximum 30 percent allowable strain. The maximum allowable strain was then calculated and it was determined that the geomembrane can accommodate a differential settlement of 64 percent for 4H:1V slopes before reaching its allowable strain limit. A differential settlement of 64 percent far exceeds the maximum differential settlement that was calculated for the final cover due to waste settlement (please refer to **Appendix J.5-A**). However, the final cover will be routinely observed for differential settlement. The geomembrane will be evaluated for over-stressing in locations where differential settlement exceeds 64 percent. See **Appendix J.5-B** for the calculation.

Final Cover Geocomposite Transmissivity

The final cover geocomposite was evaluated to see if it will remain free-draining based on stormwater impingement rates through the final cover. A 6-oz/yd^2 double-sided geocomposite drainage layer was analyzed over the minimum final landform slope of 10H:1V. The maximum daily peak head from the HELP model in Appendix K was used to estimate the amount of head on the final cover geocomposite. Using this information, the field geocomposite flow rate was determined to be 2.6x10^{-5} ft^3/sec. This value is greater than the maximum flow rather through the overlying final cover soils, which was determined to be 1.4x10^{-6} ft^3/sec, and therefore the final cover geocomposite will be free-draining. See **Appendix J.5-C** for the calculation.

Toe Drain Capacity

The proposed 4-inch toe drains (discharge pipes) were evaluated to ensure they are adequately sized to drain water that percolates through the final cover and is transmitted downslope through the 6-oz/yd^2 geocomposite. The toe drains are designed with a 200-foot spacing interval. The maximum flow rate of the water converging on the toe drain from the geocomposite was determined to be 0.20 ft^3/sec across the 200-ft wide spacing. The maximum flow rate for the 4-inch pipes at full capacity was determined to be 0.39 ft^3/sec. Based on these values it was determined the proposed toe drain spacing and sizing will pass a flow rate of water greater than the maximum flow rate of water discharging from the geocomposite and entering the toe drain. See **Appendix J.5-D** for the calculation.

Terrace Berms

The proposed terrace berm configuration was evaluated to determine the factor of safety against slope failure for static and seismic conditions. The terrace berms for the proposed final cover will typically have a 2H:1V slope and will rise approximately 2.0-feet above the highest common point of the slope. In the analysis it was assumed that the berms will be constructed from the same materials as the final cover soils.

The results of the analysis can be seen in **Table 2.3-5** below. Based on this analysis, the terrace berms have been designed to meet the required factor of safety for both static (at least 1.5) and seismic conditions (at least 1.3).

Table 2.3-5 Terrace Berms Factor of Safety		
Analysis	**Short-term Conditions**	**Long-term Conditions**
Static	26.6	2.73
Seismic	22.5	2.20

See **Appendix J.5-E** for an in-depth analysis and calculations.

Design Period

The incremental capacity of the proposed Expansion will begin to be filled at the end of the operating life of the existing Landfill, which is currently estimated to be in 2027. The estimated operating life of the landfill may vary due to changes in incoming waste volume and waste compaction rates but is estimated to continue through approximately 2044. The Landfill will be constructed and operate to perform safely throughout and after the entire design period, including a minimum of thirty (30) year of post-closure. Additional information and calculations of the operating life are provided in **Appendix N** of this Application

Construction Phasing

The Expansion consists of approximately seven cells (Cells 11-17) in the horizontal expansion and a vertical expansion over Cells 6, 7, and 9 of the existing Landfill. The Landfill will be developed starting with Cell 11 on the southern portion of the facility and progressing sequentially northward. The vertical expansion will be filled concurrently with Cells 11 and 12. It is noted that cells may be constructed incrementally (portions of a cell) based on the waste throughput needs at the time of construction.

Following the construction of each Landfill cell, or portion thereof, operating permits must be granted from the IEPA prior to waste acceptance. In the event that landfill regulations change prior to cell construction, the Landfill design, technology, or construction technique will be modified as necessary to be in compliance with the new regulations. Once active, each cell of the Facility will generally be sequentially filled as shown in **Drawings D30-D37**. Cell boundaries are depicted on **Drawings D7-D10**, and each phase of cell construction is shown in **Drawings D30-D37**. The actual size and configuration of each phase will depend upon a number of factors, including waste volumes, stormwater routing, permitting, etc. As a result, the phasing plan illustrated in **Drawings D30-D37** is considered to be preliminary; actual phasing could vary from that shown.

The site development provides for sequential construction, filling, and closure of parts of the proposed Landfill throughout the operating life. The final cover will be placed contemporaneously with the Landfill development when possible. This will be accomplished by constructing the final cover in phases as portions of the Landfill achieve final grade. Construction of the stormwater features will be developed concurrently with development to ensure adequate stormwater controls are provided.

The phasing of Landfill development will have a number of important benefits that enhance the environmental safety of the facility:

1. Construction will occur in a planned, orderly manner.

2. Adequate disposal areas will be constructed to handle incoming waste flows.

3. The size of “active” disposal areas will be minimized, reducing the quantities of leachate generated and the potential for nuisance impacts (e.g., dust, odors) to develop.

4. Completed sections of the Landfill may be capped with final cover as they reach final grades, reducing the quantities of leachate generated.

Estimated Phasing Schedule

Table 2.3-6 summarizes the approximate size and the projected year of construction, filling, and closure of the waste disposal areas comprising the proposed Landfill. Note that filling simultaneously occurs in multiple phases as phases cannot be filled to final grade until adjacent cells approach final grade. The anticipated phasing is dependent upon variable conditions such as incoming waste volumes and weather conditions. The phasing schedule assumes that cell construction will occur in the spring, summer, or fall preceding the year when the capacity will be needed. Placement of final cover and establishment of permanent vegetative cover will occur as soon as practicable. Estimated closure dates are expected to be representative of side-slope closure periods, with plateau areas being closed in later years when final grades are achieved and waste settlement has occurred.

Considering all of the various influences on construction schedules, including weather and fill volumes, the estimated sequence of construction represents the phasing envisioned at the time of design. Adjustments and modifications are anticipated considering the size, complexity and life of this project, and the design of the Landfill provides the flexibility to adjust phasing as necessary.

Table 2.3-6
Approximate Phasing of Cell Development

Phase	Phase Description	Approx. Year of Construction	Approx. Year of Filling	Approx. Year of Side-Slope Closure
A	Cell 11	2026	2027-2030	2031
B	Cell 12	2027	2028-2033	2034
C	Cell 13	2028	2029-2035	2036
D	Cell 14	2032	2033-2038	2039
E	Cell 15	2034	2035-2040	2041
F	Cell 16	2037	2038-2042	2043
G	Cell 17	2039	2040-2044	2045

1. Years of Construction, Filling, and Closure are approximate.
2. Years of Closure reference expected year of side-slope closure for each cell. Plateau areas will be closed in later years when final grades are achieved and waste settlement has occurred.
3. Phasing Plan may differ from what is shown.
4. The vertical expansion will progressively be filled as Cells 11 and 12 approach final grades.

Cell Development

Initially, Cell 11 of the Landfill will begin to be filled; this is the first area of construction. Concurrent with Cell 11 construction and prior to operation of Cell 11, the following features and structures will be developed or installed:

- New leachate storage tank with secondary containment and leachate forcemain to the leachate storage tank;

- Perimeter access road along at least the west side of the Expansion and providing access through the leachate loadout, ancillary northern entrance, and northern maintenance building; and

- New Stormwater Basin 8 and corresponding perimeter drainage ditches to convey stormwater to the basin.

Construction will continue such that each phase and cell will generally be filled to grade so that final cover may be applied as landfilling activities continue, as shown in **Drawings D30-D37**. If the surface of a fill area has been left inactive for a period greater than 60 days, the area will be covered with one foot of compacted clean soil (intermediate cover). The cover will be sloped to promote drainage and will minimize infiltration into the fill.

No Landfill areas will be developed without adequate stormwater management controls. It is noted that because the stormwater controls have been designed to accommodate the fully developed Landfill, they are also sufficiently sized to handle interim conditions. However, additional temporary measures will be incorporated to divert stormwater away from active landfilling and liner construction areas. Prior to the start of liner construction, diversion berms and drainage ditches will be developed to prevent runoff from impacting construction areas. These perimeter features will intercept the runoff from undisturbed areas before it reaches construction areas.

Construction of subsequent areas will be phased to ensure that adequate Landfill capacity is continuously available. Once construction of a new area is complete and the operating authorization from the IEPA has been received, waste disposal will be diverted from the area currently receiving waste to the newly developed area to establish a protective layer of waste.

The following is a summary of the main points regarding the sequence of construction:

1. Landfill construction will be scheduled to the greatest extent possible so that the initial filling of each area will occur prior to winter.

2. Once constructed and operating authorization has been received from the IEPA, the waste disposal operations will be transferred to the newly constructed cell phase as soon as practical to cover and protect the liner.

3. Only one active face will be utilized during operation unless conditions arise that require more than one active face to be operated at a time. An example of such a condition is when a phase is “topped out” to reach its final permitted grades.

4. Any previously active face or waste disposal area that is inactive for more than 60 days will be covered with intermediate cover consisting of at least one foot of clean compacted soil.

5. Construction of the final cover will commence as soon as practical.

6. Stormwater management controls and monitoring systems for groundwater and landfill gas required for each area will be developed in advance of waste filling and expanded as necessary as filling progresses.

Groundwater Seepage

Excessive groundwater seepage in and around excavation areas during construction can result in inadequate fill subgrade conditions (i.e. too soft to allow the first lift of Compacted Foundation Fill or Earth Liner to be compacted to the specified density), and/or can result in excessive hydrostatic uplift pressures on the completed liner system.

The CQA Officer or designate CQA Officer-in-Absentia shall observe excavations and fill subgrades for evidence of excessive groundwater seepage and notify the Contractor and the Design Engineer in the event that excessive seepage is noted. In such areas, an underdrain collection system will be constructed prior to continuing with construction. Typical undrain collection system details are shown in **Drawing D15**, **D16** and **D18**. Groundwater will be transported via the underdrain control system to sumps which will be constructed similar to those constructed above the liner.

The underdrain collection system will be pumped only during construction and until the placement of waste in the cells results in a fill elevation that counteracts the potential for hydrostatic uplift of the liner system, as calculated in **Appendix J.3-A**. After waste filling has reached the necessary elevation in the cell, the underdrain collection system sump will be shut off, allowing the soil to re-saturate. No other monitoring or abandonment activities will be required for the underdrain collection system once the sump is shut off.

Initial Filling Sequence

After receipt of the operating authorization, waste filling will initiate, and select waste will be placed over the leachate collection drainage layer. The initial waste lift will be placed approximately 5 to 10 feet thick to cover the entire floor. Select fill will be placed against the sidewalls as equipment access allows. The initial waste and select fill layers will serve as a protective and insulating layer over the leachate collection system and synthetic liner. Daily (or intermediate) cover will be placed over the initial lift of waste to serve as a working surface. Subsequent lifts of waste will be covered at the end of each day with daily cover.

Seasonal Construction and Filling Considerations

The anticipated sequence of the Landfill construction and filling is dependent upon variable conditions such as incoming waste volumes and weather conditions. Therefore, typical seasonal conditions and the corresponding construction activities most suited to the temperature and precipitation associated with these seasons have been assumed.

The construction of the liner system and leachate collection system will generally take place in the drier late spring and summer, and possibly during early months of fall. However, if weather permits, construction may occur outside these seasons.

Daily cover placement, haul road construction, fill placement and other necessary activities will take place throughout the year as needed. Construction materials such as pipe, geotextile, and processed gravel for the leachate collection system may be stockpiled on-site to be ready for placement at all times. The proposed sequence of construction will allow for orderly construction and minimize the periods in which there is either a lack or an excess of manpower and equipment.

Placement of Final Cover

Construction of the final cover is recognized to have a direct influence on the amount of leachate generated. Therefore, placement of the final compacted cohesive soil cover will take place as soon as practical. Final cover will be constructed in phases. The compacted cohesive soil final cover will be covered with a low-permeability layer consisting of a 40-mil LLDPE geomembrane overlain by a double-sided geocomposite drainage net and protective soils as shown on **Drawing D15**. The top 6 inches of the protective layer will be capable of supporting vegetation such as grass for erosion protection. The objective will be to establish the stabilized final surface as quickly as possible after the filling has been completed in a particular area.

Material Balance

Soil from future cell excavations, sediment basin construction, and additional borrow areas will be used to meet the needs for daily and intermediate cover, and for construction of the bottom liner, final cover and other engineered features as documented in **Appendix N**. It is anticipated that aggregates for the leachate drainage and collection systems will be obtained from approved off-site sources. The development, operation, and closure of the Landfill will produce a surplus of 2,353,277 yd^3. Surplus soil will not be stockpiled or distributed over closed areas of the Landfill; off-site uses of the soil or off-site stockpile locations will instead be identified through the Landfill's operating life.

It is anticipated that soil for the Landfill development will primarily be derived from site excavations that satisfy the CQA requirements. Any material from offsite sources will comply with all the applicable CQA requirements.

During excavation, material types will be identified and segregated. Excavated materials meeting specifications for clay liner and cover construction will be directly hauled to the area of construction or stockpiled near the areas intended for utilization. In accordance with the conditions of the Siting Ordinance, soil or excavation materials shall not be stockpiled within the Site 2 North Expansion area above elevation 890 feet, and shall only be stockpiled within (not outside) the berm area surrounding the Site 2 North Expansion, except as needed for construction of berms and, to the extent outside the permitted boundary, in compliance with the zoning ordinance. In order to reduce the amount of stockpiling, daily and intermediate cover will be taken as needed from excavation areas.

SECTION 2.4

Stormwater Management Plan

2.4 STORMWATER MANAGEMENT PLAN

Overview of Project

Zion Landfill, Inc. is seeking IEPA approval to expand its currently permitted Zion Landfill (Facility) in Lake County, Illinois. The proposed Site 2 North Expansion includes a vertical waste expansion component (building on top of the currently permitted waste mass); and a horizontal waste expansion component (expanding the waste footprint laterally). The design for the proposed landfill expansion has been developed with appropriate stormwater controls and best management practices (BMPs) in order to safely collect, route, detain and discharge stormwater runoff from the Facility in an environmentally sound manner. These controls offer the opportunity to improve stormwater discharge quality prior to discharge and allow for the controlled release of storm events. The Stormwater Management Plan (Plan) contains design features that meet or exceed the regulations applicable to stormwater management.

As detailed in this report, the proposed Site 2 North Expansion design has been modeled on a detailed level to ensure that each stormwater management element is appropriately sized to prevent overtopping while minimizing the potential for erosion or scour. Some stormwater conveyance features associated with the currently permitted landfill will be used to convey stormwater associated with the expansion. As such, select areas of the existing stormwater management system have been revised, and/or partially or wholly integrated into the proposed expansion. These stormwater features include terrace berms, flume pipes (letdown pipes), perimeter ditches, a stormwater basin, and outlet structures. The proposed expansion design only discusses stormwater facilities utilized and/or affected by the proposed landfill expansion. An overview of the proposed stormwater management system is shown on **Drawing No. D13.**

The results of these analyses demonstrate that the permitted and proposed stormwater control features to be used as part of the expansion are appropriately designed to manage stormwater. In fact, the control features exceed state regulations applicable to stormwater management: all stormwater controls for the Facility have been sized to handle 100-year storm events.

The proposed system is designed to manage stormwater in the area of the landfill and reduce the flooding potential of downstream areas. Stormwater will be directed away from the landfill waste boundary. Stormwater which contacts waste will be contained and treated as leachate and will <u>not</u> discharge to off-site waterways.

Stormwater Regulatory Requirements

Illinois Environmental Protection Agency

The Illinois Environmental Protection Agency (IEPA) provides stringent stormwater control/surface water drainage regulations for Landfill facilities. These include:

35 Ill. Admin. Code, Section 811.103.

35 Ill. Admin. Code, Section 812.110.

Title 35 Ill. Admin. Code, Section 811.103 establishes several requirements for stormwater runoff from disturbed areas.

- ❑ Runoff from disturbed areas during a 25-year, 24-hour storm or smaller is subject to the water quality standards contained in 35 Ill. Admin. Code, Part 304.
- ❑ All discharges from disturbed areas are subject to the permitting requirements within 35 Ill. Admin. Code, Part 309.
- ❑ All discharge structures must be designed to have flow velocities that will not cause erosion and scouring of the natural or constructed lining of the receiving stream channel.

This same section also outlines requirements for the diversion of runoff from undisturbed areas.

- ❑ Diversion facilities must be designed to prevent runoff from the 25-year, 24-hour storm from entering disturbed areas to the extent practical.
- ❑ The diversion structures must be designed to have flow velocities that will not cause erosion and scouring of the natural or constructed channel lining.
- ❑ Runoff from undisturbed areas that becomes comingled with runoff from disturbed areas must be handled and treated as runoff from disturbed areas.

Title 35 Ill. Admin Code, Section 812.110 outlines the specific details that need to be included in the permit application, including a map of the location of structures affected by stormwater runoff from disturbed areas and detailed designs of structures to be constructed. These are included as **Drawing Nos. D13, D20, D21, D22, D23**, **D24, D25 and Appendix M**, respectively.

Lake County Stormwater Management Commission

The Lake County Watershed Development Ordinance, effective October 13, 2020, contains performance standards and other regulations that are applicable to the development of the proposed stormwater management system. These include:

- ❑ 501.05 – Existing depressional storage volume shall be maintained, and the volume of detention storage provided to meet the requirements of the Ordinance shall be in addition to the existing storage.
- ❑ 502.01 – The detention volume required shall be calculated using a rating curve based on maximum release rates of 0.04 cubic feet per second per acre for the 2-year, 24-hour storm event and 0.15 cubic feet per second per acre for the 100-year, 24-hour storm event.
- ❑ 502.04 – The design of the stormwater management systems shall not result in the inter-basin transfer of drainage, unless no reasonable alternative exists.
- ❑ 507.01 – All stormwater facilities shall be provided with:

    A. An emergency overflow structure capable of passing the critical duration base flood inflow rate without damages to downstream structures or property.

B. The top of the impounding structure shall be a minimum of one (1) foot above the design high water level within the emergency overflow structure based on 507.01A.

C. A minimum 8-foot wide safety shelf with a maximum depth of three (3) feet below normal water level sloped back towards the shoreline.

D. Features for maintenance and emergency ingress and egress capability.

- ❑ 600.06(D) – Sediment basins shall have both a permanent pool (dead storage) and additional volume (live storage) with each volume equal to the runoff amount of a 2-year, 24-hour event over the on-site hydrologically disturbed tributary drainage area to the sediment basin. The available sediment volume below normal water level, in addition to the dead storage volume, shall be sized to store the estimated sediment load generated from the site over the duration of the construction period. For construction periods exceeding one (1) year, the 1-year sediment load and a sediment removal schedule may be submitted.

- ❑ 600.12 – Stormwater conveyance channels, including ditches, swales and diversions, and the outlets of all channels and pipes shall be designed to withstand the 10-year frequency storm without erosion. All constructed or modified channels shall be stabilized within 48 hours.

- ❑ Design considerations for ditches, detention basins, outlet structures and all other necessary stormwater control structures.

Benchmark Objectives

Based on the above stormwater regulatory requirements, this Plan has been developed to demonstrate the appropriateness for design of each stormwater control as it conveys the runoff associated with various storm frequencies required for analysis by state and local regulations.

The Facility has been designed to appropriately manage multiple storms, up to and including the 100-year, 1-hour and 24-hour storm events. 1-hour storms typically result in the fastest stormwater velocities in ditches, whereas the 24-hour storms typically produce the greatest volume of water directed to the detention basins. Designing a landfill facility to perform during these 100-year design storms provides a significant additional environmental safeguard above regulatory requirements.

The following benchmark objectives were identified in addition to the regulations above to determine whether the Facility was appropriately designed:

- ❑ Non-erosive flow: Demonstrate that non-erosive flow will be maintained within stormwater terrace berms and drainage ditches for the peak storm event (100-year, 1-hour storm).

  It is noted that flow velocities less than 5 feet per second (fps) in grass-lined terrace berm and ditches are assumed to meet this requirement. If flow velocities were identified to exceed 5 fps for any terrace berm or ditch during modeling, it is recommended that the lining material include riprap or another erosion control material that provides sufficient shearing resistance to prevent erosion.

- ❑ Adequate Size: Demonstrate that stormwater conveyance features will be sufficiently sized to prevent overtopping during the peak flow volume associated with all storm events, including the 25-year and 100-year storms.

- ❑ Minimize Downstream Flooding: Ensure that the detention basins discharge at a lower release rate under proposed conditions than existing conditions to minimize the potential for downstream flooding. It is noted that because the proposed landfill expansion is located on a watershed divide between the Des Plaines River and Lake Michigan watersheds, there are no upstream impacts to be considered.

Existing Conditions

Physiography and Topography

The existing landfill facility is generally bounded to the east by Kenosha Road, to the west by Green Bay Road, to the south by 9th Street, and to the north by a tree nursery, golf course, and residential properties along Kenosha Road. The Site 2 North Expansion will include expansion to the north within property owned by Zion Landfill, Inc. that includes properties previously developed with residential uses and a tree nursery. The residential properties generally have grassed lots with stands of trees. The nursery contains rows of trees and shrubs, open vegetated areas, buildings, and a pond. The expansion area will be bounded to the west by a golf course, to the north by Russell Road, and to the east by Kenosha Road and residential properties.

The proposed expansion area is located in two distinct sub-Watershed boundaries. The western side of the proposed landfill area drains to the Upper Des Plaines River sub-watershed (part of the Des Plaines River Watershed), while the eastern side is part of the Kellogg Creek sub-watershed (part of the Lake Michigan Watershed). **Drawing No. D3** shows an overview of the Facility, including the existing and proposed solid waste sites.

Currently Permitted Stormwater Controls

The existing permitted landfill has a detailed stormwater management system that has been reviewed and approved (permitted) by the IEPA in the Site 2 East Expansion application. Generally, stormwater that falls on the landfill is intercepted by terrace berms or benches and directed to flume pipes (letdown pipes) or downchute ditches. The flume pipes and downchutes convey water down the landfill sideslopes into ditches that follow the perimeter of the landfill. Water passes through energy dissipators prior to discharge into the perimeter ditches, with the purpose of reducing the potential for erosion and/or scour. The perimeter ditches then drain to stormwater basins, which provide temporary storage so that water can be released from outlet structures at a controlled rate. The design of stormwater controls and measures for the proposed expansion incorporate design elements that have proven to be successful at the existing landfill facility.

Existing Landfill Areas and Controls Modified by the Expansion

As previously noted, the proposed landfill expansion will vertically expand on the current landfill and also extend its footprint to the north. As such, the proposed expansion design will both incorporate and modify some existing stormwater controls. The stormwater management system that is described in this application represents all areas that will convey stormwater associated with the proposed Site 2 North Expansion, which includes hydraulically connected areas of the existing landfill that share common stormwater management features. Specifically, the drainage area that is currently routed to the permitted Stormwater Basin 5R will be modified as part of the proposed expansion. Since Stormwater Basin 5R will serve portions of the existing landfill and the proposed expanded landfill, all areas that drain into Stormwater Basin 5R have been fully incorporated into the stormwater analysis.

The existing conditions study area also considers undisturbed areas to be developed and incorporated into the proposed expansion. Please refer to **Figure M.2-1**, located in **Appendix M** for a figure identifying the major stormwater drainage areas and their ultimate discharge location.

Soil Conditions

For the existing (pre-development) conditions in the proposed horizontal expansion areas, local surficial soil boundaries and designations, as identified by Natural Resources Conservation Service (NRCS), were reviewed. Local surficial soils influence the current rate of runoff within the proposed expansion area. Surficial soils are the soils located at the surface and are not necessarily indicative of the subsurface geology. Some of the existing land area within the horizontal expansion area is comprised of soils in the dual hydrologic soil groups B/D or C/D. This means the soils can behave differently depending on whether drained or undrained conditions are exhibited. For all soils within the "Dual Hydrologic Soil Group", an assumption of HSG-D was made. A map of the soil boundaries and a copy of the NRCS soil survey is provided in **Appendix M.3**.

It is noted that portions of the proposed landfill expansion area have been permitted for temporary soil stockpiles that have been or will be in place prior to landfill expansion development. Dedicated basins have been or will be developed to serve these stockpiles and the stockpiles will not increase discharge rates compared to existing conditions. Therefore, these temporary stockpiles have not been included in the pre-development condition. The groundcover and surficial soil type prior to stockpile development are assumed in these locations.

For the proposed conditions, the land covers were determined by the proposed facility design and proposed uses. Major land covers (including open space/grass cover, paved streets and roads, and water) were delineated with AutoCAD Civil 3D 2018 (AutoCAD) and manually imported into stormwater modeling software (HydroCAD). Similar to the approach to the temporary soil stockpiles for existing conditions, the pre-stockpile ground cover was assumed for existing conditions model. Please see **Appendix M.3** for additional information.

Land Cover

A tree nursery is located in the horizontal expansion area. Due to the fact that the surficial soil and land cover type impact stormwater runoff rates, the land cover has been delineated using AutoCAD, and manually imported into HydroCAD. As the existing tree nursery has rolling stock, the "existing" condition periodically changes. Therefore, a simplified land cover type of "woods/grass combination" land use was assumed for a majority of the proposed landfill development area. A figure showing these land covers is provided in **Appendix M.3**.

Wetland Delineation

A delineation study was completed in accordance with the current US Army Corps of Engineers (USACOE) methodology to determine whether wetlands and Waters of the U.S. are located within the proposed landfill development footprint. Only the horizontal expansion footprint was considered, since the vertical expansion area was already assessed in previous Zion Landfill development applications.

The study identified wetlands that were determined to be Isolated Waters of Lake County (IWLC) and different wetlands that were determined to be Waters of the U.S. The locations of the IWLC and Waters of the U.S. were determined by Hampton, Lenzini, and Renwick, Inc. in its report, Wetland Delineation Report for Zion Landfill Site 2 North, Zion, Lake County, Illinois, December 2019.

Prior to disturbing any Waters of the United States, Zion Landfill, Inc. will acquire an individual permit from the USACOE (complying with Section 404 of the Clean Water Act) which will include mitigating loss of Waters of the US and/or jurisdictional wetlands as required by the USACOE. The wetland delineation study and a map of the wetland boundaries located within the proposed landfill development footprint are provided in **Appendix F**.

Drain Tile Survey

A drain tile survey was completed for the proposed expansion area to determine whether upstream or downstream drainage areas would be impacted by the development of the proposed expansion. The results of the survey show that there is one 12-inch concrete drain tile located within the proposed expansion area that drains water from land both within and outside the expansion area. This pipe will be re-routed to allow continued conveyance of water from upstream areas located west of the expansion area. All other drain tiles exclusively drain water from land within the expansion area and will be abandoned when they are encountered during construction activities.

During construction of the horizontal expansion, the 12-inch concrete drain tile described above will be replaced with a 16-inch HDPE pipe located along the western boundary of the facility property. The replacement pipe will be routed toward a Russell Road culvert crossing where water is currently discharged from the existing drain tile. The replacement pipe will be constructed to tie-in to existing upstream outfall systems that service the area located west of the proposed expansion area. Since the replacement pipe diameter is larger than the existing 12-inch drain tile, and since drain tiles within the expansion area will be abandoned and will not contribute flow volume, the replacement pipe is adequately sized. The proposed pipe location is shown in Drawing No. D11.

In the unlikely event that unknown drain tiles are encountered during development, they will be managed to ensure proper drainage and discharge so as not to adversely affect upstream or downstream properties. A copy of the drain tile survey is provided in **Appendix M.9**.

Depressional Storage

Section 501.05 of the Lake County Watershed Development Ordinance requires that the landfill stormwater basins maintain the existing depressional storage volume that will be disturbed by the proposed landfill expansion in addition to the required detention volumes necessary to meet applicable requirements of the Watershed Development Ordinance.

Depressions have been identified in the horizontal expansion area of the Site 2 North Expansion area. The depressions fall within the drainage area of Stormwater Basins 5R and 8. As such, the volumes of these depressions will be maintained as part of the proposed design. An evaluation of depressional storage was conducted in the previous Site 2 East Expansion application and no depressions were found to be disturbed by that expansion, meaning no additional storage was required.

Refer to **Appendix M.7** for depressional storage calculations and additional detention basin sizing calculations.

Floodplain

No 100-year floodplain is present within the proposed landfill expansion footprint, as determined through the review of the flood insurance rate map that covers the facility. The flood insurance rate map used was for Lake County, Illinois, Map Number 17097C0076K, dated September 17, 2013, published by the Federal Emergency Management Agency (FEMA).

Proposed Conditions

Overview

The proposed Site 2 North Expansion will expand the currently permitted landfill vertically over its north sideslopes and horizontally to the north. As such, select areas of the existing stormwater management plan have been revised, and/or partially or wholly integrated into the proposed stormwater management plan. The proposed stormwater management model only considers stormwater facilities utilized and/or affected by the proposed expansion.

The final slopes of the proposed landfill expansion area will have 4H:1V sideslopes and a 10H:1V plateau. The maximum elevation of the proposed landfill expansion area will be approximately 896 feet MSL, which is roughly 34 feet lower in elevation than the Site 2 East peak elevation of 930 ft MSL. The final waste grades shall be overlaid with a final cover which satisfies the requirements of 35 Ill. Admin. Code, Section 811.314. In order to minimize the potential for erosion and scour, the final slopes of the Landfill will be vegetated.

The proposed landfill will be developed with similar controls as the existing landfill, although it is noted that some of these features' dimensions have been modified as appropriate for the new development. However, the overall conveyance strategy remains similar. The stormwater management system has been designed to accommodate fully-developed conditions of the landfill and ancillary facilities. Runoff from the final landform will be directed into existing Stormwater Basin 5R and proposed Stormwater Basin 8 using a series of terrace berms, flume pipes (letdown pipes), downchute ditches, and perimeter ditches.

Stormwater Basin 5R will discharge into the Lake Michigan Watershed and Stormwater Basin 8 will discharge into the Des Plaines Watershed. The management of stormwater collection and the proposed discharge locations have been intentionally developed to minimize inter-basin transfer, to the extent practical. Refer to **Figure M.2-2,** located in **Appendix M,** for a figure identifying the proposed expansion's major stormwater drainage areas and their ultimate discharge location.

Terrace Berms

Terrace berms will be used to intercept stormwater sheet flow, collect runoff, and control erosion along the sideslopes of the Landfill. Terrace berms located on the 4H:1V sideslopes will be constructed as part of the closure of the Landfill in the approximate locations shown on **Drawing No. D13**, which reflect a typical spacing of approximately 35 vertical feet. Each terrace berm is named according to the watershed that contributes to the stormwater flow into the terrace berm. Refer to **Drawing No. D20** for a typical terrace berm detail.

The terrace berms have been sized to accommodate the peak runoff flow rates and volumes associated with the modeled 100-year storm event. Calculations demonstrating that the terrace berms are sufficiently designed are presented in **Appendix M.6**.

Flume Pipes (Letdown Pipes) or Downchutes

Flume pipes and downchute ditches will be used to convey the stormwater collected by the terrace berms down the slope of the Landfill into the perimeter ditches. The only downchute ditch segment modeled as part of this stormwater analysis was previously permitted and approved in the Site 2 East Expansion Application. This downchute ditch has already been constructed at the Landfill.

Most of the downslope conveyance features at the Landfill will consist of flume pipe systems. Flume pipes will be a constant diameter for their entire run (e.g the same above and below on each feeder line). Proposed flume pipes will be either 16, 18, or 24-inch ADS-N12 corrugated polyethylene (PE) pipes with smooth interior or equivalent materials. Proposed Flume Pipe Runs 3A/3B and 4A/4B are designed to have two (2) flume pipe trunk lines that will run parallel to each other and perpendicular to the 4H:1V proposed sideslopes of the final cover. Both proposed flume pipe runs will consist of dual, parallel flume pipes that collect and convey stormwater from separate terrace berm sections, as provided in **Appendix M.6**.

Existing flume pipes modeled as part of the proposed conditions evaluation include Flume Pipe Runs 1, 2, 5, and 6 and consist of corrugated PE pipes with smooth interior. Flume Pipe Runs 5 and 6 were designed to be 12-inch and 16-inch corrugated PE pipes with smooth interior, respectively, in the Site 2 East Expansion Application and have not yet been constructed. For the Site 2 North Expansion, Flume Pipe Run 5 has been modified to a 24-inch corrugated PE pipe with smooth interior and Flume Pipe Run 6 has been modified to a 18-inch corrugated PE pipe smooth interior to accommodate stormwater flows from both the permitted and proposed landfill expansion area.

All flume pipes are designed to handle runoff flow rates from the peak 100-year storm events. The locations of the flume pipes are shown on **Drawing No. D13** and typical details of both features are shown on **Drawing Nos. D21 and D22**. Calculations demonstrating that the flume pipes are sufficiently sized are presented in **Appendix M.6**.

Perimeter Ditches

As shown on **Drawing No. D13,** ditches are used to convey stormwater around the perimeter of the Landfill to the basins. The landfill perimeter ditches have been designed with excess capacity to convey the flow rates of the peak 100-year storm events. Therefore, the design exceeds the requirements contained within 35 Ill. Admin. Code, Section 811.103, which requires that all ditches pass the 25-year, 24-hour storm event.

The landfill perimeter ditches are designed for low maintenance after the Landfill is vegetated. All landfill perimeter ditch bottoms will be vegetated with grasses or lined with riprap or an alternate erosion control material. The proposed landfill perimeter ditches are designed with sideslopes of 3H:1V, a bottom width from 0 to 10 feet, and a ditch slope ranging from 0.0050 ft/ft to 0.0057 ft/ft. The wide grassed bottoms will promote sedimentation and foster a natural environment. Existing perimeter ditches modeled as part of the proposed conditions evaluation vary in bottom width and depth while maintaining 3H:1V sideslopes. Existing perimeter ditches range from V-notch channels (0-ft bottom width) to a bottom width of 10 feet, depths ranging from 2.5 - 3.0 ft, and slopes ranging from 0.0039 to 0.0080 ft/ft. Calculations demonstrating that the perimeter ditches are sufficiently sized are presented in **Appendix M.6**.

Culverts

Based on the proposed expansion design, two (2) sets of existing culverts located along the landfill perimeter ditches have been re-evaluated as additional stormwater will be directed through these features. In addition, two (2) sets of proposed box culverts will be utilized to convey stormwater underneath perimeter roadways or directly into sediment forebays at the Landfill. Both existing and proposed culvert locations vary in pipe size, pipe material, slope, and the number of pipes at each culvert location. Key parameters for both existing and proposed culverts are provided in **Table 2.4-1** and depicted in **Drawing No. D25.** It is noted that traditional circular culverts may be installed in lieu of box culverts at the discretion of owner and engineer provided that equivalent flow capacity is provided.

Culverts have been sized to handle the peak 100-year storm events. Calculations demonstrating that the culverts are sufficiently sized are presented in **Appendix M.6**.

TABLE 2.4-1
KEY PARAMETERS FOR EXISTING AND PROPOSED CULVERTS

Culvert Name	Existing or Proposed	Slope (ft/ft)	Pipe Diameter (in.)	Number of Pipes	100-yr Peak Depth in Culvert(s) (in.)
Culvert A	Existing	1.00	36	2	22.44
Culvert 3	Existing	0.97	36	3	22.80
Culvert 1	Proposed	0.45	48-W by 24-H	4	20.52
Culvert 2	Proposed	0.95	48-W by 24-H	2	18.24

Note: Traditional culverts may be installed in lieu of box culverts at the discretion of owner and engineer provided that equivalent flow capacity is provided.

Drain Tile

A mature stand of trees is located in the proposed landfill expansion area near Kenosha Road. The landfill has been designed such that these trees will be preserved. Stormwater that falls in the location of the trees will be allowed to infiltrate into the ground, similar to existing conditions. However, as the ground surface in the location of the trees generally flows to the east, the potential exists for water to collect along the toe of the Kenosha Road screening berm during large rain events. Therefore, a drain tile will be installed near the toe of the slope of the screening berm to enhance stormwater drainage.

In addition to the area of preserved trees, the new drain tile system will also be placed along the property line in the northeast corner of the proposed landfill expansion area. Stormwater will be allowed to infiltrate into the ground between adjacent properties and the perimeter screening berm along the toe of the slope during large rain events. The drain tile system will enhance stormwater drainage in this area and discharge collected stormwater to Stormwater Basin 8.

Although the ultimate drainage discharge point of the entire drain tile system will be Stormwater Basin 8, the significant lag time prior to discharge will result in negligent flows during storm events. Therefore, the drain tile conveyance feature is not modeled within HydroCAD. The drain tile will be sized and sloped based on the recommendations of qualified contractors at the time of installation.

Stormwater/Sedimentation Basins

The design of the proposed Landfill Expansion will require the construction of one (1) stormwater/sedimentation basin (Stormwater Basin 8) and the utilization of existing Stormwater Basin 5R. Both stormwater basins are designed to function as wet-bottomed basins to maximize stormwater holding time, which will promote sedimentation within the basin for improved water quality prior to discharge. Both stormwater basins are designed with sedimentation forebays at inlet locations to facilitate sediment dropout. Stormwater Basin 5R has one forebay (associated with its one inlet on the north side) and Stormwater Basin 8 has two forebays (associated with inlets on the east and west sides). Both Stormwater Basin 5R and Stormwater Basin 8 have been designed with an 8-foot safety ledge that is located 3-feet below the normal water level of each basin.

Stormwater Basin 8 will accommodate stormwater from the western and northern portions of the landfill expansion area and stormwater from the north slope of the existing landfill that previously drained to Stormwater Basin 5R. The eastern portion of the landfill expansion area, as well as the east and southeast areas of the existing landfill, will drain to Stormwater Basin 5R.

The basins have both been designed with sufficient dead storage (water below the normal water level) to accommodate one year's worth of estimated sediment loading in addition to storage volume equal to the runoff volume of a 2-year, 24-hour event over the disturbed drainage area, as required by the Lake County Watershed Development Ordinance (WDO) published by the Lake County Stormwater Management Commission, effective October 13, 2020. The basins have also been designed with sufficient live storage (water above the normal water level) to offset all storage volume from existing depressional areas that will be removed as part of landfill development, in addition to the 2-year, 24-hour event over the disturbed drainage area, as required by the Lake County WDO.

Stormwater basin designs have been evaluated to demonstrate that they meet requirements detailed in the Lake County Watershed Development Ordinance and Illinois Administrative Code. Please see **Appendix M.7** for calculations demonstrating appropriate design and function of the detention basins.

Stormwater/Sedimentation Basin Outlet Structures

Both stormwater basins will contain outlet structures designed to facilitate the controlled release of stormwater. The existing permitted outlet structure at Stormwater Basin 5R is a riser discharge pipe. The structure contains a 30-inch vertical riser pipe connected to a 30-inch discharge pipe, as shown in **Drawing No. D23**. The existing outlet structure within Stormwater Basin 5R will perform in a manner that will maintain compliance with state regulations while accommodating the proposed landfill expansion area. As such, there are no proposed changes/additions to the existing Stormwater Basin 5R outlet structure.

The outlet structure within the proposed Stormwater Basin 8 will consist of a standpipe equipped with multiple rows of orifices, as shown in **Drawing No. D24**. The row elevations and hole sizes have been selected based on modeling such that stormwater holding time is maximized for each storm while meeting discharge rate and freeboard requirements specified in the Lake County Watershed Development Ordinance. This design approach will promote improved stormwater quality by increasing hold times and facilitate sediment drop-out in conjunction with the wet-bottom basin design, maximizing the potential for improved stormwater quality prior to discharge. It is also noted that orifice design has been completed to ensure that the basin high water level for the 100-year 24-hour storm remains below the inlet pipe invert elevations so that water does not back up into the perimeter ditches for storms equal to or smaller than the 100-year design storms.

The top of the standpipe is designed to be open at an elevation slightly higher than the 100-year 24-hour design storm high water level. Due to the fact that Stormwater Basin 8 is located inside of a screening berm and perimeter screening barrier wall, Stormwater Basin 8 will use pipe flow as its spillway mechanism. The spillway overflow is designed to convey the 100-year, 1-hour and 24-hour storm events.

In accordance with the conditions of the Siting Ordinance, valves or other equivalent devices shall be installed on the stormwater outlet device for Stormwater basin 8 prior to waste placement at the Site 2 North Expansion.

An evaluation was performed to ensure that the spillways for both basins are appropriately sized to convey the 100-year, 1-hour storm produces the highest inflow rate and the 100-year, 24-hour storm produces the largest volume while maintaining one foot of freeboard from the basin crest. The evaluation sets the initial water level of each basin to be equal to the spillway elevation and forces all discharge through the spillway only (e.g. no flow through the submerged standpipe orifices). Based on this evaluation, it is determined that the spillways are capable of passing the inflow rate associated with the 100-year, 1-hour and 24-hour design storms. In addition, more than one-foot of freeboard is maintained below the basin crest elevation for the 100-year, 1-hour and 24-hour design storms. These analyses demonstrate the appropriateness of the spillway designs.

Both outlet structures have been designed to meet the maximum allowable release rates specified Section 502.01 of the Lake County Watershed Development Ordinance, as shown in **Table 2.4-2**:

TABLE 2.4-2 MAXIMUM ALLOWABLE RELEASE RATE COMPARISON					
Basin	**Stormwater Basin Inflow Area (acres)**	**Storm Event**	**Lake County WDO Maximum Release Rate (cfs/acre)**	**Maximum Allowable Discharge Rate (cfs)**	**Anticipated Peak Discharge Rate (cfs)**
Basin 5R	53.0	2-Year, 24-Hour	0.04	2.1	1.60
		100-Year, 24-Hour	0.15	8.0	3.28
Basin 8	148.0	2-Year, 24-Hour	0.04	5.9	5.59
		100-Year, 24-Hour	0.15	22.2	22.16

In addition, an evaluation was performed as a best engineering practice to ensure that the basins discharge at a lower release rate under proposed conditions than existing conditions to minimize the potential for downstream flooding and damages to downstream structures or property, as demonstrated in **Appendix M.7**. No upstream impacts are considered because the proposed landfill expansion is located on a watershed divide.

Hydrologic Analyses

Methodology Overview

The stormwater management system that has been modeled represents all areas that will convey stormwater associated with the proposed Site 2 North Expansion. As such, the area for analysis in both the existing and proposed conditions includes all land development areas that will be hydrologically disturbed and hydraulically connected portions of the existing facility that share common stormwater management features. Specifically, the existing landfill facility currently uses Stormwater Basin 5R. With the new landfill expansion design, stormwater from both the existing landfill and the proposed landfill expansion area will be directed to Stormwater Basin 5R; therefore, all areas that drain into Stormwater Basin 5R are delineated for evaluation.

A detailed stormwater analysis for Stormwater Basin 5R was developed as part of the Site 2 East Expansion, which was reviewed and approved by the Lake County Stormwater Management Commission and the IEPA. As such, the permitted evaluations and discharge rate results for Stormwater Basin 5R, which are used in the existing conditions model, have not been remodeled. Instead, the total discharge rates reported in the Site 2 East Expansion calculations are reported and added to the additional discharge rates for the horizontal expansion area. This method allows a comparison between existing and proposed discharge rates over equivalent areas. For the proposed conditions, the contributing area stormwater management features will change for Stormwater Basin 5R and are therefore fully modeled.

AutoCAD was utilized to delineate key features, and the computer model HydroCAD was used to develop discharge rates and volumes for various storm events for each stormwater feature described in this Plan. HydroCAD is a computer aided design program used to model hydrology and hydraulics of stormwater using either TR-20 or TR-55 procedures developed by the Soil Conservation Service (now the Natural Resource Conservation Service).

The HydroCAD model was evaluated for both 1-hour and 24-hour durations for the 2-year, 10-year, 25-year, and 100-year recurrence intervals. All modeled storm events were utilized to evaluate regulatory compliance with the Illinois Administrative Code and the Lake County Watershed Development Ordinance and to demonstrate that the additional environmental safeguard benchmark objectives were met.

The stormwater modeling methodology used the following analysis methods, as further describe in subsequent text:

Runoff Calculation Method:	SCS TR-20
Reach Routing Method:	Storage Indication Plus Translation Method
Pond Routing Method:	Storage Indication Method (Modified-Plus)
Storm Distribution:	Updated Huff Distribution - Bulletin 75 (Rainfall Distributions for Illinois) 1st Quartile, 1-hour storms 3rd Quartile, 24-hour storms
Unit Hydrograph:	SCS
Antecedent Moisture Condition:	2

The Soil Conservation Service (SCS), now renamed the Natural Resources Conservation Service (NRCS) developed methods TR-20 and TR-55 as standardized stormwater modeling. Both provide similar results; the main differentiation in methodology is based on the use of chart-based solutions vs. computer modeling. TR-20 is the computer-based modeling approach that is more complex and generally considered slightly more accurate than TR-55. TR-55, frequently called the “tabular method” was developed after TR-20 to help simplify the modeling process. As such it was developed to utilize chart-based solutions to use the SCS runoff equation. For the purpose of this hydrologic model, TR-20 methodology was used.

Model Input Parameters

Precipitation Data

Precipitation data used to determine stormwater impacts for the study area was obtained from Appendix I of the Lake County Watershed Development Ordinance (WDO) published by the Lake County Stormwater Management Commission, effective October 13, 2020. This reference and an associated discussion about rainfall totals and distributions are included in **Appendix M**. The total precipitation and storm durations are summarized in **Table 2.4-3**.

TABLE 2.4-3
STORM FREQUENCIES AND VOLUMES

Recurrence Interval	1-Hour (inches)	24-Hour (inches)
2-Year	1.57	3.34

10-Year	2.42	5.15
25-Year	3.03	6.45
100-Year	4.03	8.57

Subcatchment Boundaries

As previously noted, the stormwater management system that is described in this application represents all areas that will convey stormwater associated with the proposed Site 2 North Expansion, which includes hydraulically connected areas of the existing landfill that share common stormwater management features. Specifically, the drainage area that is currently routed to the permitted Stormwater Basin 5R will be modified as part of the proposed expansion. Since Stormwater Basin 5R will serve portions of the existing landfill and the proposed expanded landfill, all areas that drain into Stormwater Basin 5R have been fully incorporated into the stormwater model.

For the analysis of the proposed Landfill, the study area was subdivided into multiple subcatchments (also known as watersheds). Subcatchment boundaries were delineated using AutoCAD based on topographic breaks within the areas to be hydrologically disturbed. After delineation, subcatchment areas were manually imported into HydroCAD. Please see **Appendix M.2** for figures that show the subcatchment boundaries and additional information.

Runoff Coefficient Variables

In order to determine the runoff coefficients for the existing (pre-development) and proposed (post-development) conditions, land cover and surficial soil types were considered. Each land cover was assigned a run-off coefficient for each Hydrologic Soil Group based on TR-20/TR-55 standard values that reflect these cover types. Based on land cover and hydrologic soil group, HydroCAD determines the weighted curve number for each subcatchment area.

For the existing (pre-development) conditions in the proposed horizontal expansion areas, local surficial soil boundaries and designations, as identified by NRCS, were used in HydroCAD to determine runoff coefficient variables. Land covers for these areas were determined based on a review of aerial photography and the topographic survey. As the existing tree nursery has rolling stock, the "existing" condition periodically changes. Therefore, a simplified land cover type of "woods/grass combination" land use was assumed for a majority of the proposed landfill development area.

It is noted that portions of the proposed landfill expansion area have been permitted for temporary soil stockpiles that have been or will be in place prior to landfill expansion development. Dedicated basins have been or will be developed to serve these stockpiles and the stockpiles will not increase discharge rates compared to existing conditions. Therefore, these temporary stockpiles have not been included in the pre-development condition. The groundcover and surficial soil type prior to stockpile development are assumed in these locations.

For the proposed conditions, the land covers were determined by the proposed facility design and proposed uses. Major land covers (including open space/grass cover, paved streets and roads, and water) were delineated with AutoCAD and manually imported into HydroCAD. Please see **Appendix M.3** for additional information.

Time of Concentration

The time of concentration, defined as the longest amount of time a waterdrop would take to travel from the headwater of a subcatchment area to its downstream edge (i.e. prior to being managed by a downstream element) was delineated in AutoCAD and entered for each subcatchment in HydroCAD. A discussion of how the flow paths are used to calculate time of concentration is further discussed in **Appendix M.4**.

Stormwater Conveyance Features

Stormwater conveyance features with defined stormwater flow paths (also called “reaches” in stormwater modeling terminology) include existing and proposed terrace berms, flume pipes, downchute ditches, perimeter ditches, and culverts at this Landfill. For each conveyance feature, the dimensions, length, and slope were manually inputted into the model. The model assumes that runoff enters each conveyance feature at the same point and flows along the entire length of the structure.

A Manning’s coefficient (unitless coefficient of a surface’s hydraulic roughness) was also applied to each reach based on its anticipated material properties. Hydraulic roughness is the measure of the amount of frictional resistance water experiences when passing over the material. The Manning’s coefficients were selected from HydroCAD’s lookup tables for each material type. Please see **Appendix M.6** for more information.

Stormwater/Sedimentation Basins and Outlet Structures

The storage volumes of the stormwater/sedimentation basins were modeled by entering the area at each minor contour interval to determine incremental detention volumes. The outlet structures were modeled by inputting the standpipe and orifice diameters, elevations, number of orifices at each elevation, outlet slope, and Manning’s coefficient for the pipe material. The normal water elevation was specified as the lowest orifice invert of the standpipe outlet. Spillways were modeled by defining the dimensions of the standpipe/weir structure, elevation of features, and Manning’s coefficient, if applicable. The outlets are inputted in the order in which stormwater will flow, so that HydroCAD can determine which element controls the flow based on sequence and element restrictions.

Please see **Appendix M.7** for more information.

Key Model Results

The HydroCAD results for both 1-hour and 24-hour durations were analyzed to determine which storm duration produces the larger peak discharge and detention requirements. Results of the hydrologic analysis indicate that the 1-hour duration produces a larger peak discharge rate. Therefore, all stormwater conveyance features were designed to adequately handle the peak 100-year, 1-hour storm event.

The stormwater/sedimentation basins were sized to handle both the 100-year, 24-hour and 100-year, 1-hour storm events because these storms produce the greatest stormwater volumes. Model results show the design of each basin is appropriate (stormwater does not overtop the basin) for either storm event. Specifically, both basins were sized to handle the peak inflow rate associated with the 100-year, 1-hour storm, and the peak volume and associated high water level associated with the 100-year, 24-hour storm for the proposed conditions.

All stormwater conveyance features were found to be appropriately sized to convey the 100-year storm events, surpassing all benchmark requirements and local, state, and federal requirements. Key findings include the following:

1) All terrace berms are appropriately sized to pass the peak discharge of all modeled storm events without overtopping or scouring (exhibiting erosive flow). (See **Appendix M.6**).

2) All flume pipe runs (four new, two modified from currently permitted design) serving the expansion can safely convey all modeled storm events. The modified flume pipe runs (Flume Pipe Run 5 & 6), which has not yet been constructed, will be equipped with an energy dissipator structure as was previously permitted. However, the dimensions of the energy dissipator structures (Energy Dissipator 5 & 6) will be modified to accommodate increased flow volumes. For all other proposed flume pipe runs, riprap aprons will be used at flume pipe outlets as the structural feature to reduce exit velocities. Both energy dissipation methods will minimize erosion and scour of perimeter ditch segments (See **Appendix M.6**).

3) All stormwater perimeter ditches are appropriately sized to convey all modeled storm events. Stormwater perimeter ditches PD-3, PD-4, PD-5, and PD-6 exhibit flow velocities greater than 5 fps. Riprap or another appropriate erosion control material is recommended for proposed ditch segments PD-5 and PD-6. Existing ditch segments PD-3 and PD-4 have not demonstrated erosion or scour in their constructed condition and no additional subcatchment area is proposed for these ditches, therefore, erosion control material is not deemed necessary for these existing ditches. In the event that routine erosion or scour is observed, these ditch segments will be lined with riprap or another appropriate erosion control material. Erosive flow rates are not anticipated in any other stormwater perimeter ditches that were modeled as part of this analysis (See **Appendix M.6**).

4) All culvert locations are appropriately sized to convey all modeled storm events. None of the culverts will exhibit full-flow conditions, nor will they surcharge or overtop into the surrounding stormwater perimeter ditches (See **Appendix M.6**).

5) The stormwater/sedimentation basins are appropriately sized (See **Appendix M.7**):

    a. The basins provide live storage for both the 2-year 24-hour storm over the disturbed drainage area and additional live storage capacity to offset depressional storage volume that will be removed as part of development.

    b. The basins provide dead storage for both the 2-year 24-hour storm over the disturbed drainage area and additional dead storage capacity to accommodate sediment accumulation estimated to be generated over a one-year period.

    c. The basin high water level for the 100-year 24-hour storm remains below the inlet pipe invert elevations and spillway (See **Appendix M.7**).

6) The basins discharge at a lower release rate under proposed expanded conditions than existing conditions (permitted Site 2 East permitted design), minimizing the potential for downstream flooding (See **Appendix M.7**).

7) The spillway structures are appropriately sized to convey the 100-year, 1-hour and 24-hour design storms while maintaining one-foot of freeboard in the event that the primary discharge structure is not functional (See **Appendix M.7C**).

8) The management of stormwater collection and the proposed discharge locations has been intentionally developed to minimize inter-basin transfer (transfer of catchment area between existing watersheds) to the extent practical (See subcatchment boundaries, **Appendix M.2**).

Runoff Volume Reduction Hierarchy

The proposed landfill expansion has been strategically designed to meet the waste disposal needs of City of Zion and the surrounding communities while meeting Runoff Volume Reduction requirements established by the Watershed Development Ordinance Section 503, as summarized below:

Preserving Natural Resource Features

The proposed design has been developed to preserve natural resource features to the greatest extent deemed feasible. Approximately 8 acres of mature trees along Kenosha Road has been preserved. The existing depressional storage volume in the horizontal expansion area will be preserved within the storage volume of the proposed sediment basins, as previously described. Refer to **Appendix M.7** for depressional storage calculations.

Minimization of Impervious Surfaces

The proposed development will be predominantly pervious, including the landfill final cover, mature tree stand, and screening berms. The proposed final cover system includes a 3-foot thick soil cap that will be vegetated. Minimal impervious surfaces will be constructed within the proposed expanded facility boundary to facilitate operation of the landfill, primarily for the purpose of ingress and egress of vehicles and a transportation route surrounding the waste boundary. Roads have been developed as narrow as practical while maintaining safe driving conditions.

Enhancement of the Infiltration and Storage Characteristics

Both stormwater/sedimentation basins receiving runoff from the proposed expansion are designed to detain water and allow sediment drop-out prior to discharge. Sedimentation forebays are included in both basin designs for additional dead storage and to provide additional water quality benefits. All live and dead storage requirements specified in the Watershed Development Ordinance are met or exceeded, as demonstrated in **Appendix M.7** while providing appropriate freeboard.

Use of Channels with Native Vegetation

All stormwater conveyance ditches, terrace berms, and benches will be vegetated in areas that can accommodate anticipated flow velocities.

Structural Measures that Improve Water Quality and Volume Reduction

Stormwater/sedimentation basins are designed with sedimentation forebays to improve water quality prior to off-site discharge. Outlet structures at each basin have been designed to restrict the flow rate to meet acceptable discharge rates, as defined within the Lake County Watershed Development Ordinance.

Structural Measures that Provide Volume Reduction and Other Rainwater Harvesting Practices

Water that collects in the stormwater basins will be harvested for dust suppression measures as a part of routine landfill operations.

Measures that Provide Water Quality and Quantity Control

The stormwater/sedimentation basins have been designed to offset depressional storage that exists in pre-development conditions. In addition, the basins have been designed with greater storage volume than required by the Watershed Development Ordinance, providing significant detention volume for storms larger than the 100-year design storm while maintaining a controlled release. The basins have also been designed with sedimentation forebays to improve water quality prior to off-site discharge.

Measures that Provide Water Quantity Control

As discussed above, outlet structures at each basin have been designed to restrict the flow rate to meet acceptable discharge rates, as defined within the Lake County Watershed Development Ordinance.

Stormwater Controls During Cell Development

The development of the perimeter ditches and stormwater/sedimentation basins will be phased to correspond with development of the expansion. No Landfill areas will be developed without appropriate stormwater management controls. It is noted that because the stormwater controls have been designed to accommodate the entire, fully developed Landfill, they are also sufficiently sized to handle interim conditions.

Stormwater Basin 5R is used as part of the existing landfill and will therefore be in operation prior to initial construction activities. Stormwater Basin 8 and stormwater perimeter ditch segments leading to Stormwater Basin 8 will be constructed once horizontal expansion development necessitates its use. Runoff from disturbed areas will be directed to the developed basins or temporary stormwater management structures prior to discharge.

Prior to the start of liner construction, diversion berms and drainage ditches will be developed as necessary to prevent runoff from impacting construction areas. These perimeter features will intercept the runoff from undisturbed areas before it reaches the construction areas (disturbed areas), and the runoff will be conveyed to the Landfill perimeter as practical. Any stormwater that collects within the Landfill excavation will be routed to temporary stormwater collection sumps. Similarly, any rainfall which collects on the liner and leachate collection system prior to the placement of waste will be pumped into the stormwater management system.

Once waste placement begins within a new cell, stormwater which contacts waste or collects within the leachate collection system will be treated as leachate, in accordance with the leachate management section of this Application (**Appendix K**). However, temporary diversion berms will be constructed around the active landfilling areas to the extent practical in order to divert stormwater from adjacent daily, intermediate and final cover slopes before it contacts any waste, thereby preventing it from coming in contact with waste. These temporary berms will divert stormwater runoff to the perimeter collection channels or to below grade stormwater collection sumps located within the excavation. The temporary berms will complement the permanent perimeter ditches and berms which surround the active cell and prevent excavation side slope runoff from entering the active disposal area.

NPDES Requirements

A National Pollutant Discharge Elimination System (NPDES) permit shall be required prior to the commencement of any construction activities which disturb more than one acre. The site currently maintains a NPDES Permit, which is provided in **Appendix M.10**.

Final Grading

The final slopes are designed at a grade capable of supporting vegetation to minimize erosion. These slopes will drain runoff from the cover and prevent ponding. Vegetation will be promoted on all reconstructed surfaces to minimize wind and water erosion of the final protective cover. In addition, terrace berms will be constructed on the final landform to collect runoff and control erosion along the slopes of the Landfill as shown on **Drawing No. D11**.

Vegetative Soil Stabilization

A grass seed mixture will be incorporated into the upper surface of the protective soil layer. The mixture selected will be amenable to the soil quality/thickness, slopes and moisture/climatological conditions that exist without the need for continued maintenance and with minimal potential for root penetration into the compacted final cover.

Landscaping or seeding professionals knowledgeable of local soil and climatological conditions will be consulted in determining the specific seed mixtures, necessary soil amendments, and application rates based upon specific seasonal conditions at the time of closure. Finalized areas of the Landfill will be seeded as soon as practical, with seeding usually conducted in the spring or fall.

Access Roads

Access roads leading to the active waste disposal area and other frequently traveled onside roads will either be paved or surfaced with a suitable thickness of aggregate. Traffic areas will be maintained to prevent tracking of mud from the active face in order to improve stormwater quality and to control dust.

Erosion and Sediment Control

Erosion control techniques will be used in addition to the foregoing to minimize the generation of sediment in the runoff from disturbed areas. These techniques will not only minimize sediment erosion but will improve the water quality of the stormwater runoff. These may include, but not be limited to:

1. *Barrier Filters*. Barrier filters, e.g. silt fences, wattles, rock checks, etc., are intended to filter sediment from runoff in areas where runoff is not routed into a detention basin or sediment trap. Barrier filters will be used for both sheet flow and channel flow. Barrier filters will be used at a minimum along the entire length of all disturbed slopes that are being directly discharged off-site until permanent

vegetation has been established and sediment control is no longer necessary. Barrier filters placed within channels will be spaced at approximately 300-foot intervals.

Barrier filters placed on slopes shall be installed parallel to the contours. When used around inlets, as much filter area as possible will be provided. For channel flow application, the barrier shall be extended to such a length that the ends of the barrier are higher in elevation than the top of the expected flows. Barrier filters will be routinely inspected in accordance with the stormwater pollution prevention plan and best management practices.

2. *Vegetative Filter*. Vegetative filters provide biological filtration to improve water quality where concentrations of sediment are high and flow velocities are relatively low. Vegetative filters may be used along drainage-ways or property lines. Vegetative filters may also be used on the side slopes of the detention basin to filter sediment from overland flow.

3. *Terrace Berming.* Terrace berms will be constructed as necessary to intercept sheet runoff and direct it into downslope ditches or flume pipes. Terrace berms will be installed at locations selected by the site engineer.

4. *Stormwater/Sedimentation Basins*. Stormwater runoff from disturbed areas typically contains sediment. The sediment includes soil that erodes off of earth surfaces and aggregates that accumulate on paved surfaces. Stormwater from the landfill expansion area will be directed the stormwater/sedimentation basins. These basins have been designed with sedimentation forebays to remove sediment from the stormwater runoff. In addition, the basin perimeter will be lined with vegetation to further facilitate sediment knock-out and water quality benefits.

5. *Energy Dissipator Features*. At points of concentrated flow (such as where there is a quick change in elevation or a change in material use), an energy dissipater, riprap, or other approved erosion control material will be used to prevent erosion and scouring.

Inspection and Maintenance

Temporary and permanent erosion control measures will be maintained and repaired as needed to ensure continued performance of their intended function. This program will consist of performance checks of facilities and grades, remedial grading, sedimentation cleaning, vegetative care and maintenance. Inspections will be performed at an appropriate frequency in compliance with the Landfill NPDES permit and Solid Waste permits. Maintenance includes clearing of sediment from barriers and the basins. Sediments will be dredged from the sedimentation basins as necessary to maintain adequate stormwater detention and functionality of the outlet structures. Sediment removed from the barriers and the basins will not be placed in areas without adequate BMPs in-place. As necessary, runoff collection features will be cleaned, regraded, relined, rip-rapped, etc., to restore design capacities and correct problem areas. A written record of all inspections and maintenance will be prepared and placed in the facility Stormwater Pollution Prevention Plan (SWPPP), which will be kept at the site.

Conclusion

The stormwater management system has been designed and is proposed to be operated in a manner that meets local, state, and federal requirements. The discharge rates will be controlled to facilitate sedimentation and to prevent flooding as well as maintaining the drainage conditions of off-site areas located upstream or downstream of the Landfill. Stormwater will be controlled to prevent contact with waste, and stormwater which contacts waste will be contained and treated as leachate. Erosion control techniques and best management practices will be used to minimize the generation of sediment in the runoff from disturbed areas. These techniques will not only minimize sediment erosion but will improve the water quality of the stormwater runoff.

SECTION 2.5

Construction Quality Assurance Plan

2.5 CONSTRUCTION QUALITY ASSURANCE

Introduction

A Construction Quality Assurance (CQA) Plan has been developed for the Zion Landfill Site 2 North Expansion and is included in **Appendix O** of this application. The CQA Plan provides an organizational framework for testing, observation and monitoring activities that will be performed during facility construction in order to document that the constructed facility will meet or exceed all design criteria, drawings and specifications. The CQA Plan also outlines the organization, the implementation and the review of the various CQA activities, the responsibilities of the parties involved in the program, and provides sampling and testing programs to be carried out during the construction of critical facility components. The ultimate goal of the CQA program is to provide a means of evaluating and controlling the quality of the constructed facility so that the intent and minimum material and construction specifications of the design have been met.

The CQA Plan will become effective upon receipt of the landfill expansion development permit from the IEPA. The CQA Plan is subject to change upon IEPA approval as a result of improved construction materials and methods, changed conditions, etc.

Please see **Appendix O** for the CQA Plan.

SECTION 2.6

Operating Plan

2.6 OPERATING PLAN

Introduction

An operating plan has been prepared that addresses proposed procedures for facility operations at the Zion Landfill Site 2 North Expansion (Landfill) and for maintenance and monitoring of the engineered systems at the facility. These procedures have been developed based on the applicant’s knowledge regarding safe and efficient landfill operations, as well as regulatory requirements. The Landfill will operate at all times to protect the public health, safety and welfare under the direction of an experienced operator certified by the Illinois Environmental Protection Agency (IEPA).

The operating plan is based on the applicable landfill requirements contained in 35 Ill. Admin. Code Parts 811 and 812, the IEPA Bureau of Land permit for the existing facility, and Federal landfill regulations. The operating plan also incorporates operating provisions negotiated between Zion Landfill, Inc., Lake County, and the Solid Waste Agency of Lake County during the amendment of the existing host agreement between the parties (see **Appendix C.2**). Siting conditions are addressed in the operating plan and throughout this application to the IEPA. The operating plan is subject to change based on regulatory changes, permit conditions, and/or requested IEPA approval.

Please see **Appendix R** for the Operating Plan.

SECTION 2.7

Groundwater Impact Assessment

2.7 GROUNDWATER IMPACT ASSESSMENT

Introduction

This Groundwater Impact Assessment (GIA) has been performed to demonstrate that the site-specific setting (geology and hydrogeology) and the proposed Site 2 North Landfill Expansion design (which was developed with the geology and hydrogeology in mind) are protective of the public health, safety, and welfare. In other terms, the site geology and hydrogeology and design have been conjoined to each other and the GIA evaluates how the landfill functions in this setting.

This GIA has been prepared in general accordance with 35 Ill. Admin. Code Section 811.317 and 812.316.

The existing Facility consists of two units that have ceased acceptance of waste and are closed, as well as the currently active unit, referred to as the Site 2 Landfill (Landfill). The currently operated Landfill, which is proposed to be expanded as described in this application, has undergone two previous expansions. The Landfill is permitted by the Illinois Environmental Protection Agency (IEPA)(Site No. 0978020002).

The original landfill, referred to as Old Site 2, is a non-hazardous solid waste unit that was regulated under 35 IAC, Part 807. Old Site 2 commenced landfilling operations on December 23, 1981, pursuant to IEPA Permit No. 1980-24-DE. In 1993, a final cover system was constructed over the site. Siting approval for the first Site 2 Expansion (initially identified as Site 3 at that time) was granted by the Zion City Council on April 17, 1995 which approved a new landfill unit east of Old Site 2 including a "piggyback" onto the eastern portion of Old Site 2. The IEPA approved development of the first Site 2 Expansion in Permit No. 1995-343-LF on March 21, 1997. Waste placement activities commenced at the first Site 2 Expansion in March 1998 following IEPA approval of the construction acceptance report for Cell 1 in Modification No. 2 of the permit on March 24, 1998. The Site 2 Expansion was originally permitted under 35 IAC, Part 812, Subparts A and C, and is now regulated under 35 IAC, Part 811 regulations, which meet or exceed Subtitle D Federal landfill regulations.

A second expansion, referred to as the Site 2 East Expansion, included a vertical and an approximate 26.5 acre horizontal expansion to the east of the previous Site 2 Expansion footprint. The initial phase of the Site 2 East vertical expansion was permitted on June 3, 2011, with the remainder of the expansion approved for development on June 13, 2014. The Site 2 East Expansion is regulated under Subtitle D landfill regulations. For purposes of this section, references to "Site 2" include both the Site 2 Expansion and Site 2 East Expansion. A groundwater model has been previously developed and permitted by the IEPA for Site 2.

This application proposes to expand horizontally to the north of the currently permitted Site 2 and vertically onto Site 2. As such, the currently permitted groundwater model for Site 2 was reviewed and has been incorporated into this GIA. The permitted groundwater model had an anticipated life that started in 1998 and only modeled the Shallow Drift Aquifer (uppermost aquifer). As part of the permitting of the groundwater model for Site 2, the Applicant made a demonstration to the IEPA that the intra-till sediments alongside Site 2 were discontinuous and that groundwater movement within these sediments is predominantly directed vertically downwards. Likewise, the hydrogeological investigation provided in Section 2.2 of this Application identified the intra-till sediments as being discontinuous in the Site 2 North Expansion area. Therefore, intra-till sediments were not included in the permitted groundwater model and were not included or modeled in this GIA.

The design and hydrogeologic setting of the proposed Site 2 North Expansion has been evaluated using the data generated during the recent and previous hydrogeologic investigations, the proposed landfill design, and a computer-generated model. The site geology and hydrogeology are documented in Section 2.2 of this application.

The proposed landfill expansion and currently permitted Site 2 have been designed with extensive environmental safeguards, including a composite liner system consisting of a 60 mil HDPE geomembrane liner and a 5 foot recompacted cohesive soil liner (1 x 10^{-7} cm/sec), a leachate collection and removal system, and a composite final cover. The design of the proposed landfill expansion is discussed in greater detail in Section 2.3 of this application. The site-specific data obtained from the hydrogeologic investigation and the proposed design were incorporated into the computer model. When site-specific data were not available, conservative estimates or assumptions (representing more stringent or safe environmental conditions) of model input parameters were used. The main conservative estimates or assumptions used in the model are as follows:

1. A constant leachate concentration was used throughout the 147-year modeling period. The concentration of each constituent in leachate can be assumed to be constant or a specific mass can be assumed to be present. Assuming a specific mass results in a decreasing source concentration over time, which would most accurately represent the fact that leachate concentrations in landfills with leachate collection and removal systems will gradually decrease over time. However, a constant concentration was assumed as it results in conservative model results.

2. The landfill will have an inward gradient throughout the 147-year GIA modeling period, with groundwater flowing into the landfill in the unlikely event that a puncture of the liner was to occur. Conservatively, the groundwater model assumed that the landfill will have an outward gradient with one (1) foot of leachate head acting on the liner. A one (1) foot leachate head was used in the calculation of the landfill vertical seepage rate, resulting in higher predicted concentrations at the base of the Wadsworth Formation prior to reaching the Shallow Drift Aquifer (uppermost aquifer) and the Zone of Attenuation (ZOA).

3. Poor liner contact was assumed in the calculation of the landfill vertical seepage rate, resulting in a higher seepage rate. A more conservative model is created by using a higher seepage rate through the liner. Section 2.5 of this application discusses the Construction Quality Assurance Program, which details specifications for liner installation. Good contact between the 60 mil HDPE liner and recompacted soil liner is expected at the site, making the poor liner contact assumption conservative.

4. The maximum reported leachate concentration for each constituent was used in the development of the model prediction table. An average of the reported leachate concentrations would result in a less conservative evaluation of the landfill expansion. Therefore, the maximum reported concentrations were used in the model.

5. Adsorption was conservatively not applied to the baseline groundwater model. Adsorption can play a significant role in retarding the migration of numerous constituents in groundwater. Not using adsorption in the model results in a higher predicted concentration at the base of the Wadsworth Formation prior to reaching the Shallow Drift Aquifer (uppermost aquifer) and the ZOA.

6. Degradation was conservatively not used in the baseline groundwater model. Degradation can play a significant role in reducing the migration of numerous constituents in groundwater. Not using degradation in the model results in a higher predicted concentration at the base of the Wadsworth Formation prior to reaching the Shallow Drift Aquifer (uppermost aquifer) and the ZOA.

Proposed Landfill Evaluation

The potential impact from the proposed landfill was evaluated by first developing a conceptual model of the site stratigraphy and hydrogeologic conditions, and then assigning physical characteristics and engineering properties to the principal material types to be included as model input parameters for the conceptual model. The model was then used to evaluate the site hydrogeologic conditions after development of the landfill and site closure. The model considered the properties and physical conditions most likely to represent expected site conditions. Conservative assumptions were used in the modeling. The results of the model were evaluated at the base of the Wadsworth Formation prior to reaching the Shallow Drift Aquifer (uppermost aquifer) and the ZOA.

Summary of GIA Findings

The findings of the model evaluations are as follows:

1. None of the constituents analyzed as part of the model will have an impact on the groundwater quality of the Shallow Drift Aquifer (Uppermost Aquifer) under the IEPA modeling criteria;

2. The calculated maximum predicted groundwater concentrations represent the results of the models when considering the proposed landfill design, site hydrogeologic conditions, and conservative modeling assumptions; and

3. The proposed landfill is located and designed so as to protect the public health, safety, and welfare.

Groundwater Impact Assessment Approach

This GIA was performed following the approach outlined below:

1. The conceptual site hydrogeologic model was developed and the pertinent landfill design details were identified;

2. Applicable Groundwater Quality Standard (AGQS) values were obtained from the March 8, 2022 Permit (No. 1995-343-LFM, Modification No. 155) for Site 2. The AGQS values have been used to evaluate the results of the GIA. Leachate concentrations were also obtained from Site 2. The AGQS values and leachate concentrations from Site 2 are representative of conditions that would be expected for the proposed expansion;

3. A model (POLLUTE), which adequately simulates the varying hydrogeologic conditions at the site for both advective and chemical transport, was selected;

4. The potential for advective chemical transport at the site was modeled. Site- and chemical-specific data were used whenever possible. When site- or chemical-specific data were not available, appropriate and conservative values from literature or values recommended by the IEPA were used (i.e., calculating outward leakage through the liner system, using the diffusion coefficient for chloride; using the model user's guide recommended porosity of 1 for the 60 mil HDPE geomembrane liner, and estimating effective porosities based on the site-specific porosities and effective porosity literature);

5. The groundwater model was used to generate groundwater concentration prediction factors at different times and distances;

6. Predicted concentration versus time and distance graphs were generated;

7. Sensitivity analyses were performed to evaluate changes to the contaminant transport results with variations in model input parameters; and

8. The model predicted groundwater concentrations were compared to the lowest permitted AGQS value for each constituent in order to evaluate the results of the GIA.

Leachate Quality Characterization and Groundwater Quality Standards

Leachate Quality Characterization

The leachate quality data (2nd quarter 2010 through 4th quarter 2021) established for Site 2 was used in the model predictions. A summary of the leachate data for Site 2 is provided in Appendix P and also included on the model prediction table in this GIA (See Table 2.7-2).

Groundwater Quality Standards

Applicable Groundwater Quality Standard (AGQS) values were obtained from the March 8, 2022 Permit (No. 1995-343-LFM, Modification No. 155) for Site 2. The AGQS values have been used to evaluate the results of the GIA. The AGQS values are provided in Appendix P and also included on the model prediction table in this GIA (Table 2.7-2). The lowest permitted AGQS for each constituent was used in the development of the model prediction table.

Design Considerations and Groundwater Impact Evaluation Model

Landfill design features must be considered prior to developing the conceptual model and establishing model input values. The landfill design features considered in the GIA include the final cover design, efficiency of the leachate collection system, and liner design.

As discussed earlier, the landfill will have an inward gradient throughout the 147-year GIA modeling period, with groundwater flowing into the landfill in the unlikely event that a puncture of the liner was to occur. Conservatively, the groundwater model assumed that the landfill will have an outward gradient with one (1) foot of leachate head acting on the liner. A one (1) foot leachate head was used in the calculation of the landfill vertical seepage rate, resulting in higher predicted concentrations.

After reviewing the hydrogeologic setting and proposed design of the Site 2 North Expansion and the current design of Site 2, it was determined that contaminant transport would be modeled vertically through the liner system to the base of the Wadsworth Formation prior to reaching the Shallow Drift Aquifer (uppermost aquifer). A one-dimensional POLLUTE model assessing the liner system and Wadsworth Formation as possible migration pathways was created for the proposed landfill expansion (Figure 2.7-1).

The model input is discussed in greater detail in the Model Input section later in this report. The Model Input section will also provide a more detailed discussion of how site-specific design was incorporated into the computer model selected for use for this GIA.

Sensitivity analyses were performed on the model to identify the effect of changes in the model input parameters on the model predicted representative maximum Groundwater Concentration Prediction Factor (GCPF). Further explanation and the results of these sensitivity analyses are located in the Sensitivity Analysis section of this report.

Conversion of Conceptual Model to Mathematical POLLUTE Model

The potential transport mechanisms that may occur at the subject site for the various layers include advection, mechanical dispersion, and diffusion. For intact material, these transport mechanisms are represented by the following one-dimensional flow equation (Rowe and Booker, 1990):

$$n\frac{\partial c}{\partial t} = nD\frac{\partial^2 c}{\partial z^2} - V_a\frac{\partial c}{\partial z} - \rho K\frac{\partial c}{\partial t} - \lambda c \qquad \text{(Equation 1)}$$

where:

c = concentration of contaminant at distance z at time t;

n = effective porosity of soil at distance z;

ρ = dry density of soil at distance z;

K = distribution coefficient at distance z;

D = Coefficient of hydrodynamic dispersion at distance z;

V_a = nv = Darcy Velocity;

v = groundwater (seepage) velocity at distance z; and

λ = constituent degradation constant.

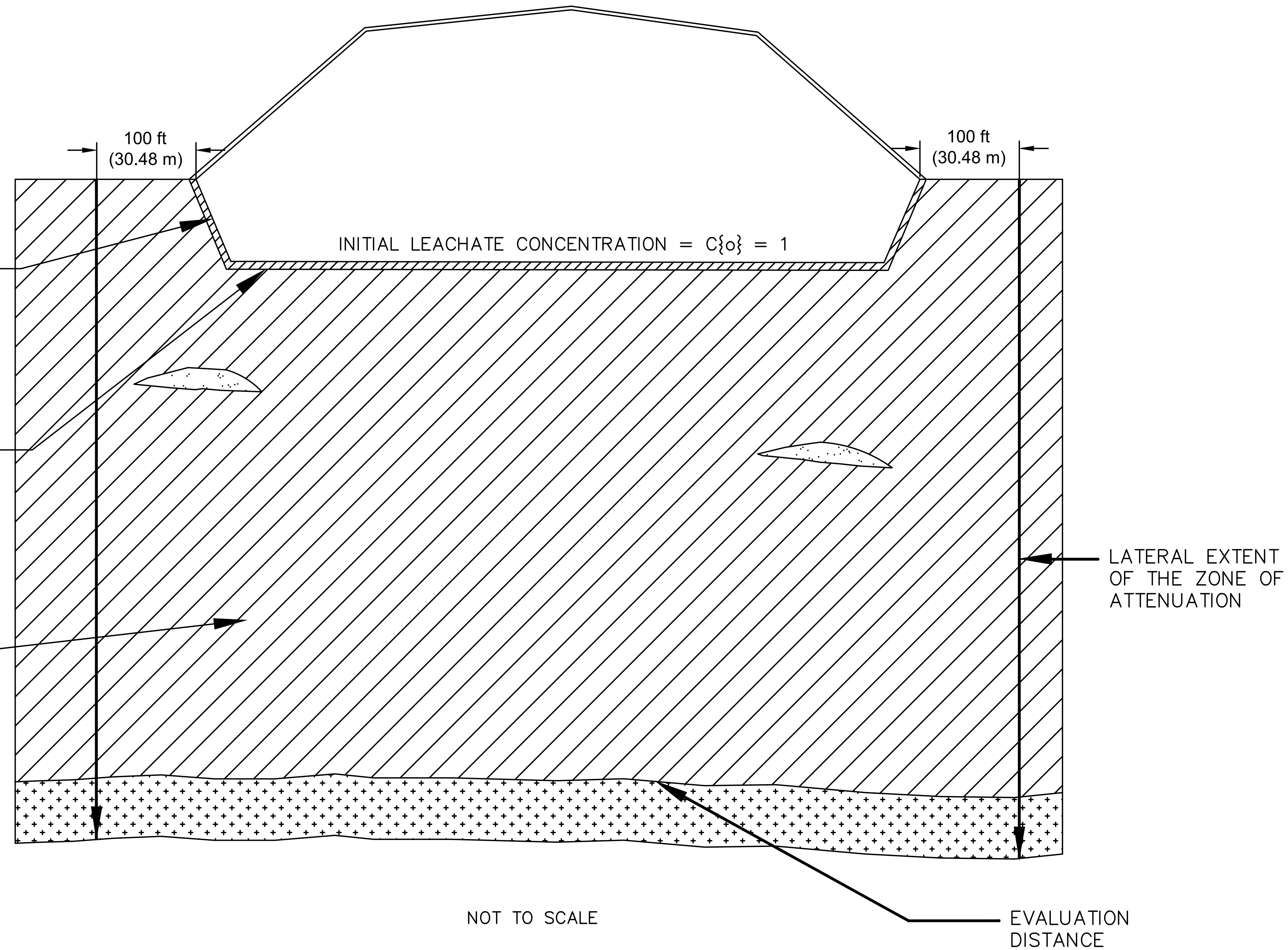
The solution of the Equation 1 yields both the temporal and the spatial distribution of predicted concentrations due to the leachate migration rate. The above equation incorporates the various transport mechanisms discussed with the conceptual model.

Rowe and Booker (1987) proposed a semi-analytical solution to the above-mentioned groundwater flow equation governing advective and chemical transport (Laplacean and Talbot inversion schemes). These mathematical procedures require the subsurface to be modeled in separate layers. Each layer can have different physical properties. The theory behind the above equation and its solution technique can be found in Rowe and Booker (1987), and Rowe and Booker (1985, 1986, 1987, 1988).

HDPE GEOMEMBRANE – LAYER 1
TOTAL THICKNESS – 0.0015 m
D_v – 3.0 x 10^{-5} m^2/yr
POROSITY – 1
DISTRIBUTION COEFF. – 0
DEGRADATION – 0
DENSITY – 940 kg/m^3
V_v – 3.08 x 10^{-4} m/yr

RECOMPACTED COHESIVE SOIL LINER – LAYER 2
TOTAL THICKNESS – 1.524 m
D_v – 0.019 m^2/yr
POROSITY – 0.25
DISTRIBUTION COEFF. – 0
DEGRADATION – 0
DENSITY – 1,896.5 kg/m^3
V_v – 3.08 x 10^{-4} m/yr

WADSWORTH FORMATION – LAYER 3
TOTAL THICKNESS – 10.27 m
D_v – 0.019 m^2/yr
POROSITY – 0.25
DISTRIBUTION COEFF. – 0
DEGRADATION – 0
DENSITY – 1,896.5 kg/m^3
V_v – 3.08 x 10^{-4} m/yr



LEGEND

- RECOMPACTED COHESIVE SOIL LINER
- WADSWORTH FORMATION
- INTRA – TILL SEDIMENTS
- SHALLOW DRIFT AQUIFER

REV. NO.	DATE	DESCRIPTION

GFL GREEN FOR LIFE environmental

APTIM Environmental & Infrastructure, LLC

APTIM

ZION LANDFILL SITE 2 NORTH EXPANSION LAKE COUNTY, ILLINOIS

FIGURE 2.7-1
GROUNDWATER IMPACT ASSESSMENT CONCEPTUAL MODEL

DRAWN BY:	NV	APPROVED BY:	DJD	PROJ. NO.:	631020105	DATE:	MAY 2022

Transport phenomena in the subsurface model layers is simulated using the groundwater transport model POLLUTE (Rowe et. al., 1994). POLLUTE was developed based on the semi-analytical solution to Equation 1. This program assumes that transport phenomena is governed by Equation 1.

The data input for POLLUTE is set up in such a way that it acquires all the input parameters, performs calculations for individual transport processes, and then uses the semi-analytical solution for the above-mentioned transport equation to yield predicted concentrations at specified times and distances.

The conceptual model indicates that the HDPE, recompacted soil liner, and Wadsworth Formation are relatively uniform. Due to the relative uniform variables, a one-dimensional model such as POLLUTE can accurately predict potential transport. The use of representative site-specific parameters and incorporating landfill design and post-development conditions in modeling more closely simulates actual conditions in the field with respect to the groundwater flow. Therefore, a formal groundwater flow calibration process is not required. Additional discussion about the model suitability can be found in the Model Reliability section.

Calculating Predicted Groundwater Concentrations

An initial leachate concentration value of one (1) was used in the model. This value is not meant to represent a specific concentration for a specific constituent. The value represents a unit concentration of any constituent in the leachate. The results from the model can be used to predict the concentration in the groundwater for any leachate constituent by multiplying the model result for any given distance and time by the established initial leachate concentration. This concept is expressed as the following formula:

$$PGC_{tx} = GCPF_{tx} * C_o \qquad \text{(Equation 2)}$$

where:

PGC_{tx} = Predicted Groundwater Concentration at t years and x meters from the edge of waste;

$GCPF_{tx}$ = Groundwater Concentration Prediction Factor at t years and x meters. The model result, expressed as a fraction, is used to predict the concentration of any particular constituent in the groundwater; and

C_o = Established Leachate concentration of the constituent of concern.

Interpretation of POLLUTE Model Results

In order to evaluate the design and hydrogeologic setting of the Site 2 North Expansion and the Site 2, the leachate concentrations and the appropriate permitted minimum AGQS values were used in conjunction with the groundwater concentration prediction factor obtained from the POLLUTE model. A discussion of the results of the model is provided later in this GIA.

Model Input

The following information documents the assumptions and values used for the model. The model represents the anticipated site conditions for the design and hydrogeologic setting of the proposed Site 2 North Expansion and Site 2. The assumptions and values are based on the actual design and CQA plan proposed in this application and the information obtained from the hydrogeologic investigation (Section 2.2). When site-specific values were not available, appropriate and conservative values from literature or values recommended by the IEPA were used.

Model Input

POLLUTE requires values for the input parameters identified in Table 2.7-1. The sources of the assigned parameter values for this GIA are described as follows. To the extent possible, site- or chemical-specific values were used. As previously mentioned, when site- or chemical-specific parameters were not available, appropriate values were obtained from published literature or by values recommended by the IEPA. In general, the input parameter values assigned for use in this GIA were intentionally biased when site-specific values were not available, to result in a higher predicted groundwater concentration at the evaluation distance to conform to IEPA conservative approaches. An example of a "conservative" value is using an adsorption coefficient, Kd, equal to zero for constituents that would readily be adsorbed to the liner material.

All model input must have consistent units. Each of the model input parameters are discussed briefly in the following paragraphs. Documentation for model input parameters is included within Appendix P.

Model Length

As discussed earlier, three (3) layers will be modeled at the site: a 60 mil HDPE geomembrane liner (0.0015 m), a five (5) foot (1.524 m) recompacted cohesive soil liner (1.0 x 10^{-7} cm/sec), and approximately 33.7 feet (10.27 m) of the Wadsworth Formation (extending from the base of liner system to the top of the Shallow Drift Aquifer (uppermost aquifer). A figure showing the thickness of the Wadsworth Formation below the mass excavation base grades (not including the sidewalls) and the top of the Shallow Drift Aquifer (uppermost aquifer) is provided in Appendix P.

Because the model predicts contaminant transport out of the liner system and vertically to the base of the Wadsworth Formation, the model length is the sum of the liner thickness and the distance to the base of the Wadsworth Formation. The HDPE is 0.0015 m thick and the recompacted clay liner is 1.524 m thick, resulting in a total liner system thickness of 1.5255 m. The total model length is 11.7955 m. Although the model has been set up assuming an infinite bottom boundary, the model was evaluated at the base of the Wadsworth Formation (11.7955 m).

With the Site 2 being incorporated into this GIA, it was important to review the Wadsworth Formation thickness used in the currently permitted groundwater model for Site 2 and make sure the thickness is consistent with the thickness used in this GIA. The thickness used in the currently permitted groundwater model for Site 2 was 32.5 feet (9.91 m) with minimum and maximum thicknesses of 26.4 feet (8.05 m) and 89.0 feet (27.13 m), respectively.

TABLE 2.7-1
BASELINE MODEL INPUT PARAMETER VALUES
SITE 2 NORTH EXPANSION

Parameter	Value	Notes	Data
Model Length (L)(m)	11.7955	Total Length of Model including the HDPE, Recompacted Cohesive Soil Liner, and Thickness of Wadsworth Formation Below the Recompacted Cohesive Soil Liner	1,2
Initial Leachate Concentration (Co)	1	Unit Leachate Concentration	2
Number of Layers	3	Total Number of Modeled Layers	1,2
Modeling Period (years)	147	47 Years of Active Life Plus 100 Years Past Closure	2
TALBOT PARAMETERS			
TAU	7	Talbot Parameters for the Numerical Inversion of the Laplace Transform	2
Sigma	0		2
RNU	2		2
N	20		2
LAYER 1 - 60 mil HDPE Geomembrane Liner			
Sublayers	1	Model Parameter	2
Thickness (b) (m)	0.0015	Design Specification	1,2
Porosity (n)	1	Assume All Flow Through Pinholes	1,2
Adsorption Coefficient (K) (Kg/m^3)	0.0	No Adsorption Modeled	2,3
Degradation (λ)	0.0	No Degradation Modeled	2,3
Density (ρ) (Kg/m^3)	940	HDPE Manufacturer's Specification	1,2
Vertical Darcy Velocity (m/yr)	3.08×10^{-4}	Assuming Outward Gradient	1,2
Coeff. of Hydrodynamic Dispersion (D) (m^2/yr)	3.0×10^{-5}	D = D* (Due to the low seepage rate, movement will be dominated by diffusion)	2

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TABLE 2.7-1 (CONTINUED)
BASELINE MODEL INPUT PARAMETER VALUES
SITE 2 NORTH EXPANSION

Parameter	Value	Notes	Data
LAYER 2 - Recompacted Cohesive Soil Liner			
Sublayers	5	Model Parameter	2
Thickness (b) (m)	1.524	Design Specification	1,2
Estimated Effective Porosity (n_e)	0.25	Derived from the average total porosity from laboratory results for the Wadsworth Formation based on Sara (1994). (Used in Liner Construction)	1,2
Adsorption Coefficient (K) (Kg/m^3)	0.0	No Adsorption Modeled	2,3
Degradation (λ)	0.0	No Degradation Modeled	2,3
Density (ρ) (Kg/m^3)	1,896.5	Value Obtained from Laboratory Results for the Wadsworth Formation	1,2
Vertical Darcy Velocity (m/yr)	3.08×10^{-4}	Assuming Outward Gradient	1,2
Coeff. of Hydrodynamic Dispersion (D) (m^2/yr)	0.019	D = D* (Due to the low seepage rate, movement will be dominated by diffusion)	1,2
LAYER 3 - Wadsworth Formation			
Sublayers	34	Model Parameter	2
Thickness (b) (m)	10.27	Model Specification	1,2
Estimated Effective Porosity (n_e)	0.25	Derived from the average total porosity from laboratory results for the Wadsworth Formation based on Sara (1994).	1,2
Adsorption Coefficient (K) (Kg/m^3)	0.0	No Adsorption Modeled	2,3
Degradation (λ)	0.0	No Degradation Modeled	2,3
Density (ρ) (Kg/m^3)	1,896.5	Value Obtained from Laboratory Results for the Wadsworth Formation	1,2
Vertical Darcy Velocity (m/yr)	3.08×10^{-4}	Assuming Outward Gradient	1,2
Coeff. of Hydrodynamic Dispersion (D) (m^2/yr)	0.019	D = D* (Due to the low seepage rate, movement will be dominated by diffusion)	1,2

Explanation of Data:
1. Value is based on actual anticipated site conditions.
2. Value is required model input parameter.
3. Value is conservative value which will result in higher predicted concentrations than the actual anticipated site conditions.

The figure provided in Appendix P for the thickness or the Wadsworth Formation, below the mass excavation base grades and the top of the Shallow Drift Aquifer (uppermost aquifer), includes the portion of the Site 2 North Expansion that expands vertically onto Site 2. The average thickness of the Wadsworth Formation was 33.7 feet (10.27 m) with minimum and maximum thicknesses of 19.7 feet (6.00 m) and 43.6 feet (13.29 m), respectively. With the average thicknesses of Wadsworth Formation being relatively the same in this GIA and the currently permitted groundwater model for Site 2, it was determined that using the thickness of the Wadsworth Formation documented in Appendix P would be appropriate.

Initial Leachate Concentration

The initial leachate concentration input used was one (1). This value is unitless because it represents unit leachate concentration of any given constituent. Therefore, the model results represent a fraction of the initial leachate concentration for any particular constituent.

Number of Layers

As discussed above, three layers will be modeled at the site: a 60 mil HDPE geomembrane liner (0.0015 m), a five (5) foot (1.524 m) recompacted cohesive soil liner (1.0 x 10^{-7} cm/sec), and approximately 33.7 feet (10.27 m) of the Wadsworth Formation (Figure 2.7-1). POLLUTE also allows a layer to be subdivided so that the predicted concentration distribution within a layer can be evaluated.

The HDPE geomembrane liner, recompacted cohesive soil liner, and Wadsworth Formation were divided into 1, 5, and 34 sublayers, respectively.

Modeling Period

The modeling period is the expected life of the landfill plus 100 years after closure. The expected life of the landfill has been estimated to be approximately 47 years (1998 to 2044), resulting in a modeling period of 147 years. The expected life of approximately 47 years is based on the time when waste placement activities commenced at the first Site 2 Expansion in March 1998 and an ending operating life of 2044.

Talbot Parameters

POLLUTE uses a Laplace transform to find the solution to the advection-dispersion equation. The numerical inversion of the Laplace transform depends on the Talbot parameters. The model provides default values for the Talbot parameters or they can be selected by the user. When the user selects Talbot parameters, integration is increased, and computation times are also increased.

The use of default and select Talbot parameters resulted in the same model-predicted representative maximum GCPF in this groundwater model. Therefore, the use of select Talbot parameters provided no additional accuracy in the model-predicted representative maximum GCPF and the default Talbot parameters were used in this groundwater model.

Boundary Conditions

POLLUTE requires the specification of an upper and lower boundary condition. The top boundary condition typically represents the landfill as a potential source. When modeling the landfill as a surface boundary, the concentration of each constituent in leachate can be assumed to be constant or a specific mass can be assumed to be present. Assuming a specific mass results in a decreasing source concentration over time, which would most accurately represent the fact that leachate concentrations in landfills with leachate collection and removal systems will gradually decrease over time. However, a constant concentration was assumed as it results in conservative model results.

The lower boundary condition was specified as an infinite bottom layer. This boundary condition assumes that horizontal flow can continue to any distance, which allows for realistic analysis of conditions at the base of the Wadsworth Formation.

Advective (Darcy) Velocity

POLLUTE requires the input of a Darcy velocity, which is calculated across the complete length of the groundwater model. Table 2.7-1 lists the Darcy velocity value for the model. The Darcy velocity was set equal to the calculated outward seepage rate of 3.08×10^{-4} m/yr. The seepage rate was calculated using an equation derived by Giroud and Bonaparte (1989). This equation and value (3.08×10^{-4} m/yr) have been accepted by the IEPA. Documentation of the calculated outward seepage rate is provided in Appendix P.

Hydrodynamic Dispersion Coefficient

POLLUTE requires the input of a hydrodynamic dispersion coefficient for each layer. The hydrodynamic dispersion coefficient is calculated by the following equation:

$$D = D^* + av \quad \text{(Equation 3)}$$

where,

D = the hydrodynamic dispersion coefficient (m^2/yr);

a = the dispersivity (m);

v = the groundwater seepage velocity (m/yr); and

D^* = the effective diffusion coefficient (m^2/yr).

Table 2.7-1 lists the model input dispersion coefficient values. The dominant transport mechanism for the HDPE and recompacted cohesive soil liner, and Wadsworth Formation is diffusion due to the low outward seepage rate (3.08×10^{-4} m/yr). Dispersivity is negligible due to the low outward seepage rate, therefore the hydrodynamic dispersion coefficient is equal to effective diffusion coefficient. The diffusion rate in the clay liner and Wadsworth Formation will be greater than the conservative seepage rate out of the landfill. An effective diffusion coefficient of 3.0×10^{-5} m^2/y has historically been recommended by the IEPA for the 60 mil HDPE geomembrane liner. This value was used in the model described within this

GIA and is the default value for POLLUTE. This value has been the baseline value for all of the groundwater models APTIM has previously submitted to the IEPA for approval and has been permitted at numerous Illinois landfills. An input of 0.019 m^2/yr (Rowe, Quigley, Brachman, and Booker, 2004) was used to represent the effective diffusion coefficient in the five (5) foot recompacted cohesive soil liner and Wadsworth Formation. Documentation of the Hydrodynamic Dispersion Coefficients is provided in Appendix P.

Effective Porosity and Dry Density Input

Table 2.7-1 lists the porosity and effective porosity and dry density values for the model layers. The porosity of the 60 mil HDPE geomembrane liner was assumed to be one (1) with all flow occurring through the pinholes in the liner. The density of the HDPE liner was obtained from manufacturer's specifications.

The effective porosity value for the compacted cohesive soil liner and Wadsworth Formation was derived from laboratory data for the Wadsworth Formation, which has been provided in Section 2.2 of this Application. The laboratory measured porosity values of the Wadsworth Formation were converted to effective porosities based on empirical data provided by Sara (1994) as shown in Appendix P. As provided in Appendix P, the effective porosity of the Wadsworth Formation ranges from 0.21 to 0.32, with an average of 0.25. The percentage difference between the total and effective porosity for a clay was conservatively used to calculate the effective porosities. The clay from the Wadsworth Formation will be used for construction of the compacted cohesive soil liner.

With Site 2 being incorporated into this GIA, it was important to review the porosity value for the compacted cohesive soil liner and Wadsworth Formation used in the currently permitted groundwater model for Site 2 and make sure the porosity is similar to the effective porosity used in this GIA. The porosity used in the currently permitted groundwater model for Site 2 was 0.29 with minimum and maximum porosity of 0.20 and 0.38, respectively.

With the average effective porosity value for the compacted cohesive soil liner and Wadsworth Formation being more conservative in this GIA, than the currently permitted average porosity for the groundwater model for Site 2, it was determined that using the effective porosity value documented in this GIA in Appendix P would be more appropriate.

Adsorption Coefficient

The adsorption coefficient (K_d) is used to simulate retardation of constituents in the subsurface. The adsorption coefficient is specific to each particular compound and the geologic material.

Although adsorption can play a significant role in retarding the migration of numerous constituents in groundwater, it is conservatively assumed that the adsorption coefficients are zero.

Degradation

Degradation is used to simulate degradation of constituents in the subsurface. Degradation is specific to each particular compound.

Although degradation can play a significant role in reducing the migration of numerous constituents in groundwater, it is conservatively assumed that degradation is not present in the baseline groundwater model.

Model Evaluation Distance

The model evaluation distance is not a model input parameter. However, this distance is needed in order to evaluate the results of the GIA since the model only provides results for specified distances. The model was evaluated at the base of the Wadsworth Formation, a distance of 11.7955 m.

It is important to note that this GIA was evaluated at the base of the Wadsworth Formation (prior to the ZOA), resulting in a more conservative model by not including horizontal transport in the Shallow Drift Aquifer (uppermost aquifer) out to the ZOA, 100 feet (30.48 m) from the waste boundary. The Wadsworth Formation could also have been modeled horizontally out to the ZOA from the base of the landfill, but the distance would have been more than three (3) times longer than the baseline model distance (thickness) for the Wadsworth Formation (33.7 feet or 10.27 m).

Model Results

The GIA was completed to evaluate the anticipated site conditions based upon the hydrogeology and the proposed designs, the CQA plan, the operations, and the post-closure care of the facility. The results of the GIA, as discussed below, demonstrate that the landfill will not have an adverse impact on groundwater quality at the ZOA for 100 years after closure of the landfill.

Anticipated Site Conditions Phase

The model output for the Site 2 North Expansion is included in Appendix P. The model-predicted representative maximum GCPF for the entire 147-year simulation period at the edge of the zone of attenuation is 5.81×10^{-7}.

As discussed earlier, the model predicted groundwater concentration for each of the constituents can simply be obtained by multiplying the maximum GCPF and the initial leachate concentration corresponding to the respective constituent.

The leachate quality data established for Site 2 was used in conjunction with the groundwater concentration prediction factors to compare the predicted groundwater concentrations at the base of the Wadsworth Formation to the AGQS values in Table 2.7-2. As indicated in Table 2.7-2, the model predicted groundwater concentrations at the base of the Wadsworth Formation (prior to the ZOA) do not exceed the AGQS for each respective constituent at the proposed Site 2 North Expansion and Site 2.

Thus, the proposed expansion design and site hydrogeologic characteristics are such that there will be no adverse impact on groundwater quality in the Shallow Drift Aquifer (Uppermost Aquifer). Expected concentrations in the groundwater will actually be lower than those predicted in the GIA because of the overly conservative nature of the model.

Concentration versus time and depth plots for the baseline model are presented in Appendix P.

Table 2.7-2

POLLUTE Model Groundwater Concentration Prediction Table - Shallow Drift Aquifer

Constituent	Units	AGQS	Maximum Leachate Concentration	Model Predicted Groundwater Concentration at the Base of the Wadsworth Formation	Does the Model Predict Exceedance of the AGQS Values?
Arsenic (total)	ug/L	6.2	371	2.16E-04	NO
Barium (total)	ug/L	248.0	252	1.46E-04	NO
Cadmium (total)	ug/L	10.0	< 50	2.91E-05	NO
Iron (total)	ug/L	992.0	149,000	8.66E-02	NO
Ammonia (as Nitrogen)	mg/L	0.6	1,680	9.76E-04	NO
1,1,1,2-Tetrachloroethane	ug/L	5.0	< 50	2.91E-05	NO
1,1,1-Trichloroethane	ug/L	5.0	< 50	2.91E-05	NO
1,1,2,2-Tetrachloroethane	ug/L	10.0	< 100	5.81E-05	NO
1,1,2-Trichloroethane	ug/L	5.0	< 50	2.91E-05	NO
1,1-Dichloroethane	ug/L	5.0	< 50	2.91E-05	NO
1,1-Dichloroethene	ug/L	5.0	< 50	2.91E-05	NO
1,1-Dichloropropene	ug/L	5.0	< 80	4.65E-05	NO
1,2,3-Trichlorobenzene	ug/L	5.0	< 50	2.91E-05	NO
1,2,3-Trichloropropane	ug/L	15.0	< 150	8.72E-05	NO
1,2,4-Trichlorobenzene	ug/L	10.0	< 217	1.26E-04	NO
1,2,4-Trimethylbenzene	ug/L	5.0	< 50	2.91E-05	NO
1,2-Dibromo-3-chloropropane (DBCP)	ug/L	25.0	< 130	7.55E-05	NO
1,2-Dichlorobenzene	ug/L	10.0	< 100	5.81E-05	NO
1,2-Dichloroethane	ug/L	5.0	22.7	1.32E-05	NO
1,2-Dichloropropane	ug/L	5.0	< 50	2.91E-05	NO
1,3,5-Trimethylbenzene	ug/L	5.0	< 50	2.91E-05	NO
1,3-Dichlorobenzene	ug/L	5.0	< 50	2.91E-05	NO
1,3-Dichloropropane	ug/L	5.0	< 50	2.91E-05	NO
1,3-Dichloropropene	ug/L	5.0	< 267	1.55E-04	NO
1,4-Dichlorobenzene	ug/L	5.0	< 80	4.65E-05	NO
1-Propanol	ug/L	1,000.0	< 5,320	3.09E-03	NO
2,2-Dichloropropane	ug/L	15.0	< 150	8.72E-05	NO
2,4,5-tp (Silvex)	ug/L	2.0	< 20	1.16E-05	NO
2,4-Dichlorophenoxyacetic Acid (2,4-D)	ug/L	10.0	< 25	1.44E-05	NO
2-Chloroethyl Vinyl Ether	ug/L	8.8	< 10,000	5.81E-03	NO
2-Hexanone	ug/L	50.0	< 500	2.91E-04	NO
2-Propanol (Isopropyl Alcohol)	ug/L	1,000.0	28,400	1.65E-02	NO
4-Methyl-2-pentanone [MIBK]	ug/L	50.0	181	1.05E-04	NO
Acetone	ug/L	100.0	23,100	1.34E-02	NO

Table 2.7-2 (continued)

POLLUTE Model Groundwater Concentration Prediction Table - Shallow Drift Aquifer

Constituent	Units	AGQS	Maximum Leachate Concentration	Model Predicted Groundwater Concentration at the Base of the Wadsworth Formation	Does the Model Predict Exceedance of the AGQS Values?
Alachlor	ug/L	2.0	< 20	1.16E-05	NO
Aldicarb	ug/L	3.0	< 120	6.97E-05	NO
Aldrin	ug/L	1.0	< 10	5.81E-06	NO
Aluminum (total)	ug/L	173,078.4	7,760	4.51E-03	NO
Antimony (total)	ug/L	6.0	< 200	1.16E-04	NO
Atrazine	ug/L	3.0	< 30	1.74E-05	NO
Benzene	ug/L	5.0	< 50	2.91E-05	NO
Benzo (a) Pyrene	ug/L	0.2	< 502	2.92E-04	NO
Beryllium (total)	ug/L	4.0	< 40	2.32E-05	NO
Beta-BHC	ug/L	0.05	< 1	5.81E-07	NO
Bis (2-Chloro-1-Methylethyl) Ether	ug/L	10.0	< 407	2.36E-04	NO
Bis (2-Ethylhexyl) Phthalate	ug/L	6.0	519	3.02E-04	NO
Boron (total)	ug/L	574.0	14,400	8.37E-03	NO
Bromobenzene	ug/L	5.0	< 50	2.91E-05	NO
Bromochloromethane	ug/L	1.0	< 50	2.91E-05	NO
Bromodichloromethane	ug/L	5.0	< 50	2.91E-05	NO
Bromoform	ug/L	10.0	< 132	7.67E-05	NO
Bromomethane (Methyl Bromide)	ug/L	10.0	< 100	5.81E-05	NO
Calcium	mg/L	300.0	913	5.30E-04	NO
Carbofuran	ug/L	40.0	< 400	2.32E-04	NO
Carbon disulfide	ug/L	5.0	< 100	5.81E-05	NO
Carbon tetrachloride	ug/L	5.0	< 50	2.91E-05	NO
Chlorodane	ug/L	2.0	< 17.3	1.01E-05	NO
Chloride (total)	mg/L	12.0	3,250	1.89E-03	NO
Chlorobenzene	ug/L	5.0	< 50	2.91E-05	NO
Chloroethane	ug/L	10.0	< 100	5.81E-05	NO
Chloroform	ug/L	5.0	< 50	2.91E-05	NO
Chloromethane	ug/L	10.0	< 100	5.81E-05	NO
Chromium (total)	ug/L	10.0	540	3.14E-04	NO
Cis-1,2-Dichloroethene	ug/L	5.0	33.4	1.94E-05	NO
Cobalt (total)	ug/L	100.0	< 200	1.16E-04	NO
Copper (total)	ug/L	40.0	54	3.14E-05	NO
Cyanide (Total)	mg/L	10.0	< 0.12	6.97E-08	NO
DDT	ug/L	10.0	< 10	5.81E-06	NO

Table 2.7-2 (continued)

POLLUTE Model Groundwater Concentration Prediction Table - Shallow Drift Aquifer

Constituent	Units	AGQS	Maximum Leachate Concentration		Model Predicted Groundwater Concentration at the Base of the Wadsworth Formation	Does the Model Predict Exceedance of the AGQS Values?
Di-n-butyl phthalate	ug/L	10.0	<	2,230	1.30E-03	NO
Dibromochloromethane	ug/L	5.0	<	86.7	5.04E-05	NO
Dibromomethane (Methylene Bromide)	ug/L	10.0	<	100	5.81E-05	NO
Dichlorodifluoromethane	ug/L	5.0	<	50	2.91E-05	NO
Dichloromethane (Methylene Chloride)	ug/L	5.0		80.20	4.66E-05	NO
Dieldrin	ug/L	10.0	<	10	5.81E-06	NO
Diethyl phthalate	ug/L	10.0	<	902	5.24E-04	NO
Dimethyl phthalate	ug/L	10.0	<	515	2.99E-04	NO
Endrin	ug/L	0.2	<	2	1.16E-06	NO
Ethylbenzene	ug/L	5.0	<	50	2.91E-05	NO
Ethylene Dibromide (EDB)	ug/L	0.05	<	30	1.74E-05	NO
Fluoride	mg/L	1.86		41	2.38E-05	NO
Heptachlor epoxide	ug/L	0.2	<	10	5.81E-06	NO
Heptachlor	ug/L	0.4	<	1	5.81E-07	NO
Hexachlorobutadiene	ug/L	10.0	<	220	1.28E-04	NO
Hexachlorocyclopentadiene	ug/L	50.0	<	1,690	9.82E-04	NO
Iodomethane	ug/L	10.0	<	150	8.72E-05	NO
Isopropylbenzene	ug/L	5.0	<	50	2.91E-05	NO
Lead (total)	ug/L	20.0		63.90	3.71E-05	NO
Lindane	ug/L	0.2	<	1	5.81E-07	NO
Magnesium (total)	mg/L	3.5		439	2.55E-04	NO
Manganese (total)	ug/L	63.0		6,080	3.53E-03	NO
Mercury (total)	ug/L	0.2	<	4.20	2.44E-06	NO
Methoxychlor	ug/L	40.0	<	10	5.81E-06	NO
Methyl Ethyl Ketone (2-Butanone)	ug/L	10.0		30,900	1.80E-02	NO
Naphthalene	ug/L	5.0	<	632	3.67E-04	NO
Nickel (total)	ug/L	119.0		350	2.03E-04	NO
Nitrate-Nitrogen (total)	mg/L	0.5		10.8	6.27E-06	NO
Oil (Hexane-Soluble)	mg/L	14.0		185.0	1.07E-04	NO
Parathion	ug/L	10.0	<	2,850	1.66E-03	NO
Pentachlorophenol	ug/L	1.0	<	1,950	1.13E-03	NO
Phenols (total recoverable)	ug/L	63.9		4,600	2.67E-03	NO
Phosphorus	ug/L	1,590.0		43,300	2.52E-02	NO
Polychlorinated Biphenyls	ug/L	0.5	<	5	2.91E-06	NO

Table 2.7-2 (continued)

POLLUTE Model Groundwater Concentration Prediction Table - Shallow Drift Aquifer

Constituent	Units	AGQS	Maximum Leachate Concentration	Model Predicted Groundwater Concentration at the Base of the Wadsworth Formation	Does the Model Predict Exceedance of the AGQS Values?
Potassium	mg/L	6.56	936	5.44E-04	NO
Selenium	ug/L	5.0	< 200	1.16E-04	NO
Silver (total)	ug/L	50.0	< 100	5.81E-05	NO
Sodium	mg/L	110.0	2,500	1.45E-03	NO
Styrene	ug/L	10.0	< 100	5.81E-05	NO
Sulfate (total)	mg/L	9.7	346	2.01E-04	NO
tert-Butylbenzene	ug/L	5.0	< 50	2.91E-05	NO
Tetrachloroethene	ug/L	5.0	< 50	2.91E-05	NO
Tetrahydrofuran	ug/L	20.0	2,050	1.19E-03	NO
Thallium	ug/L	9.2	< 400	2.32E-04	NO
Toluene	ug/L	5.0	271	1.57E-04	NO
Toxaphene	ug/L	3.0	< 50	2.91E-05	NO
trans-1,2-Dichloroethene	ug/L	1.0	< 50	2.91E-05	NO
trans-1,3-Dichloropropene	ug/L	10.0	< 146	8.48E-05	NO
Trichloroethene	ug/L	5.0	< 50	2.91E-05	NO
Trichlorofluoromethane	ug/L	5.0	< 50	2.91E-05	NO
Vinyl acetate	ug/L	10.0	< 200	1.16E-04	NO
Vinyl chloride	ug/L	2.0	< 20	1.16E-05	NO
Xylene	ug/L	10.0	57.4	3.33E-05	NO
Zinc (total)	ug/L	32.0	1,710	9.94E-04	NO
m+p-Xylene	ug/L	10.0	49	2.85E-05	NO
n-Butylbenzene	ug/L	5.0	< 50	2.91E-05	NO
n-Propylbenzene	ug/L	5.0	< 50	2.91E-05	NO
o-Chlorotoluene	ug/L	1.0	< 50	2.91E-05	NO
o-Xylene	ug/L	10.0	20.7	1.20E-05	NO
p-Chlorotoluene	ug/L	5.0	< 50	2.91E-05	NO
p-Cresol	ug/L	10.0	831	4.83E-04	NO
p-Isopropyltoluene	ug/L	5.0	< 50	2.91E-05	NO
sec-Butylbenzene	ug/L	5.0	51	2.98E-05	NO

Notes:
1) Leachate data was collected from 5-17-2010 to 10-25-2021 and the values reported include the highest non detect values when the constituent was not detected in the leachate.
2) AGQS values were obtained from the IEPA Permit No. 1995-343-LFM Modification No. 155.
3) The AGQS for each constituent is the lowest available value from the total, dissolved, or intrawell values.
4) ug/L = micrograms per Liter (parts per billion).
5) mg/L = milligrams per Liter (parts per million).

Sensitivity Analysis

As discussed in the Model Input section, many of the model input parameters were site-specific. The baseline model used representative values from these site-specific parameters. As discussed in the Model Results section and shown in Tables 2.7-2, model predicted GCPF values and thus groundwater concentrations were noted at the base of the Wadsworth Formation prior to the ZOA. Accordingly, the sensitivity analysis focused on the effect of changes in baseline model input parameters on the model predicted representative maximum GCPF at the base of the Wadsworth Formation. The sensitivity analyses are provided in Appendix P. Justification for the variation used in the sensitivity analyses is discussed as follows. A table at the front of the sensitivity analyses summarizes the sensitivity analyses performed on the baseline POLLUTE model.

Coefficient of Hydrodynamic Dispersion

The coefficient of hydrodynamic dispersion of the HDPE (3.0 x 10^{-5} m^2/yr) was increased and decreased by 25%. This value has been derived from laboratory testing. Therefore, a 25% change is considered conservative and will result in a satisfactory sensitivity evaluation of this parameter. In the five (5) foot recompacted clay liner and the Wadsworth Formation, the baseline value used for the coefficient of hydrodynamic dispersion was 0.019 m^2/yr, which was obtained from published literature (Rowe, Quigley, Brachman, and Booker, 2004) as provided in Appendix P. This value is conservative because it is the diffusion coefficient for chloride through clay, which is considered to have a high ability to diffuse relative to other leachate constituents and is not easily retarded by clay. As the baseline value is set at the maximum of the diffusive range, it was determined that a 25% change would be considered conservative and will result in a satisfactory sensitivity evaluation of this parameter.

Sensitivity analysis of the above mentioned parameter resulted in satisfactory results for all of the sensitivity runs (Appendix P). Changes in the coefficient of hydrodynamic dispersion had little effect on the resulting prediction factors. As the coefficient of hydrodynamic dispersion increased in the HDPE, compacted clay liner, and in-situ clay so did the predicted concentration. As the coefficient of hydrodynamic dispersion decreased in the HDPE, compacted clay liner, and in-situ clay so did the predicted concentration.

Effective Porosity

The effective porosity of the five (5) foot recompacted clay liner and the Wadsworth Formation that was used for the baseline model (0.25) is the site-specific average of the effective porosity from laboratory data for the Wadsworth Formation, which was provided in the Hydrogeologic Investigation (Section 2.2). The clay from the Wadsworth Formation will be used for construction of the recompacted cohesive soil liner. Due to the availability of site-specific data, it was possible to obtain a range of values (0.21 to 0.32) from the samples tested. As such, sensitivity analyses were run using both the maximum and minimum effective porosity expressed in the laboratory results for the Wadsworth Formation. As a result, it was determined that a maximum and minimum change of effective porosities for the five (5) foot recompacted clay liner and the Wadsworth Formation would result in a satisfactory sensitivity evaluation of this parameter.

It should be noted that the porosity of the HDPE was not changed because it is at the maximum recommended value suggested by the POLLUTE User's Guide (Rowe, Booker, and Fraser, 1994). This value is documented in Appendix P.

Sensitivity analysis of the above mentioned parameter resulted in satisfactory results for all of the sensitivity runs (Appendix P). As expected, the effective porosity changes in the compacted clay liner and in-situ clay showed little effect on the model. Transport through the liner and in-situ clay is diffusion based and the effective porosity has little effect on the results.

Layer Thickness

The average thickness of the Wadsworth Formation was calculated to be approximately 33.7 feet (10.27 m) between the base of the liner system and the base of the Wadsworth Formation prior to the Shallow Drift Aquifer (uppermost aquifer). The minimum and maximum thickness of the Wadsworth Formation between the base of the liner system and the base of the Wadsworth Formation will be 19.7 feet (6.00 m) and 43.6 feet (13.29 m), respectively. It was determined that sensitivity runs that used the minimum and maximum thickness of the Wadsworth Formation would result in a satisfactory sensitivity evaluation of this parameter.

The thickness of the five (5) foot recompacted clay liner and HDPE will not vary. These layers will be installed / constructed and will be inspected in accordance with the CQA plan.

Sensitivity analysis of the above mentioned parameter resulted in satisfactory results for all of the sensitivity runs (Appendix P). As the thickness of the in-situ clay increased the predicted concentration decreased and as the thickness of the in-situ clay decreased the predicted concentration increased.

However, it is important to note that at the minimum thickness, the predicted groundwater concentration for several constituents was greater than their respective AGQS (ammonia as nitrogen, 2-chloroethyl vinyl ether, benzo (a) pyrene, methyl ethyl ketone (2-butanone), and pentachlorophenol).

Review of the leachate monitoring results for 2-chloroethyl vinyl ether, benzo (a) pyrene, and pentachlorophenol indicated that the none of these constituents have been detected in the leachate over the last 12 years and that the exceedances where being created by high reporting limits for several of the sampling events. For example, 2-chloroethyl vinyl ether had a non-detect reporting limit of 10,000 *u*g/L on a sample collected from leachate sump L302 on May 17, 2010, whereas the typical reporting limit for 2-chloroethyl vinyl over the remaining 12 year period was between 20 *u*g/L and 100 *u*g/L. At these lower reporting limits, the predicted groundwater concentration for 2-chloroethyl vinyl ether is less than the AGQS for 2-chloroethyl vinyl ether.

Benzo (a) pyrene had a non-detect reporting limit of 150 *u*g/L during 2nd Quarter 2012, 502 *u*g/L during 2nd Quarter 2018, and 291 *u*g/L during 4th Quarter 2020 on respective samples, whereas the typical reporting limit for benzo (a) pyrene over the whole 12 year period was 100 *u*g/L. At a reporting limit of 100 *u*g/L, the predicted groundwater concentration for benzo (a) pyrene is less than the AGQS for benzo (a) pyrene.

Pentachlorophenol had a non-detect reporting limit of 1,000 *u*g/L during 2nd Quarter 2012, 1,950 *u*g/L during 4th Quarter 2020, and 1,970 *u*g/L during 4th Quarter 2020 on respective samples, whereas the typical reporting limit for pentachlorophenol over the whole 12 year period was 500 *u*g/L. At a reporting limit of 500 *u*g/L, the predicted groundwater concentration for pentachlorophenol is less than the AGQS for pentachlorophenol.

Methyl ethyl ketone (2-butanone) and ammonia as nitrogen are the only detected constituents that indicate an exceedance of their respective AGQSs for this sensitivity analysis.

Review of the leachate monitoring results for methyl ethyl ketone (2-butanone) indicates that only three (3) of the 19 detections over the last 12 years produces an exceedance of its AGQS for this sensitivity analysis. At the average methyl ethyl ketone (2-butanone) concentration (4,545 *u*g/L) for the 19 detections, the predicted groundwater concentration for methyl ethyl ketone (2-butanone) is less than the AGQS for methyl ethyl ketone (2-butanone).

However, with three (3) of the 19 methyl ethyl ketone (2-butanone) detections producing an exceedance of its AGQS for this sensitivity analysis, the conservative assumptions used in this sensitivity analysis were examined. Apart from the basic conservative assumptions (outward seepage rate, use of maximum detected leachate concentrations, no degradation applied, no adsorption applied, constant concentration throughout the modeling period, calculating predicted groundwater concentrations at the base of the Wadsworth Formation (prior to the ZOA), etc.) that apply to all the sensitivity analyses, it should be noted that the minimum thickness (19.7 feet (6.00 m)) was calculated in the sump below the proposed vertical expansion area and that it is lined not only by the 60 mil HDPE geomembrane liner and five (5) foot recompacted cohesive soil liner but also a geosynthetic clay liner (GCL) and a second 60 mil HDPE geomembrane liner. All of the sumps in the vertical and horizontal expansion areas, have or are proposed to have the additional GCL and 60 mil HDPE geomembrane liner. These additional environmental safeguards greatly reduce the vertical seepage into and out of the landfill and make the minimum thickness sensitivity model extremely conservative.

Based on the presence of the GCL and additional 60 mil HDPE geomembrane liner in the sumps, a secondary minimum thickness (25.3 feet (7.71 m)) was calculated that did not include the sump areas, only including areas that have the same amount and types of layers (60 mil HDPE geomembrane liner, a five (5) foot recompacted cohesive soil liner, and Wadsworth Formation) as the baseline model.

When the secondary minimum thickness is run as a surrogate (in accordance with IEPA accepted methodology), the predicted groundwater concentration for all constituents (including ammonia as nitrogen, 2-chloroethyl vinyl ether, benzo (a) pyrene, methyl ethyl ketone (2-butanone), and pentachlorophenol) lies below their respective AGQS (See Appendix P for the surrogate secondary minimum thickness model run).

Review of the leachate monitoring results for ammonia as nitrogen indicates that two of the detections over the last 12 years do not indicate an exceedance of its AGQS for this sensitivity analysis. However, the other detections over the last 12 years do indicate an exceedance of its AGQS for this sensitivity analysis. As discussed above, there are several basic conservative assumptions (outward seepage rate, use of maximum detected leachate concentrations, no degradation applied, no adsorption applied, constant concentration throughout the modeling period, calculating predicted groundwater concentrations at the base of the Wadsworth Formation (prior to the ZOA), etc.) that apply to all the sensitivity analyses. Additionally, the minimum thickness (19.7 feet (6.00 m)) was calculated in a sump area that is lined not only by the 60 mil HDPE geomembrane liner and five (5) foot recompacted cohesive soil liner but also a geosynthetic clay liner (GCL) and a second 60 mil HDPE geomembrane liner.

As mentioned above, when the secondary minimum thickness (25.3 feet (7.71 m)) is run as a surrogate (in accordance with IEPA accepted methodology), the predicted groundwater concentration for ammonia as nitrogen lies below its respective AGQS (See Appendix P for the surrogate secondary minimum thickness model run).

Darcy Velocity

For the baseline model, the vertical Darcy velocity was conservatively calculated with one (1) foot of leachate head and poor liner conditions. This value is already overly conservative but was increased by one (1) order of magnitude for the sensitivity analysis. As a result, it was determined that a Darcy velocity increased by one order of magnitude would result in a satisfactory sensitivity evaluation of this parameter.

Sensitivity analysis of the above mentioned parameter resulted in satisfactory results for the sensitivity run (Appendix P). The increased vertical Darcy velocity through all modeled layers increased the predicted concentrations.

As discussed in the Model Results section, the model predicted representative maximum GCPF for the uppermost aquifer corresponds to the time period of 147 years. All the sensitivity analysis runs were carried out corresponding to a time period of 147 years.

The 'Summary of Results - Sensitivity Analysis' table in Appendix P includes all of the sensitivity analyses runs.

Model Reliability

The computer based transport model used in the present GIA is based on rigorous and sound analytical solutions to the advective and chemical transport equations. The equations were specifically derived for the purpose of modeling physical and chemical transport from subsurface waste impoundments. Numerous publications, comprehensive documentation and extensive use of this solution approach indicates the versatility of the model for groundwater impact assessment purposes (Rowe, 1987; Rowe and Booker, 1987; Rowe, 1988; Rowe and Booker, 1989; Rowe and Booker, 1985; Talbot, 1979). Results obtained using this solution approach are comparable to those obtained using other solution approaches to the transport equation (Rowe and Booker, 1990).

Conservativeness of Baseline Model

Site-specific data were used for input parameters in the baseline model when possible. When site-specific data were not available, appropriate input data was determined based on the extensive knowledge of the site and documented with research literature. These parameters, if they had a high degree of uncertainty, were conservatively estimated based upon research literature.

GIA Conclusions

This GIA was performed in order to evaluate the proposed Site 2 North Expansion and Site 2 design and site hydrogeologic conditions. The GIA transport model created to evaluate contaminant transport below the proposed Site 2 North Expansion and Site 2 yields groundwater concentration prediction factors, resulting in predicted groundwater concentrations at the base of the Wadsworth Formation (prior to ZOA) that are less than the permitted AGQS values.

The findings of this GIA demonstrate that the design features of the proposed facility are effective in protecting the groundwater quality in the Shallow Drift Aquifer (uppermost aquifer) at the proposed Site 2 North Expansion and Site 2 and the site hydrogeologic conditions are favorable for the development of the expansion.

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SECTION 2.8

Environmental Monitoring

2.8 ENVIRONMENTAL MONITORING PROGRAM

Introduction

The proposed Zion Landfill Site 2 North Expansion (Site 2 North Expansion) has been designed to be protective of the public, health, safety and welfare. To assure that the facility functions as designed, this Environmental Monitoring Program has been developed in accordance with applicable regulations and sound environmental practices. It includes a description of groundwater, leachate, subsurface gas, ambient air, and other environmental monitoring which will take place at the facility. The details of the Environmental Monitoring Program are described in greater detail within the following sections and within other sections of this Application.

Groundwater Monitoring

Groundwater Monitoring Overview

A groundwater monitoring program has been developed in accordance with 35 Ill. Admin. Code, Sections 811.318 and 811.319. The Groundwater Impact Assessment (GIA) has determined that groundwater quality will not be impacted at or beyond the edge of the zone of attenuation (ZOA) within 100 years after closure of the landfill, as discussed in Section 2.7 of the Application. Furthermore, the groundwater monitoring network will serve as an additional safeguard to verify that the landfill is not having any adverse impact on the groundwater quality and to provide an early warning system in the unlikely event of an impact. In other words, the groundwater monitoring network has been developed to provide assurance that the landfill will function as designed. The proposed groundwater monitoring network has been developed in accordance with current regulatory requirements based on: 1) the geology and hydrogeology, 2) the proposed landfill design features, and 3) the results of the well spacing model.

Title 35 Ill. Admin. Code Sections 811.318(b)(3) requires that monitoring wells be located as close to the potential source as practicable without interfering with operations and within one-half the distance from the edge of the potential source to the edge of the ZOA. The ZOA is located 100 feet from the waste boundary. As such, all new detection monitoring wells for the Uppermost Aquifer have been proposed to be located within 50 feet of the waste boundary.

Additionally, Title 35 Ill. Admin. Code Section 811.318(b)(2) requires that monitoring wells be located in hydrostratigraphic horizons that could serve as preferred contaminant migration pathways. Therefore, the proposed groundwater monitoring network has been designed to target the Shallow Drift Aquifer. The selection of this zone for monitoring is based on these units meeting the definition of the Uppermost Aquifer as stated in the Hydrogeologic Investigation Section (Section 2.2).

Groundwater will be routinely sampled and analyzed from the groundwater monitoring network. These monitoring results will be statistically analyzed to check that the background groundwater quality is not exceeded as defined in 35 Ill. Admin. Code Section 811.320.

Monitoring results, including the results of the data comparisons will be promptly reported in the Illinois Environmental Protection Agency (IEPA) following each sampling period.

Monitoring Well Spacing Determination

The Monitoring Analysis Package (MAP) was utilized to develop the proposed monitoring network and assure that it exceeds IEPA requirements. The Plume Generation Model (PLUME), one of three modeling packages contained within the MAP application, was utilized to determine the appropriate monitoring well spacing while taking into account current hydrogeological characteristics.

PLUME utilizes a fundamental two-dimensional analytical transport model responsible for configuring plumes. The governing equation for the transport model, originally presented in Domenico and Robbins (1985) and later modified by Domenico (1987), is:

$$C(x,y,t) = \left(\frac{C_O}{4}\right) \exp\left\{\left(\frac{xv}{2\alpha_x}\right)\left[1-\left(1+\frac{4\lambda\alpha_X}{v^2}\right)^{1/2}\right]\right\}$$

$$efrc\left\{\frac{\left[x - vt\left(1+\frac{4\lambda\alpha_X}{v^2}\right)^{1/2}\right]}{2(\alpha_X t)^{1/2}}\right\}$$

$$\left\{erf\left[\frac{\left(y+\frac{S_W}{2}\right)}{2\left(\frac{\alpha_Y x}{v}\right)^{1/2}}\right] - erf\left[\frac{\left(y-\frac{S_W}{2}\right)}{2\left(\frac{\alpha_Y x}{v}\right)^{1/2}}\right]\right\}$$

where,

C (x,y,t) =	The concentration of the contaminant at location x, y from the source at time t;
C_O =	Source concentration - the highest concentration of the contaminant in the groundwater at the source;
x =	Distance from planar source to the location of concern along the center line of the plume;
y =	Distance from planar source to the location of concern perpendicular to the centerline of the plume;
λ =	1st order decay constant;
S_W =	Width of source area;
v =	Average Contaminant Velocity (ki/n_e);

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α_x = Dispersivity in the x direction;

α_y = Dispersivity in the y direction; and

t = Time.

To determine an appropriate down-gradient well spacing, hypothetical plumes were generated with PLUME using site specific input parameters presented in the Hydrogeologic Investigation Report (Section 2.2) and as described in greater detail within the following section. Source leaks at the landfill base were assumed and average advection times of 33,950 days on the northwest side, 23,900 days on the north side, 181,000 days on the northeast side, and 180,500 days on the southeast side of the landfill were found to maximize the extent of the PLUMES while assuring that they do not extend past the zone of attenuation on the northwest, northeast, and east (down-gradient) sides of the landfill. The modeled plumes were then able to be used to determine what minimum well spacing will be necessary to assure that any leak would be detected.

PLUME Input Data

Units. Consistent units of meters and days were used within the PLUME model.

Advection Time. As previously indicated, advection times of 33,950 days on the northwest side, 23,900 days on the north side, 180,500 days on the northeast side, and 165,000 days on the southeast side of the landfill were used in order to maximize the extent of the plumes while keeping them within the zone of attenuation along the northwest, northeast, and east (down-gradient) sides of the landfill.

Dilution Contours. Dilution contours are utilized by PLUME as criterion by which to illustrate the shape of the hypothetical plume at a percentage of the source concentration. The MAP User's Manual defines a dilution contour as the ratio of the concentration of the contaminant at the detected point in the plume to the concentration of the source. MAP documentation suggests that the concentration of the contaminant at the outermost perimeter of the plume (detection point) is equal to the laboratory's detection limit. The concentration at the source is the concentration of the constituent as it occurs in leachate. Chloride is chosen to represent the constituent released in a hypothetical plume from the landfill, because it is transported conservatively due to its resistance to degradation and non-sorbing properties. The laboratory detection limit of chloride is 1.0 mg/L. The IEPA recommends utilizing 2,000 mg/L as the concentration of chloride in leachate for the modeling purposes, however the model used a slightly more conservative site specific concentration of 1,945 mg/L, which is the average concentration of chloride in leachate at the existing landfill from 2010 through 2019. The resultant outermost dilution contour of 5.14×10^{-4} was used for the model to define the shape of the plume.

Longitudinal Dispersivity. Longitudinal dispersivity is derived from the following empirical equation developed by Schulze-Makuch (2005):

$$\alpha_L = 0.085(L)^{0.81}$$

where,

α_L = longitudinal dispersivity; and
L = flow path length.

It is conservatively assumed that a failure occurs at the downward gradient edge of the proposed landfill at the base of the landfill sideslope. Therefore, the flowpath length for the northwestern portion of the downgradient edge of the proposed expansion is determined as follows:

$$\begin{aligned} L &= D1 + D2 \\ &= 221.18ft + 50ft \\ &= 271.18ft = 82.66m \end{aligned}$$

where,

L = Flow Path Length for the northwestern portion of the downgradient edge of the proposed expansion,
D1 = Average Distance from the base of the leachate collection system to the waste boundary across the northwestern edge of the proposed landfill; and
D2 = Average Distance from Waste Boundary to Compliance Point.

The flowpath length for the northern portion of the downgradient edge of the proposed expansion is determined as follows:

$$\begin{aligned} L &= D1 + D2 \\ &= 136.13ft + 50ft \\ &= 186.13ft = 56.73m \end{aligned}$$

where,

L = Flow Path Length for the northern portion of the downgradient edge of the proposed expansion,
D1 = Average Distance from the base of the leachate collection system to the waste boundary across the northern edge of the proposed landfill; and
D2 = Average Distance from Waste Boundary to Compliance Point.

The flowpath length for the northeastern portion of the downgradient edge of the proposed expansion is determined as follows:

$$\begin{aligned} L &= D1 + D2 \\ &= 181.90ft + 50ft \\ &= 231.90ft = 70.68m \end{aligned}$$

where,

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L = Flow Path Length for the northeastern portion of the downgradient edge of the proposed expansion,
D1 = Average Distance from the base of the leachate collection system to the waste boundary across the northeastern edge of the proposed expansion; and
D2 = Average Distance from Waste Boundary to Compliance Point.

The flowpath length for the southeastern portion of the downgradient edge of the proposed expansion is determined as follows:

$$L = D1 + D2$$
$$= 203.92ft + 50ft$$
$$= 253.92ft = 77.39m$$

where,

L = Flow Path Length for the southeastern portion of the downgradient edge of the proposed expansion,
D1 = Average Distance from the base of the leachate collection system to the waste boundary across the southeastern edge of the proposed expansion; and
D2 = Average Distance from Waste Boundary to Compliance Point.

Longitudinal dispersivity for the northwestern edge of the proposed landfill is calculated as follows:

$$\alpha_L = 0.085(82.66)^{0.81} = 3.04m$$

Longitudinal dispersivity for the northern edge of the proposed landfill is calculated as follows:

$$\alpha_L = 0.085(56.73)^{0.81} = 2.24m$$

Longitudinal dispersivity for the northeastern edge of the proposed expansion is calculated as follows:

$$\alpha_L = 0.085(70.68)^{0.81} = 2.68m$$

Longitudinal dispersivity for the southeastern edge of the proposed expansion is calculated as follows:

$$\alpha_L = 0.085(77.39)^{0.81} = 2.88m$$

Transverse Dispersivity. In accordance with IEPA LPC-PA2, the transverse dispersivity is determined as 20% of the longitudinal dispersivity.

Diffusion Coefficient. The diffusion coefficient of the Uppermost Aquifer was assumed to be 0.064 m^2/y (1.75 x 10^{-4} m^2/d) which is the "free solution" diffusion coefficient for chloride at infinite dilution in water at 25^0 C[1]. This value is conservative when evaluating the movement of a contaminant through a porous media such as the Uppermost Aquifer.

Average Contaminant Velocity. The average contaminant velocity is defined as follows[2]:

$$V = \frac{ki}{n_e}$$

where,

v = Average Contaminant Velocity;
k = Geometric Mean Horizontal Hydraulic Conductivity;
i = Average Gradient (February 2019 through February 2021)[3]; and
n_e = Average Effective Porosity.

$$V(north) = \frac{112.58(0.002030)}{0.367} = 0.62\ m/yr$$

$$V\ (east) = \frac{112.58(0.000359)}{0.367} = 0.107\ m/yr$$

The Average Contaminant Velocity used for the north side is the highest calculated seepage velocity of 0.62 m/yr for the Uppermost Aquifer (0.0017 m/d).

The Average Contaminant Velocity used for the east side is the highest calculated seepage velocity of 0.107 m/yr for the Uppermost Aquifer (0.0003 m/d).

Width of Line Source. As suggested in IEPA LPC-PA2, the width of line source is 1.00 m. This value was used in the model.

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[1] R. Kerry Rowe, Robert M. Quigley, Richard W.I. Brachman & John R. Booker (2004). Barrier Systems for Waste Disposal Facilities. CRC Press, London.

[2] Walton, William C. (1991). Principals of Groundwater Engineering. Lewis Publishers, Inc., Chelsea, Michigan.

[3] The gradients used for calculation of the average contaminant velocity are the average of measurements taken from potentiometric data collected in February 2019 through February 2021 (data available at time of publication of the Application for Local Siting Approval for this proposed expansion).

Results of PLUME Model

The results of the PLUME evaluation indicate that well spacings of approximately 169.90 feet on the northwest, 117.48 feet on the north, 183.90 feet on the northeast, and 182.80 feet on the southeast (down-gradient) sides of the landfill will be adequate to detect any potential leak (refer to **Figures 2.8-1 and 2.8-2**). The output files from the PLUME models are included in **Appendix Q**.

It should be noted that the PLUME modeling is overly conservative and has resulted in a proposed well spacing that is much tighter than the minimum 200 foot spacing typically allowed by the IEPA. The modeling did not consider the significant environmental safeguards that are inherent in the landfill design or the conservative assumptions that have been used in the Groundwater Impact Assessment models.

The proposed landfill design includes a composite liner system consisting of a 60-mil HDPE geomembrane liner and a 4-foot compacted cohesive soil liner (1 x 10-7 cm/sec), leachate and landfill gas collection and removal systems, and a composite final cover. In addition, the base of the landfill will be below the potentiometric surface, creating an inward gradient landfill. The inward gradient will limit the potential outward migration of any contaminant to diffusion. Groundwater will flow into the landfill during the active life of the landfill and the post closure care period rather than leachate attempting to exit the landfill. The monitoring network serves as an additional safeguard to monitor the groundwater sources at the facility, verify that the landfill design is functioning as intended, and provide an early warning system in the unlikely event of a release.

Furthermore, in the PLUME models, liner failure was assumed to occur on the down gradient edge of the proposed landfill at the base of the landfill sideslope, therefore reducing the flow path length of the hypothetical plume. This reduced flow path length was used in the determination of the longitudinal dispersivity. It would be more realistic for a release to occur in areas other than the leachate sump and trench areas which will be lined with two 60-mil HDPE liners and a geosynthetic clay liner (sandwiched between the two 60 mil HDPE liners). It would seem reasonable to increase the flow path length and calculate it from the interior of the landfill. Calculating the flow path length from the interior of the landfill would increase the longitudinal dispersivity and widen the hypothetical plume. It would also increase the transverse dispersivity. As a result, an increased flow path length would result in a wider hypothetical plume that could be detected and, therefore, a wider well spacing.

In addition, the MEMO models assumed a line source of one (1) meter. However, a diffusion driven release from this inward gradient landfill will result in a wider source area, creating a wider hypothetical plume that could be detected.

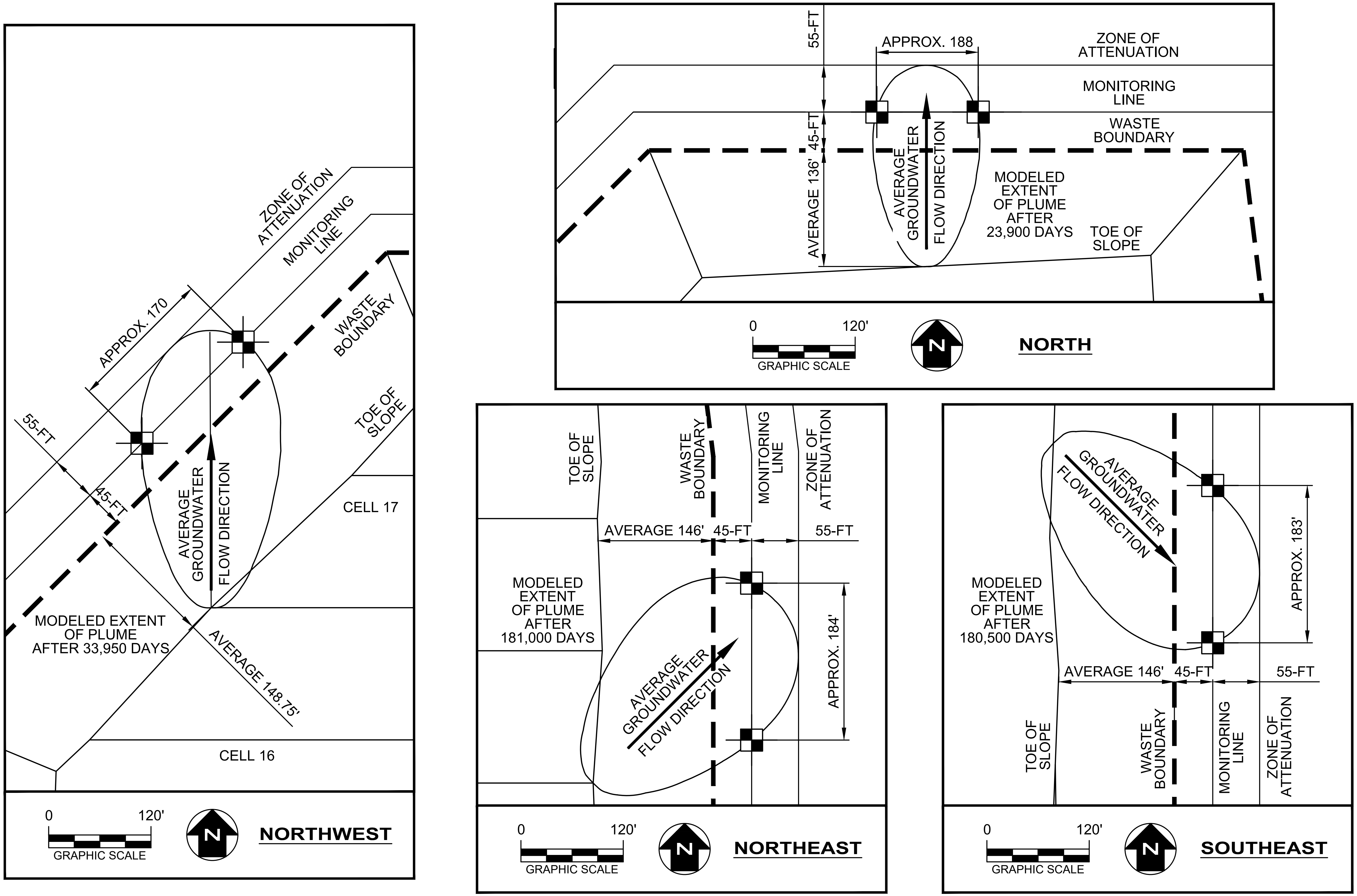
Moreover, the results of the permitted Groundwater Impact Assessment have demonstrated that the facility will not have an adverse impact on the groundwater quality. This assessment included the use of conservative model assumptions including a constant concentration, outward gradient, poor liner contact used to determine the seepage rate, and did not include the application of adsorption of degradation. The GIA determined that the proposed landfill will not adversely impact the groundwater quality at or beyond the edge of the ZOA within 100 years of landfill closure.

Description of the Proposed Monitoring Network

The proposed monitoring network for the landfill will include a total of 27 detection monitoring wells within the Uppermost Aquifer (G300 through G326). Down-gradient monitoring wells G302 through G306 have been spaced approximately 183 feet apart. Down-gradient monitoring wells G307 through G314 have been spaced approximately 184 feet apart. Down-gradient monitoring wells G315 through G321 have been spaced approximately 117 feet apart. Down-gradient monitoring wells G322 through G326 have been spaced approximately 170 feet apart. Additionally, two up-gradient monitoring wells (G300 and G301) have been added in order to provide continuous background groundwater quality data. The monitoring wells will be installed prior to waste placement in the cells to be monitored as cell development progresses. Proposed monitoring well G304 will be located down-gradient of the first cells to be constructed (Cell 11 and Cell 12) and will be installed at the compliance boundary (i.e. edge of the ZOA) in accordance with 35 Ill. Admin. Code Section 811.318(b)(5). This compliance boundary well is proposed to remain in operation during the life of the landfill and throughout the post-closure period. The proposed monitoring network for the landfill is depicted on **Figure 2.8-2** and on **Drawing No. D12**. A typical monitoring well is shown in **Photograph 2.8-1**.

Photograph 2.8-1 Typical Monitoring Well

It should be noted that during the installation of the 27 new detection monitoring wells proposed withing this application, a nested well may also be installed within any saturated intra-till sediments that may be encountered above the Uppermost Aquifer. Should nested wells be necessary, the final monitoring network will consist of more than the 27 monitoring wells indicated above.



REV. NO.	DATE	DESCRIPTION

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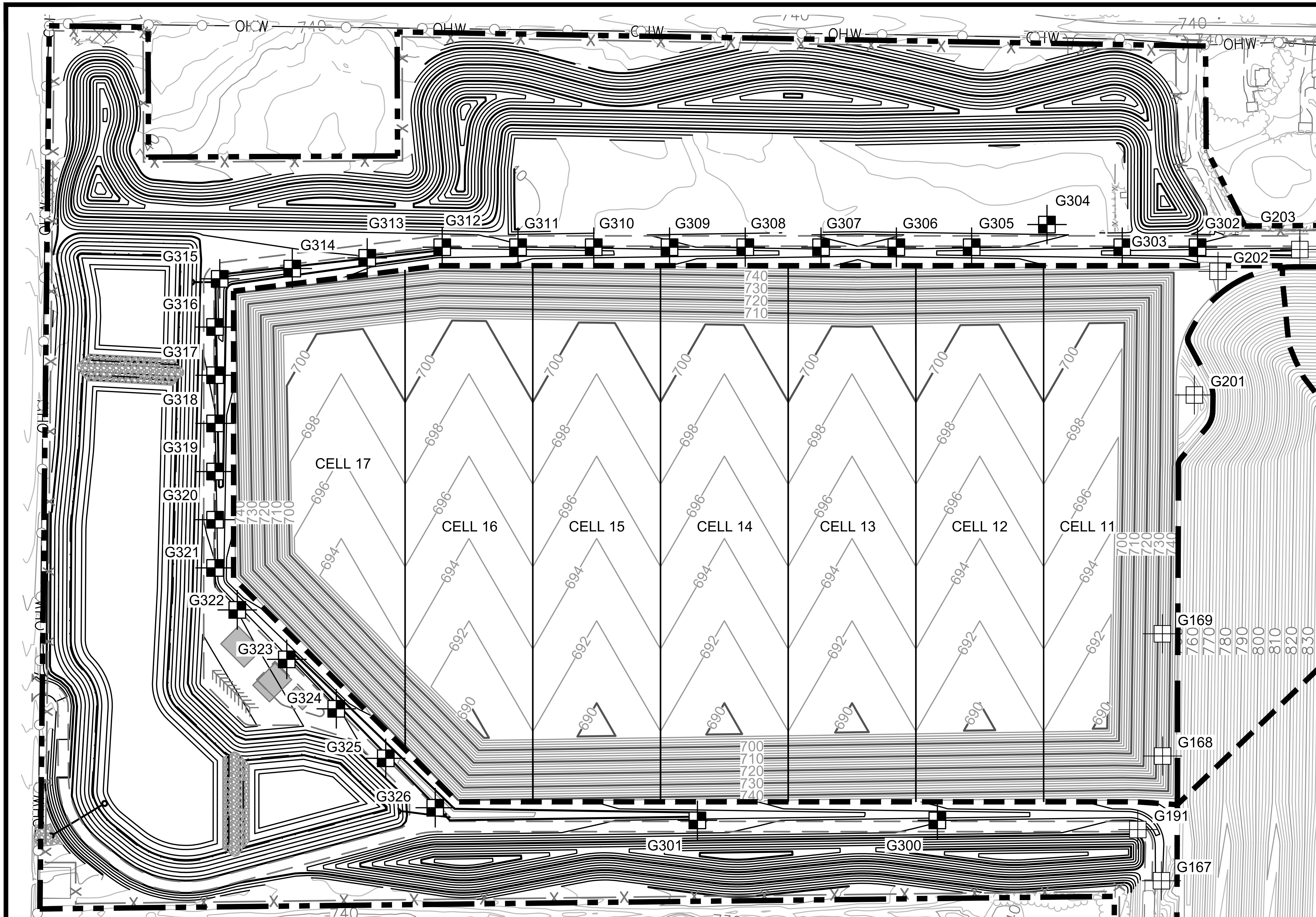
ZION LANDFILL - SITE 2 NORTH EXPANSION
CITY OF ZION, ILLINOIS

FIGURE 2.8-1
WELL SPACING DETERMINATION DIAGRAM

DRAWN BY:	MRL	APPROVED BY:	DAM	PROJ. NO.:	631020105	DATE:	MAY 2022



N

0 250'
GRAPHIC SCALE

LEGEND

- APPROXIMATE FACILITY BOUNDARY
- APPROXIMATE EXISTING WASTE BOUNDARY
- APPROXIMATE PROPOSED EXPANSION WASTE BOUNDARY
- G201 EXISTING MONITORING WELL
- G301 PROPOSED MONITORING WELL

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ZION LANDFILL - SITE 2 NORTH EXPANSION
CITY OF ZION, ILLINOIS

FIGURE 2.8-2
PROPOSED FINAL GROUNDWATER MONITORING WELL NETWORK (MODELED SPACING VERSION)

DRAWN BY:	MRL	APPROVED BY:	DAM	PROJ. NO.:	631020105	DATE:	MAY 2022

Monitoring Well Phasing

The groundwater monitoring network will be developed in phases so that each well will be installed prior to accepting waste in the cell(s) that the wells are intended to monitor. **Table 2.8-1** provides a summary of the groundwater monitoring wells and the installation / phasing status of each monitoring point.

Establishment of Applicable Groundwater Quality Standards

Applicable Groundwater Quality Standard (AGQS) values have been established for the Uppermost Aquifer (Shallow Drift Aquifer) and the Intratill Sediments at the existing Zion Landfill. These permitted AGQS values were used in the GIA model. Applicable pages of the permit which indicate permitted AGQS values for the existing Landfill have been provided in **Appendix Q**.

The AGQS values may be revised to incorporate new standards, additional wells, or intra-well evaluations as approved by the IEPA using Sanitas Groundwater Monitoring statistical software (Sanitas). Prior to calculation of the AGQS values, groundwater monitoring data will be evaluated for potential outliers and spatial variance using Sanitas.

Upon completion of the outlier and spatial variance evaluations, statistical analyses will then be performed in accordance with the USEPA 1992 Standards. Ultimately, the AGQS values will be determined using appropriate procedures specific to each constituent due to the characteristics of its data set (i.e. number of non-detects, normality, etc.).

The Sanitas software allows for the development of AGQSs through the use of a built-in decision logic framework that assures consistency with the USEPA's statistical requirements. The decision logic framework allows the software to move through the series of statistical step flow charts and testing algorithms, ultimately choosing the most appropriate statistical method and making any necessary adjustments or transformations.

For these analyses, normality will first be evaluated using Shapiro-Wilk Test with a specified alpha of 99 percent. Sanitas then utilizes a variety of power transformations in an attempt to normalize the distribution for use in the parametric tests (ladder of powers). The software then chooses the data transformation that normalizes the data with the least powerful transformation. When necessary, the software automatically substitutes a value of one half of the method detection limit for non-detects.

Parametric tests will be performed on normal and log normal datasets when the number of non-detects for a sample set is found to be less than 50 percent. Cohen's Adjustment will be used on the sample mean when the number of non-detects is found to be between 15 and 50 percent.

Additionally, if all the background values are less than the MDL for a given parameter, the Practical Quantitation Limit (PQL) will be used to evaluate data from the monitoring wells. Therefore, the AGQSs for the parameters which are non-detections will be set at their respective PQLs.

It should be noted that following the USEPA statistical requirements, as well as the use of Sanitas software, has traditionally been accepted by the IEPA.

TABLE 2.8-1 PROPOSED GROUNDWATER MONITORING WELL NETWORK PHASING								
Well Name	Location (Site-Specific Coordinate System)		Location (NAD83 Illinois State Planes, East Zone, US Foot)		Ground Surface Elevation	[1,2]Bottom of Screen Elevation	Depth to Bottom of Screen	Installation / Phasing
	Northing	Easting	Northing	Easting	ft MSL	ft MSL	ft bgs	
G300	13044.49	11685.73	2120548.40	1109231.59	745.55	640.55	105.00	Up-gradient well to be installed within 50 feet of the waste boundary prior to Cell 11 operations.
G301	13626.9	11685.73	2121130.81	1109234.64	742.17	645.82	96.35	Up-gradient well to be installed within 50 feet of the waste boundary prior to Cell 11 operations.
G302	12412.86	13077.70	2119909.49	1110620.22	743.06	639.89	103.17	Up-gradient well to be installed within 50 feet of the waste boundary prior to Cell 11 operations.
G303	12595.66	13077.70	2120092.29	1110621.18	744.02	640.31	103.71	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 11 Operations.
G304	12778.46	13132.70	2120274.80	1110677.14	745.57	640.76	104.81	Down-gradient well to be installed at the Zone of Attenuation prior to Cell 11 operations.
G305	12961.26	13077.70	2120457.88	1110623.10	746.02	641.27	104.75	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 12 operations.
G306	13144.06	13077.70	2120640.68	1110624.05	746.26	642.18	104.08	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 13 operations.

TABLE 2.8-1 PROPOSED GROUNDWATER MONITORING WELL NETWORK PHASING								
Well Name	Location (Site-Specific Coordinate System)		Location (NAD83 Illinois State Planes, East Zone, US Foot)		Ground Surface Elevation	[1,2]Bottom of Screen Elevation	Depth to Bottom of Screen	Installation / Phasing
	Northing	Easting	Northing	Easting	ft MSL	ft MSL	ft bgs	
G307	13326.86	13077.70	2120823.48	1110625.01	746.41	643.09	103.33	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 13 operations.
G308	13510.76	13077.70	2121007.38	1110625.98	746.00	643.80	102.20	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 14 operations.
G309	13694.66	13077.70	2121191.27	1110626.94	744.48	643.99	100.48	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 14 operations.
G310	13878.56	13077.70	2121375.17	1110627.90	744.01	644.12	99.89	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 15 operations.
G311	14062.46	13077.70	2121559.07	1110628.87	743.19	644.18	99.01	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 16 operations.
G312	14246.36	13077.70	2121742.97	1110629.83	742.16	643.18	98.98	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 16 operations.
G313	14428.42	13051.76	2121925.16	1110604.85	740.48	641.22	99.26	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.

TABLE 2.8-1 PROPOSED GROUNDWATER MONITORING WELL NETWORK PHASING								
Well Name	Location (Site-Specific Coordinate System)		Location (NAD83 Illinois State Planes, East Zone, US Foot)		Ground Surface Elevation	[1,2]Bottom of Screen Elevation	Depth to Bottom of Screen	Installation / Phasing
	Northing	Easting	Northing	Easting	ft MSL	ft MSL	ft bgs	
G314	14610.49	13025.83	2122107.36	1110579.87	738.59	639.26	99.33	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G315	14786.52	13000.82	2122283.52	1110555.78	740.00	638.34	101.66	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G316	14797.81	12883.82	2122295.42	1110438.84	740.15	638.54	101.61	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G317	14797.81	12766.82	2122296.03	1110321.84	740.46	639.00	101.46	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G318	14797.81	12649.82	2122296.64	1110204.84	742.01	639.45	102.56	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G319	14797.81	12532.82	2122297.26	1110087.84	742.04	639.90	102.14	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G320	14797.81	12415.82	2122297.87	1109970.84	742.34	640.35	101.99	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.

TABLE 2.8-1
PROPOSED GROUNDWATER MONITORING WELL NETWORK PHASING

Well Name	Location (Site-Specific Coordinate System)		Location (NAD83 Illinois State Planes, East Zone, US Foot)		Ground Surface Elevation	[1,2]Bottom of Screen Elevation	Depth to Bottom of Screen	Installation / Phasing
	Northing	Easting	Northing	Easting	ft MSL	ft MSL	ft bgs	
G321	14797.81	12298.82	2122298.48	1109853.85	744.00	641.08	102.92	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G322	14743.87	12196.86	2122245.08	1109751.61	743.78	642.15	101.62	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G323	14623.73	12076.73	2122125.57	1109630.85	742.22	643.78	98.44	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G324	14503.59	11956.59	2122006.07	1109510.08	742.08	644.96	97.12	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G325	14383.46	11836.45	2121886.56	1109389.32	740.40	645.93	94.47	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 17 operations.
G326	14263.32	11716.31	2121767.05	1109268.55	740.00	646.90	93.11	Down-gradient well to be installed within 50 feet of the waste boundary prior to Cell 16 operations.

Notes:
1. The screened interval will be approximately 5-10 feet.
2. The proposed groundwater monitoring network has been designed to target the Uppermost Aquifer (Shallow Drift Aquifer).

Maximum Allowable Predicted Concentrations (MAPCs)

The GIA in Section 2.7 demonstrates that the proposed expansion will not cause an exceedence of any of the constituent concentrations over the AGQS values at or beyond the edge of the ZOA within 100 years of landfill closure for the Uppermost Aquifer. MAPC values were conservatively set equal to the AGQS values.

Design and Construction of Monitoring Wells

All monitoring wells for the Site 2 North Expansion will be designed and constructed in accordance with the following procedures:

1. Standards established in 35 Ill. Admin. Code, Section 811.318(d);
2. IEPA guidance;
3. Standard Practice for Design and Installation of Groundwater Monitoring Wells in Aquifers, ASTM D 5092-90; and
4. Monitoring Well Design and Construction, Chapter 3, RCRA Groundwater Monitoring Technical Enforcement Guidance Document, U.S. EPA, September 1986.

A typical as-built diagram for groundwater monitoring well construction is provided in **Appendix Q** and on **Drawing No. D20**. The monitoring wells will be constructed to yield groundwater samples that represent the quality of groundwater at the landfill site.

The procedure for constructing the monitoring wells at the landfill will typically consist of the following steps:

1. Prior to well construction, all monitoring well locations will be staked in the field by a survey crew under the supervision of a Professional Land Surveyor licensed in the State of Illinois;
2. Borings will be drilled and continuously sampled to the target depth at each monitoring well location. Drilling fluids will be avoided to the extent practible. Soil samples will typically be obtained by either advancing a 5-foot continuous split core barrel (or similar), driving a 2-inch outside diameter split-spoon sampler (ASTM D 1586) or pushing a thin-walled 3-inch diameter Shelby tube sampler (ASTM D 1587). A geologist or geotechnical engineer will direct the field exploration operations, log the soil samples, and document the well construction. Boreholes within 10 feet of an existing continuously sampled boring need not be continuously sampled through the depth intervals that were previously sampled;
3. Each monitoring well will be constructed using a 2-inch inside diameter, flush joint, well screen and riser pipes. The screen length for the proposed monitoring wells will be approximately 5 or 10 feet. An end plug will be placed at the bottom of the screen and a vented cap will be placed on the top. Monitoring wells may be constructed of PVC, stainless steel, Teflon,

or other materials approved by the IEPA. All threaded joints will be sealed using either manufacturer supplied O-rings or Teflon tape;

4. A filter pack will be constructed in each well by filling the annular space with silica sand (approximately 2-1/2 to 3 times larger than the 50% grain size of the zone being monitored) to a depth of approximately 2 feet above the top of the screen. If the in-situ material is appropriate (i.e. sand and gravel), then the formation may be allowed to collapse around the well screen to the desired elevation;

5. A minimum 2-foot-thick bentonite chip or pellet seal may be placed above the top of the filter pack if the seal can be placed without bridging the chips or pellets. Otherwise, an approximate 3-foot thick bentonite slurry seal may be placed above the sandpack using a tremie pipe method;

6. The annular space above the bentonite seal and/or sand pack will be grouted to within 2 to 4 feet of the ground surface with a bentonite Volclay® grout, or equivalent, using the tremie method;

7. Concrete will be used to top off the annular space at the ground surface;

8. A well protector with a locking lid will then be installed in the concrete to protect and secure the monitoring well;

9. The well protector will be clearly labeled with the monitoring well number;

10. A concrete pad will be constructed around the well protector. The pad will be sloped to divert surface water away from the well;

11. The drill tooling, sampling equipment, and well screen/riser pipe that contact the in-situ geologic materials will be decontaminated using a hot water pressure washer prior to drilling each borehole. Field decontamination of certified pre-cleaned well screen/riser pipe materials will not be required. The sampling equipment will be washed in a solution of Alconox™ (or equivalent) and potable water and then rinsed in potable water prior to each use; and

12. The monitoring wells will be developed to ensure that the well screens are unobstructed and that representative groundwater is flowing into the wells.

The construction of each monitoring well will be documented by completing and submitting the IEPA Well Completion Report, the Illinois Department of Public Health (IDPH) Well Construction Report form, and an as-built diagram as provided in **Appendix Q**.

Monitoring Well and Boring, Plugging, and Abandonment

Test borings, damaged wells or piezometers and wells or piezometers no longer used for long-term monitoring at the landfill will be abandoned in accordance with 35 Ill. Admin.

Code, Section 811.315 and 811.316 Plugging and Sealing of Drill Holes, and in accordance with 77 Ill. Admin. Code, Section 920.120. Abandonment procedures as described below will be followed in the event a monitoring well becomes unserviceable and must be replaced. Abandonment procedures will also be used if any unknown wells are encountered during site development. The grout used to abandon the wells will typically be a pure bentonite grout. The specific abandonment procedures are provided in the following sections.

Test Boring Abandonment

Any test borings to be drilled at the landfill for site development will be surveyed and properly abandoned as described in this section. Abandoment will be documented by a geologist or engineer.

Test borings temporarily left unattended (e.g., to obtain water elevation readings) will be temporarily covered and marked (e.g., using flagged lath). The temporary cover will minimize the flow of stormwater runoff into the boring and prevent accidental entry by animals. If an uncased boring partially or completely collapses, resulting in a potential contaminant migration pathway, the borehole will be redrilled prior to abandonment. Immediately after the required data has been collected or the boring has been redrilled, the boring will be abandoned in accordance with the following procedure.
A tremie pipe will be inserted to the bottom of each boring to be advanced. If the boring collapses, the tremie pipe will be inserted through the hollow stem augers or casing. The slurry will be tremied under pressure. As the formation water is displaced, the tremie pipe will be withdrawn. The bottom of the augers and the tremie pipe will remain just below the top of the slurry until the grout reaches the ground surface.

The surveyed ground elevation and the location of the abandoned borehole will be recorded by the supervising engineer, geologist. An abandoned boring certification form will be completed and submitted to the IEPA in accordance with permit conditions and the IDPH requirements. A copy of this form is included in **Appendix Q.**

Monitoring Well or Piezometer Abandonment

A groundwater monitoring well or piezometer required to be removed from service will be abandoned in accordance with the following procedure.

For monitoring wells or piezometers in which the well is screened in bedrock, the following plugging procedure should typically be used (it is assumed that any obstruction in the well casing will be removed prior to this procedure; if an obstruction is not able to be removed, the second procedure described below should be followed):

1. Cut casing off at desired depth;
2. Mix grout;
3. Insert tremie pipe into well and extend to bottom;

4. Slowly pump slurry under low pressure through tremie pipe;

5. Slowly withdraw tremie pipe making sure bottom of pipe remains below the grout slurry mix; and

6. Continue slow pumping until all formation water and the grout is displaced from top of casing.

For monitoring wells or piezometers which were screened in unconsolidated sediments, the following procedure should typically be used:

1. Knock out and remove thin surface concrete plug, if present;

2. Re-auger entire length of well;

3. Remove well casing from re-augered borehole;

4. Mix grout;

5. Insert tremie pipe into augers and extend to bottom;
6. Slowly pump grout under low pressure through tremie pipe;

7. Continue slow pumping until all formation water and the watery slurry mix is displaced from top of casing;

8. Slowly withdraw tremie pipe making sure bottom of pipe remains below the grout;

9. Pull a flight of augers; and

10. Top off grout after each flight is removed.

The ground elevation and the location of the abandoned monitoring well or piezometer will be recorded by the supervising engineer or geologist. An abandoned monitoring well certification form will be completed and will be submitted to the IDPH and the IEPA in accordance with permit conditions and IDPH requirements.

Groundwater and Leachate Sampling Procedures

Upon approval of the IEPA, dedicated submersible pumps will be utilized to sample each monitoring well using low flow purging techniques. The detailed sampling procedure (including procedures for sample preservation and chain of custody) that will be followed to collect leachate or groundwater samples from the monitoring wells where a dedicated submersible pump is utilized is provided in **Appendix Q**. Care will be taken to decontaminate all equipment to prevent possible cross contamination of wells. Depth to water from top of riser and elevation of the groundwater surface in reference to Mean Sea Level (MSL) datum will also be provided.

Traditional Groundwater Sampling for a Well Without a Dedicated Pump

In the case that traditional groundwater sampling is required, the following procedures will be followed:

After unlocking the monitoring well protector and removing the vented cap, the water level will generally be obtained utilizing an electronic water level indicator. After the water level is recorded, a minimum of three (3) well volumes of water will be evacuated from the monitoring well if possible. Field measurements of water level, water temperature, pH, conductivity and well depth will be recorded after each well volume is removed.

The groundwater sample will be marked appropriately and logged on the water sample chain of custody records. Water samples will be stored on ice and transported or shipped to the laboratory in a cooler or other suitable container. The laboratory will be capable of performing all analytical analysis in accordance with standard testing methods as approved by the state. Upon arrival at the laboratory, water samples and the chain of custody records will be surrendered to the laboratory. By following these quality assurance procedures, the potential for false positives should be minimized. **Photograph 2.8-2** depicts a sample being pulled from a typical monitoring well.

Sampling and testing will be governed by the approved IEPA permit and applicable State regulations.

Detection Monitoring Parameters, Frequency and Data Analyses

Groundwater monitoring at the landfill can be divided into the following three stages:

1. Monitoring prior to accepting waste;
2. Monitoring during the landfill operations; and
3. Monitoring during post-closure.

The specific monitoring program for each stage is detailed in the following sections.

Photograph 2.8-2 Sampling of a typical monitoring well

Monitoring Prior to Accepting Waste

As cell development progresses, all groundwater monitoring wells designated for each cell will be installed prior to accepting waste in that cell. Documentation of well construction will generally be provided with the application for a significant permit modification for operating authorization for each landfill cell.

Detection Monitoring During Landfill Operation

Groundwater monitoring will be performed quarterly and semi-annually (depending on the well location) in accordance with 35 Ill. Admin. Code, Section 811.319 for the indicator parameters required within 35 Ill. Adm. Code, Section (a)(2). Organic constituents will be monitored within each new well within three months of installation and will be added to the monitoring list at least once every two years in accordance with 35 Ill. Admin. Code, Section 811.319(a)(3). The detection monitoring analytical results for the permitted monitoring wells will be evaluated in accordance with 35 Ill. Admin. Code, Section 811.319(a)(4).

Monitoring During Post-Closure

Monitoring during post-closure will remain unchanged from that performed during landfill operations, unless a change to the monitoring program is approved by the IEPA as provided for in 35 Ill. Admin. Code, Section 811.319.

Statistical Analysis of Groundwater Quality Data

As required by 35 Ill. Admin. Code, Section 811.320, routine groundwater quality monitoring data will be analyzed by comparing the results of the groundwater sampling to AGQS and MAPC values which have been established at the site using the applicable statistical procedure specific to each particular constituent and its background data set.

The routine groundwater quality monitoring data will be compared to the AGQS and MAPC values. The applicable water quality standards may be revised to incorporate new standards, additional wells, or intra-well evaluations as approved by the IEPA. The AGQS values that will be used for groundwater quality evaluation are summarized in the GIA in Section 2.7 of the Application. Additionally, applicable pages of the permit which indicate permitted AGQS values for the existing landfill have been provided in **Appendix Q**.

Evaluation of Groundwater Quality Data

The groundwater quality data for the routine monitoring parameters will be evaluated in accordance with Title 35 Ill. Admin. Code, Section 811.319(a)(4). The current required evaluations include the comparison of the concentration of constituents in wells:

1. Over the last eight consecutive monitoring periods;
2. To the applicable MAPC values (if established);
3. To the preceding measured concentration (for the organic constituents); and
4. To the applicable AGQS values.

As the AGQS and MAPC values have been established pursuant to statistical procedures, the comparison in item numbers 2 and/or 4 above will satisfy the requirement of Title 35 Ill. Admin. Code, Section 811.320(e) for statistical analysis of groundwater monitoring data. According to current regulations, a monitored (observed) increase occurs when:

1. The concentration of any constituent monitored in a particular monitoring well shows a progressive increase over eight consecutive monitoring periods;
2. The concentration of any constituent in a particular monitoring well exceeds the MAPC values at an established monitoring point within the zone of attenuation;
3. The concentration of any organic constituent monitored annually in a particular monitoring well exceeds the preceding measured concentration; and
4. The concentration of any constituent monitored in a particular monitoring well at or beyond the zone of attenuation exceeds its AGQS value.

In the event a monitored (observed) increase occurs, Zion Landfill, Inc. will, within 48 hours of the observed increase, obtain a representative sample of the source water in each well which is located within 200 feet of the affected well and whose owners have agreed to participate in the monitoring program per the terms of the host agreements in Appendix C.

Confirmation of Observed Increase

The observed increase will be confirmed in accordance with 35 Ill. Admin. Code, Section 811.319(a)(4)(B). Current confirmation procedures generally includes taking additional samples within 90 days of the initial observation to confirm the validity of the initial sample. In the event an observed increase is confirmed, the following procedures are generally followed:

1. Determine the source of any confirmed increase, which may include, but not be limited to, natural phenomena, sampling or analytical errors, or an off-site source;

2. The IEPA will be notified in writing no later than 180 days after the original sampling event of any confirmed increase. Within this notification, a demonstration will be made, if possible, that the increase is a result of a source other than the Facility, providing rationale used in such a determination; and

3. If an alternate source demonstration cannot be made or is denied by the IEPA, assessment monitoring will be proposed.

In the event that there is a confirmed increase in the concentration of any constituent in any monitoring well, and a demonstration that the confirmed increase is not caused by the landfill is not made, the necessary steps will be implemented immediately. These steps may include the following:

1. Assessment monitoring as outlined in 35 Ill. Admin. Code, Section 811.319(b);

2. Assessment of potential groundwater impact as outlined in 35 Ill. Admin. Code, Section 811.319(c); and

3. Remedial action as outlined in 35 Ill. Admin. Code, Section 811.319(d).

A remedy that will protect human health and the environment will be selected in accordance with 35 Ill. Admin. Code, Section 811.325. The corrective action, if appropriate, will be implemented and completed in accordance with the requirements of 35 Ill. Admin. Code, Section 811.326.

Leachate Monitoring

The existing facility has a leachate monitoring network as illustrated on Drawing No. D5. There will ultimately be a total of 7 new leachate monitoring points for the expansion area; one corresponding to each sump location as illustrated in Drawing No. 10. Leachate will be sampled in accordance with 35 Ill. Admin. Code 811.309(g), which currently requires semi-annual monitoring with each leachate monitoring point being sampled at least once every two years. Sampling will be conducted as long as the leachate collection system is in operation (a minimum of 30 years after closure of the facility), unless a reduced post closure sampling period is found to sufficiently protect the public health and the environment. All test results will be submitted to the IEPA. At a minimum, leachate will be analyzed for the same list of parameters as the groundwater monitoring wells. The sampling procedure that will be followed to collect leachate samples is provided in **Appendix Q**.

Landfill Gas Monitoring

Subsurface Monitoring

Subsurface landfill gas monitoring at the Site 2 North Expansion is proposed to be conducted in accordance with the requirements of 35 Ill. Admin. Code Section 811.310. The proposed landfill gas probe network will be utilized to verify that the landfill gas collection and containment systems are functioning as designed. The proposed landfill gas monitoring network is illustrated on **Drawing No. D14**. A schematic of a typical landfill gas probe is illustrated in **Diagram 2.8-1**. Landfill gas probes will be inspected at the time of monitoring events for structural integrity and proper operations.

Perimeter landfill gas monitoring probes are proposed to be constructed (see **Drawing No. D20**) of 1-inch diameter Schedule 40, or equivalent material which will not react with or be corroded by landfill gas. The probes will be equipped with valve/hose pressure

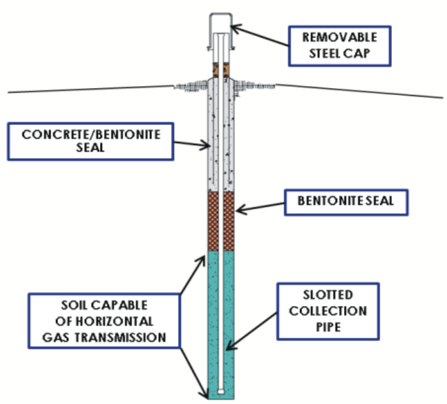

Diagram 2.8-1 Schematic of a typical gas monitoring probe

fitting(s), etc. as necessary to measure pressure and allow collection of a representative sample of gas within the probes.

The monitoring zone for these probes will be in accordance with 811.310. Pipe joints and fittings will be maintained in air-tight condition, and the probe will be installed with a bentonite seal at the surface to minimize leakage. The design and construction of the landfill gas monitoring system will not interfere with the operations of the liner or leachate collection system, or delay the construction of the final cover system.

Subsurface landfill gas monitoring devices will be sampled on a periodic basis in accordance with 811.310(c). At a minimum, below ground monitoring points will be screened for methane, pressure, nitrogen, oxygen, and carbon dioxide as required by the

IEPA. Monitoring will be adjusted as necessary to comply with the federal, state, and local regulations to ensure proper operation procedures.

Surface Emission Monitoring (SEM) and Ambient Air Monitoring

As discussed within Section 2.3 of this Application, in addition to subsurface landfill gas monitoring, ambient air monitoring will be conducted around the perimeter of the unit and in on-site buildings to verify that the landfill gas collection and containment systems are functioning as designed. At least three ambient air monitoring locations will be chosen, and samples must be taken no higher than 1 inch above the ground and 100 feet downwind from the edge of the waste boundary or at the property boundary, whichever is closer to the waste boundary. All buildings within the facility will be monitored for methane by utilizing continuous detection devices located at likely points where methane might enter each building. Ambient air monitoring locations at the site will be monitored in conformance with the requirements of the prevailing regulations which require sampling on a monthly basis for the entire operating period and for a minimum of five years after closure. The sampling frequency may be reduced to a quarterly frequency after five years of closure upon approval by the IEPA.

Surface emissions monitoring (SEM) will be performed in accordance with 40 CFR 60.755 (c) and (d); 40 CFR 60, Appendix A, Method 21; and Title 35 IAC 220.240(c). A flame ionization detector will be used to monitor the landfill surface along a site-specific traverse pattern, and at areas suspected of exceeding 500 ppm methane, including signs of gas bubbles, odors, stressed piping, etc. SEM events will be performed on a quarterly basis for the entire landfill. Prior to each monitoring event, background will be established as outlined in 40 CFR 60.755. The existing SEM Plan has been updated to include the Site 2 North Expansion Area within **Appendix L**.

In the event of a methane exceedance of 500 ppm above background, the following actions will be taken in accordance with 35 IAC 220.240(c)(4).

1. The location of each monitored exceedance will be marked and the location recorded.

2. Cover maintenance or adjustments to the vacuum of the adjacent wells to increase the gas collection in the vicinity of each exceedance shall be made and the location will be re-monitored within 10 calendar days after detecting the exceedance.

3. If the re-monitoring of the location shows a second exceedance, additional corrective action will be taken, and the location will be monitored again within 10 days after the second exceedance. If the re-monitoring shows a third exceedance for the same location, the action specified in number 5 below will be taken.

4. If re-monitoring of the location does not show an exceedance, as specified in numbers 2 or 3 above, the location shall be re-monitored 1 month from the initial exceedance. If the 1-month re-monitoring shows a concentration less than 500 ppm above background, no further monitoring of that location is required until the next quarterly monitoring period. If the 1-month re-

monitoring shows an exceedance, the actions specified in numbers 3 above or 5 below, as appropriate, will be taken.

5. For any location where there are three monitored exceedances within a quarterly period, a new well or other collection device will be installed within 120 calendar days after the initial exceedance. An alternate remedy to the exceedance, such as upgrading the blower, header pipes, or control device, and a corresponding timeline for installation may be submitted to the IEPA for approval.

Surface Water Monitoring

A Stormwater Management Plan for the Site 2 North Expansion has been designed to efficiently collect, route, and detain stormwater runoff from the Facility in an environmentally sound manner as described in greater detail within Section 2.4 of this Application. Environmental monitoring of surface water will occur in accordance with NPDES permits which will be modified for the proposed expansion as development progresses. Surface water monitoring and analysis will be performed per the site-specific Stormwater Pollution Prevention Plan and NPDES Permits.

Conclusions

The potential for the Site 2 North Expansion to impact the environment has been evaluated. In addition to the results of the GIA which demonstrate that the facility will not have an adverse impact on the groundwater quality, a comprehensive groundwater monitoring program has been designed for the Site 2 North Expansion. Additionally, Facility operations will include leachate monitoring, subsurface landfill gas monitoring, ambient air monitoring, and surface water monitoring. The Environmental Monitoring Plan at the Facility will serve as an additional safeguard to:

- Monitor potential sources of environmental impact at the facility;
- Verify that the facility design and construction are properly functioning to protect the public health, safety and welfare; and
- Provide an early warning system in the unlikely event of a leachate or landfill gas release.

Monitoring will follow strict quality control, quality assurance, and chain of custody procedures.

SECTION 2.9

Closure and Post-Closure Care Plan

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2.9 CLOSURE AND POST-CLOSURE CARE PLAN

Introduction

A closure and post-closure care plan has been prepared for the Site 2 North Expansion of the Zion Landfill in accordance with the applicable requirements of 35 Ill. Admin. Code Parts 811 and 812. The proposed final landform (refer to **Drawing No. D11**) shows the configuration of the facility after closure of all cells, including the final topography of all constructed areas and the location of all facility-related structures that will remain as permanent features after closure.

The closure and post-closure care plan details the steps necessary for the proper closure of the landfill in the event of an unplanned, premature closure of the facility as well as under the planned, routine closure of the facility. Schedules are provided for both of these scenarios. In addition, the steps necessary to care for the landfill during the post-closure period are described. Cost estimates are presented for closure and post-closure activities, and financial assurance mechanisms (to ensure that funding is available to complete those activities) are described.

Please see **Appendix S** for the Closure and Post-Closure Care Plan.