Phase 5 Landfill Lateral Expansion Application

Pickles Butte Sanitary Landfill

Canyon County, Idaho

Tetra Tech Project# 114-571040-2024

October 7, 2024

PRESENTED TO

Canyon County Solid Waste
15500 Missouri Avenue
Nampa, Idaho 83686

PRESENTED BY

Tetra Tech
3380 Americana Terrace,
Suite 201
Boise, Idaho 83706

P +1-406-489-2826
tetratech.com

Prepared By:

Ron Phillips 10/07/24

Ron Phillips, P.G. Date:
Project Scientist

Richard Salas 10/07/24

Richard Salas, PE. Date:
Civil/Environmental Engineer

Maureen S. McGraw 10/07/24

Maureen McGraw, PhD, PE Date:
Sr. Hydrologist/Civil Engineer

PROFESSIONAL ENGINEER
LICENSED
16021
STATE OF IDAHO
MAUREEN A. McGRAW

Oct 7, 2024

TABLE OF CONTENTS

TABLES

APPENDICES

ACRONYMS/ABBREVIATIONS

Acronyms/Abbreviations	Definition
AMSL	Above mean sea level
bgs	Below ground surface
CCSW	Canyon County Solid Waste
CFR	Code of Federal Regulations
CMP	Corrugated Metal Pipes
CN	Curve Number
DBS&A	Daniel B. Stephens & Associates
DEQ	Idaho Department of Environmental Quality
EPA	U.S. Environmental Protection Agency
GCCS	Gas Collection and Control System
HELP	Hydrologic Evaluation of Landfill Performance (model)
IDWR	Idaho Department of Water Resources
ISWFA	Idaho Solid Waste Facilities Act
LFG	Landfill Gas
MPH	Miles Per Hour
MSW	Municipal Solid Waste
MSWLF	Municipal Solid Waste Landfill
NOAA	National Oceanic and Atmospheric Administration
NMOC	Non-Methane Organic Compounds
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resource Conservation Service
O&M	Operations and Maintenance
OHV	Off-highway Vehicle
PBSL	Pickles Butte Sanitary Landfill
scfm	Standard Cubic Feet per Minute
SCS	Soil Conservation Service
SWDH	Southwest District Health
USACE	United States Army Corps of Engineers
U.S. BLM	United States Bureau of Land Management
USCS	Unified Soil Classification System
USGS	United States Geological Survey

1.0 INTRODUCTION

The Pickles Butte Sanitary Landfill (PBSL) is located in rural Canyon County, Idaho, approximately 6 miles south of the City of Nampa and serves Canyon and Owyhee Counties. **Figure 1** (**Appendix A)** shows location of the Landfill in relation to Nampa and Lake Lowell. The Landfill is located within approximately 1300 acres of county-owned property covering parts of Sections 20, 21, 28, and 29 of Township 2 North, Range 3 West of the Boise Meridian. The current landfill footprint is in the east-central portion of the county-owned land. **Figure 1** also shows the extent of the county-owned land and the location of the landfill within that area. Much of the adjacent land is used for farming, dairy operations, and/or the Jubilee Park off highway vehicle (OHV) area, except for areas where the topography is unsuitable for these uses.

The Idaho Department of Environmental Quality (DEQ) approved the original design and operating plan for PBSL as a Municipal Solid Waste Landfills (MSWLF) in June 1973, and reconfirmed approval in May 1975 (Holladay, 1994). Southwest District Health (SWDH) approved the landfill in December 1979 (Holladay 1994). The landfill initially began accepting municipal solid waste (MSW) in April 1983. With the implementation of Subtitle D, the County obtained site certification for the landfill from the DEQ in August 1993. The DEQ subsequently approved a Hydrogeologic Characterization, Ground Water Monitoring Plan and Facility Design Report prepared by Holladay Engineering Company (Holladay, 1994). The approval included exemptions from the requirements for a liner/leachate collection system and groundwater monitoring. This technical decision was based on the depth to groundwater, characteristics of native soils, and the arid climate at the facility. The PBSL Operations and Maintenance (O&M) manual for the facility was recertified by Southwest District Health (SWDH) on July 19, 2024 and the plan is valid through July 2027.

Waste disposed of at the PBSL consists primarily of residential municipal solid waste, construction and demolition (C&D) materials, biosolids, and other nonhazardous waste. The landfill work to divert waste for recycling or reuse, including white goods, metal, tires, clean wood waste, and green waste.

1.1 PURPOSE

This document has been prepared to support the lateral expansion of the PBSL. Based on the aerial survey conducted on September 30, 2023, there was approximately 7.5 years of air space remaining in Phase 3 and 4 years of air space remaining in Phase 4. Phase 3 and Phase 4 are part of the approved waste footprint of 116.7 acres. However, during the five-year period between October 2018 and October 2023, the waste acceptance rate has increased an average of 4.3%, which reflects the population growth that has occurred in Canyon and Owyhee Counties, as well as the greater Treasure Valley. Therefore, to continue to provide MSW disposal services it is necessary to expand the landfill capacity. The requested lateral expansion of the landfill is designated Phase 5.

Canyon County worked with Holladay Engineering Company (Holladay) to expand the characterization of the area surrounding the landfill beginning in 1992 as part of the investigation described in their 1994 report. Seven wells were installed that were designated PB-2 through PB-8. The designation PB-1 was applied to an existing domestic well located adjacent to the shop building at the Landfill. Holladay installed monitoring wells PB-9 and PB-10 in 1995. Daniel B. Stephens & Associates (DBS&A) installed wells PB-10 through PB-15 in 2011 as part of their investigation for a future expansion. The County also commissioned significant hydrogeologic investigations between 2010 and 2014 for the future expansion of the landfill. DBS&A conducted this work. The County commissioned additional borings, a geotechnical investigation, and a seismic investigation in 2021 to address additional data gaps identified. The data from all these investigations provide the foundation for the expansion design application.

There have been several different conceptual expansion designs, which is reflected in some of the data gaps analysis reports conducted for the expansion including the geotechnical evaluation and seismic evaluation. The various conceptual expansion designs do not reflect the design as submitted in this application, nor does it alter the value of the data collected during previous investigations. This document serves as the application for a lateral expansion of the PBSL using an arid design that is in compliance with the Idaho Solid Waste Facilities Act (ISWFA) §39-7409 and §39-7410.

This application is organized into the following Sections:

- **Section 1** of this report presents an introduction and regulatory requirements for the lateral expansion application under the arid design requirements.
- **Section 2** provides background information on the site characteristics of the landfill, including the climate, geology, soils, groundwater, geotechnical stability and seismic conditions.
- **Section 3** provides the lateral expansion design, including a hydrologic and hydraulic analysis of the final conditions.
- **Section 4** summarizes the supporting documentation provided electronically with this application. The supporting documents are an essential part of the application and provide the background studies and modeling conducted as the landfill prepared for a lateral expansion. They are referenced in this application but are not included as appendices in the application.
- **Section 5** provides the references for material used in the development of this document.

Appendix A provides figures. **Appendix B** provides copies of the Site Certification Approval. **Appendix C** provides data on site soils and site-specific laboratory data. **Appendix D** provides geologic cross section to show the geology as well as the distance to the water bearing zone. **Appendix E** contains copies of well and boring logs from the site. **Appendix F** contains a geotechnical report for the site. **Appendix G** contains a copy of the seismic investigation report conducted for the site. **Appendix H** contains the lateral expansion design drawings. **Appendix I** contains a copy of the Hydrology and Hydraulic calculations.

1.2 ARID DESIGN REGULATORY COMPLIANCE

The design of a lateral expansion for a MSW landfill is regulated by 40 CFR §258.40 Design criteria for MSWLFs on the Federal level and by the ISWFA §39-7409 on the State of Idaho. **Table 1** provides information on where the required information in located in the application.

Table 1: Summary of Regulatory Requirements and Location in Application

Regulation	Title/Requirement	Location in Application and Supporting Documents
ISWFA §39-7407, §39-7408	Location Restrictions – Site Certification	The submittal and approval of the site certification was previously conducted by PBSL. *Section 1.3* discuss the Site Certification approval and copies of the approval letters are provided in Appendix B.
ISWFA §39-7409	Standards for Design	
ISWFA §39-7409(1)	Applicability	The PBSL is subject to the MSWLF design standards as an existing landfill, and under this regulation for a lateral expansion.
ISWFA §39-7409(2)	Liner designs	The regulations allow for a (a) Composite liner, (b) Alternative liner design, or (c) Arid design. This application is for use of an arid design.
ISWFA §39-7409(2)(c)	This design will apply to locations with less than twenty-five (25) inches of precipitation annually, net evaporative losses greater than thirty (30) inches annually,	*Section 2.1* discuss the local climate and presents site-specific data demonstrating these conditions have been met.

Regulation	Title/Requirement	Location in Application and Supporting Documents
	holding capacity in native soils greater than annual absorbance;	The holding capacity of soil is the equivalent to laboratory measurement known as the field capacity, which represent how much water the soil can hold against gravity. *Section 2.6* provides information from laboratory testing that demonstrates the site has sufficient holding capacity.
ISWFA §39-7409(2)(c)(i)	solid waste is deposited no less than fifty (50) feet above the seasonal high level of ground water in the uppermost aquifer	The distance between the waste and the upper most water bearing zone is greater than 300 feet. *Section 2.4* discusses the site stratigraphy and *Section 2.8* discusses the vertical distance to water bearing zone. **Appendix D** contains cross-sections that show the proposed bottom of waste, potentiometric surface and top of water bearing zone.
ISWFA §39-7409(2)(c)(ii) and (iii)	the geologic formation beneath the site and above the uppermost aquifer must have capillary capacities greater than the projected maximum volume of leachate generated during the active life of the MSWLF unit; "no potential for migration" is demonstrated when the geologic formation beneath the site and above the uppermost aquifer has sufficient hydrogeological characteristics and holding capacity adequate to contain all hazardous constituents generated during the active life, closure and post-closure care periods.	The Hydrogeologic Characterization Report prepared by Daniel B Stephens & Associates in 2014 (and included as supplemental material for this application) describes work conducted to characterize the geology in and around the PBSL to aid in evaluating the potential for landfill leachate to impact groundwater. Consistent with earlier work conducted by Holliday (1994) and additional characterization work conducted by Tetra Tech (**Appendices F and G**) a very low permeability siltstone and claystone is present beneath the entire site and serves as a confining unit. As discussed in the report, the siltstone has a low hydraulic conductivity (average of 3.99×10^{-8} cm/s) and overlies the first occurrence of groundwater (average of 297 feet). The report estimates that migration of leachate to first groundwater would take thousands of years. The report provides extensive discussions on the geology, hydrogeology, and infiltration modeling with HELP and HYDRUS. There are two volumes to the report. Volume 2 contains a table upfront of all of the laboratory testing conducted including hydraulic conductivity, moisture content, particle size, and Atterberg limits. The extensive laboratory testing of soil properties was used to document and support the analysis conducted in Volume 1 of the report. The geologic conditions are also discussed in *Section 2.2* to *Section 2.6*. Geotechnical and Seismic evaluations are provided in *Section 2.9 and 2.10* as well as **Appendices F** and **G**.
ISWFA §39-7409(3)	Point of compliance	The point of compliance is the site certification boundary shown in **Figure 3** (**Appendix A**). The figure also shows the additional property owned by Canyon County beyond the site certification boundary. Groundwater flows to the southwest, so monitoring wells MW-11 through MW-15 on the south side of the landfill that are downgradient of the currently approved landfill footprint, as well as the proposed lateral expansion ensure that there are no impacts to groundwater upgradient of the compliance boundary.
ISWFA §39-7409(4)	Leachate discharge	The PBSL does not have a leachate collection system, and therefore does not have any discharges that fall under the Clean Water act (40 CFR 122)
ISWFA §39-7410	Ground water monitoring design	Although the PBSL currently operates under an arid exemption, the landfill voluntarily conducts bi-annual groundwater, monitoring and submits the reports to DEQ for review. Details

Regulation	Title/Requirement	Location in Application and Supporting Documents
		on groundwater at the site, including a summary of groundwater monitoring, are provided in *Section 2.7. Section 3.4* provides details on updates to the monitoring program associated with the proposed expansion.

1.3 SITE CERTIFICATION

There have been three different Site Certifications for the PBSL. All three Site Certifications meet the requirements for the lateral expansion of the landfill under §39-7407, Idaho Code. Copies of the approval letters are provided in **Appendix B**.

1.3.1 Original

On June 24, 1993, Holiday Engineering Company filed a Site Certification for the PBSL on behalf of Canyon County to comply with the new state requirements for an MSWLF pursuant to §39-7407, Idaho Code. At the time it had been accepting waste for approximately 10 years, the Site Certification Boundary encompassed approximately 260 acres although only 116 acres was approved to accept waste. The approval was received from the DEQ on August 9, 1993.

1.3.2 DBS&A

On June 17, 2010, Daniel B Stephens and Associates filed a Site Certification Application on behalf of Canyon County for the PBSL in preparation for an expansion design (DBS&A, 2010). The Site Certification boundary was expanded to include additional land purchased by the County and expand the area that could be included in the expansion design application. The Site Certification increased the acreage from approximately 260 acres to 490 acres. The approval was received from the DEQ on August 2, 2010.

1.3.3 Current

On November 19, 2020, Tetra Tech filed a Site Certification Application on behalf of Canyon County for the PBSL in preparation for an expansion design. The Site Certification was expanded to include additional land purchased by the County and included areas that were part of a conceptual expansion design developed in 2017 to vet the overall scope and configuration of the expansion with the County, and to be able to identify potential data gaps and design constraints. The Site Certification increased the acreage from approximately 490 acres to 600 acres. The approval was received from the DEQ on February 26, 2021.

2.0 SITE CHARACTERISTICS

Information from site investigations have been used to characterize the soil and geologic conditions at the Landfill. The investigations include those conducted for the original site certification and design, work conducted to develop the groundwater monitoring program, shallow investigations for cover material (1994, 2016, 2022), a geotechnical investigation to support landfill expansion, a seismic study, and groundwater monitoring conducted since 2017.

Additional information on the geologic setting of southeastern Canyon County has been garnered from reports and maps published through the Idaho Geological Survey, Idaho Department of Water Resources (IDWR) and the United States Bureau of Land Management (U.S. BLM).

2.1 CLIMATE

The PBSL is located in Nampa, which is a high desert that is bordered to north by the Rocky Mountain front range and to the south by the Owyhee mountains. The average precipitation in the region is 11.6 inches per year (City of Nampa, 2024). The winters are typically cooler and wetter. January is generally the coolest month. The summers are hot and dry. The warmest month is generally July. **Figure 2** (**Appendix A**) shows the wind rose for Nampa, Idaho. The data indicates that the prevailing wind direction is from the northwest, but that wind also occurs from the west and the southeast.

The PBSL installed a weather station in 2017 to determine site specific conditions. **Table 2** shows the annual data from 2018 to the present. The site-specific data indicates that the location of the landfill has less precipitation than the City of Nampa. Both the City of Nampa data and the site-specific data indicate that the site meets the requirements for an arid design in §39-7409(2)(c), Idaho code that requires an arid design to be sited in a location that has less than twenty-five (25) inches of precipitation annually.

Table 2: Summary of Annual PBSL Weather Station Data

Year	Annual Precipitation (inches)	Minimum Annual Temperature (°F)	Average Annual Temperature (°F)	Maximum Annual Temperature (°F)	Average Annual Wind Speed (mph)	Maximum Annual Wind Speed (mph)
2018	7.3	9.3	54.6	107.9	7.9	39.5
2019	8.2	14.3	53.0	99.1	7.9	41.9
2020	10.6	17.2	54.5	101.5	7.7	41.5
2021	5.3	16.0	55.6	106.7	8.1	38.8
2022	5.9	4.5	53.1	106.5	7.8	41.8
2023	9.0	8.3	53.8	103.7	8.2	40.4
2024 (thru 9/9)	6.1	-1.2	58.2	106.9	8.1	35.8

Table 2 also provides the minimum, average, and maximum temperatures, as well as the average and maximum windspeeds. The site weather station does not measure pan evaporation. Therefore, to evaluate the evaporation at the site, the Evapotranspiration and Consumptive Irrigation Water Requirements for Idaho available on the IDWR website (IDWR, 2024) was used to determine the potential evapotranspiration (ET). Selecting Nampa, ID and range grasses – long season to represent the vegetation at the site indicates that the annual potential ET was 728 ± 53 mm (28.7 ± 2.1 inches). Based on the lower precipitation at the site, maximum wind speeds, elevated summer temperatures, and the need to apply water for dust control, the actual net ET is likely on the higher range of the standard deviation which infers that the site meets the requirements for an arid design in §39-7409(2)(c), Idaho code that requires an arid design to be sited in a location that has net evaporative losses greater than thirty (30) inches annually.

2.2 GEOLOGY

The PBSL is located within a geologic structure known as the Western Snake River Plain. The Snake River Plain is a broad, arc shaped depression extending across southern Idaho. While the eastern portion of the Snake River Plain is considered to be a function of the movement of the North American Plate relative to an underlying heat source ("the Yellowstone Hotspot"), the Western Snake River Plain was formed by different geologic processes. The Western Snake River Plain is generally regarded as being a rift zone, where the earth's crust was pulled apart by tensional forces. In this case, the forces were pulling the crust to the northeast and southwest, resulting in a thinning of the crust in the middle of the northwest/southeast trending rift zone. Fault zones developed on the borders of the rift zone, perpendicular to the direction of the tensional forces. Evidence suggests this process for the Western Snake River Plain began between approximately 12 million (U.S. BLM n.d.) and 17 million years ago (Mabey, 1982), during the Miocene epoch.

The fault zone on the northeast side of the Western Snake River Plain is called the Boise Front Fault, sometimes referred to as the Boise Foothills Fault zone. The system of faults on the southwest side of the basin is often called the Owyhee Mountains Fault zone. Both are recognized as normal faults, though strike-slip movement has also been postulated for the Owyhee Mountains Fault Zone (Mayo et. al., 1984). Normal faults are those in which the hanging wall moves downward relative to the footwall. In this way, the interior of the basin (a graben or graben-like structure) decreased in elevation compared to the Boise Front on the northeast and the Owyhee Range on the southwest. The normal faults on either side of the Western Snake River Plain have an average orientation of approximately North 50° West.

The total relative vertical movement of the graben relative to the ranges on either side is not known. The Western Snake River Plain is a current topographical basin, but erosion on the ranges and filling of the basin with sediments and upwelling basalt has obscured the total vertical movement. In addition to vertical movement along the fault lines, subsidence or downwarping in the interior of the basin has likely occurred because of the weight of the sediments and volcanic rocks that have filled the depression (Swirydczuk, et. al. 1982). Malde (1959) suggests that there may have been 5000 feet of vertical displacement along the faults and an additional 4000 feet of subsidence. This is consistent with the findings of deep wells referenced by Mabey (1982) where lacustrine sediments and basalt flows accounted for more than 6000 feet of material above granite bedrock. The lithology from a deep well (Anshutz Federal No.1) in the Western Snake River Plain showed that there was 11,150 feet of sediment and basalt above granite (Maley, 1987). The granite could be an extension of the late-Cretaceous Idaho batholith. Otherwise, there is no evidence that the pre-Cenozoic rocks on the borders of the plain have been downfaulted under the plain (Digital Atlas of Idaho, 2023)

Basalt flows in the Western Snake River Plain began approximately 11 million years ago (Shervais, et.al., 2002). The basalt eruptions appear to have been a direct result of the tectonic forces that created the basin.

Most of the Western Snake River Plain basin was eventually covered by ancient Lake Idaho, possibly because of basalt flows forming a dam at the western end. Kimmell (1982) theorizes that the basin was occupied by two large lakes in succession. The sizes of the lakes may have been controlled by tectonics and changes in climate. The Western Snake River Plain basin has been largely filled with sedimentary materials from this depositional environment. Lacustrine sedimentation appears to have occurred largely between 8.5 and 2 million years ago, during the Pliocene and upper Miocene epochs (Wood, 1994). These sediments become more lithified at depth because of the weight of overlying materials. The type and nature of the sediments are important factors in preventing migration of landfill leachate through the sediments beneath the current and future landfill cells.

Later sedimentation in the Western Snake River Plain basin included fluvial and possibly eolian deposits. These are generally coarser and less indurated than the underlying lacustrine sediments. Basalt eruptions occasionally intercalate with these sediments (Digital Atlas of Idaho, 2023).

The materials filling the Western Snake River Plain basin are part of the Idaho Group, a designation first provided by Malde and Powers (1962). They divided the Idaho Group into seven formations and provided a lithologic sequence. These are shown in **Table 3**, along with ages provided by Savage (1968). The Idaho group is underlain by volcanics (rhyolite and basalt) of the Idavada Group and covered by the Snake River Group; both contacts are unconformable (Ruez, 2009; Wood and Anderson, 1981).

Wood and Anderson (1981) add a division to the Idaho Group, designating the Chalk Hills and older formations as the Lower Idaho Group and the Glenns Ferry Formation and younger materials as the Upper Idaho Group. This latter group comprises the near surface and subsurface geologic materials that have been encountered during the investigations at PBSL.

Table 3: Idaho Group Formations

Formation (Young to Old)	Rock and/or Soil Types	Geologic Age
Black Mesa Gravel	Sand and Gravel	Middle Pleistocene
Bruneau Formation	Basalt, Sand, Gravel	Middle Pleistocene
Tuana Gravel	Sandy Gravel	Lower Pleistocene
Glenns Ferry Formation	Sand, Silt, Clay, Siltstone, Claystone, some Sandstone.	Upper Pliocene to Lower Pleistocene
Chalk Hills Formation	Silt and Sand; some Ash content	Middle Pliocene
Banbury Basalt	Basalt with Tuff Beds	Middle Pliocene
Poison Creek Formation	Ash and Tuff with some Sands and Gravel	Lower Pliocene

2.3 TOPOGRAPHY

Pickles Butte and Deadhorse Canyon are the two prominent topographic features near the landfill. Pickles Butte is at the eastern end of a 1.25 mile long ridge that trends slightly north of west. The elevation at the top of the ridge ranges from 2996 to 3083 feet above mean seal level (AMSL). Much of the north face of this ridge slopes steeply into Deadhorse Canyon at a slope of approximately 30% (18 degrees). The slope decreases toward the base of Deadhorse Canyon. The northern base of the ridge essentially forms the southern extent of the current landfill footprint and the expansion area.

Deadhorse Canyon trends toward the northwest on the north side of Pickles Butte. The canyon was historically 0.5 to 0.75 miles wide as shown on topographic maps, through road construction and the landfill have altered the natural topography in the eastern and northern parts of it. Gently rolling landscape is present east and northeast of the landfill and Deadhorse Canyon. The elevation of the eastern and northern rim of the historic Deadhorse Canyon ranged from approximately 2800 to 2900 feet AMSL. The slope along the east wall of the historic canyon was approximately 30 to 35%, then decreasing to approximately 10% in the lower part of the canyon (USGS, 1958). The steep natural slope of the east wall is visible now only in a location south of Deer Flat Road where this road descends into the canyon.

The canyon funnels into a narrow outlet, approximately 200 feet wide at an approximate elevation of 2590 feet AMSL. This is near the current western boundary of Canyon County owned property. From here, the canyon opens up into a gently sloping plain, approximately 0.5 mile wide, that is used for agriculture.

2.4 STRATIGRAPHY

Stratigraphic information specific to the PBSL has been collected during several investigations beginning in 1992. These include monitoring well installations in 1992, 1995, 2011, and 2020, site specific geologic mapping by Holladay, regional geologic mapping, geotechnical drilling programs in 1998 and 2021, and observations of surface geologic materials and outcrops.

Based on the information from the site investigations, a stratigraphic sequence of geologic materials near to and underlying the landfill has been developed. **Table 4** presents the basic information for the stratigraphic sequence of the generalized soil and rock units present within the investigated areas at the landfill.

Table 4: Site Lithologic Sequence

Formation	Rock and/or Soil Types	Thickness (feet)
Unclassified (Quaternary Deposits)	Sand, non-lithified	Up to 25
Bruneau	Basalt; Sandy Silt to Silty Sand	Up to 50
Tuana Gravel	Sandy Gravel	Up to 50
Glenns Ferry	Sand, silt, clay, siltstone, claystone	Possibly >2000 feet total, 893 penetrated by monitoring well PB-14

The Bruneau Formation, Tuana Gravel, and Glenns Ferry Formation are considered the Upper Idaho Group. The formations of the Lower Idaho Group (including the Poison Creek Formation, Banbury Basalt, and the Chalk Hills Formation) lie at depths that are beyond the depths explored at the landfill. Descriptions of the geologic materials in the study area are provided below.

Six geologic cross sections across the landfill area were developed using information from the drilling investigations. The locations of the cross-section lines, identified as A through F, are shown on **Sheet X-100** (**Appendix D**). The cross sections are included as **Sheets X-101** through **Sheet X-110** (**Appendix D**).

2.4.1 Unclassified Sediments

The youngest sediments in the study area are likely the light tan to buff-colored sands found in the northern and central parts of the investigated study area. The sand is described by the National Resource Conservation Service (NRCS) as being derived from alluvium, eolian, or lacustrine sediments (NRCS, 2023). The sand appears to be most prominent on the eastern/northeastern rim of Deadhorse Canyon, generally east and north of the original landfill cell. Holladay (1994) designated these as “minor sand dunes” that are the most recent deposit in the area. Their mapping included a low ridge in the central part of the study area, west of PB-1 and south of PB-8, in the minor sand dunes classification. This would indicate deposition after most, or all of the erosion had occurred to create Deadhorse Canyon. The sand north and east of the landfill is outside of the expansion area, and the ridge of sand in Deadhorse Canyon is above the base of the future landfill cells. These younger sands, therefore, have not been further investigated or evaluated for the current study.

2.4.2 Bruneau Formation

Sediments of the Bruneau Formation include fine-grained sandy silt to silty fine-grained sand. Much of the native material in the area east of Phase 1 of the landfill, in the locations of monitoring wells PB-6, PB-7, PB-9 and PB-10, is typical of the Bruneau Formation. The silt and sand layers at these locations varies in thickness with up to 25 feet above gravel and sand deposits of the Tuana Formation (described below). In other places, little of the Bruneau Formation sand/silt is present above the Tuana Gravel. This silt and sand are generally tan-white colored.

The sediments of the Bruneau Formation are mostly unconsolidated. Test pits excavated in April and July 2016 to evaluate soil for cover material encountered loose to slightly lithified soil, except when calcium carbonate cementation was present. The amount and strength of cementation varied laterally. The thickness of the Bruneau Formation at most of the test pit locations exceeded the depth of the test pit; however, gravel belonging to the Tuana Formation was found in the five test pits along the eastern site boundary. The depth to gravel ranged from 1.4 feet in the southernmost test pit (T31) to 12.5 feet in test pit T20, located at the northeast corner of the property.

Observations from the test pits east of the eastern rim of Deadhorse Canyon indicate the soil in the Bruneau Formation grades coarser with depth. The majority of the soil found near the surface in the 2016 test pits consisted of silt with varying levels of clay. Sandy silt to silty sand was often found at depth in the test pits.

The coarsening with depth was further viewed in test pits collected on the western slope of the eastern canyon wall, north of the existing landfill. In many cases, the near surface soil was sand or silty sand, with a coarsening seen with depth. The surface elevation of these test pits was several feet lower than those east of the rim of the canyon, and thus the soil seen in them is representative of soil lower in the geologic profile.

A relatively thin layer of basalt belonging to the Bruneau Formation is present on the top of Pickles Butte and on Canyon County property in areas south and southwest of Pickles Butte. To date, the only boring that has intercepted the basalt is PB-13 which was drilled on Pickles Butte southwest of the active landfill. The upper 20 feet of this boring was in the Bruneau Formation basalt. The surface of the basalt is generally covered by loess except on the top Pickles Butte ridge, at the edge of a linear ledge southeast of the ridge, and in a small canyon further to the southeast. The top of basalt in these latter two features ranges from approximately 2900 to 3000 feet AMSL. The lack of basalt in similar depths at monitoring well locations PB-13 and PB-14 suggest that the area to the southwest may be on the downthrown side of a normal fault on the southwest side of the ridge, trending toward the northwest. This is consistent with information from various geologic maps. The geographic extent of the basalt does not coincide with the proposed landfill expansion area, and thus further investigation of the rock including hydrogeologic characteristics has not been conducted.

2.4.3 Tuana Gravel

The Tuana Gravel underlies the Bruneau Formation, though the type of contact between them is not clear. Wood and Anderson (1981) suggest that it is an unconformable contact. This has not been confirmed by the limited observations made during investigations at the landfill. The best visual exposure of the Tuana Gravel at the site is in a borrow pit located east of the southern extent of Phase 1 of the landfill (northeast of the shop and equipment staging area). Landfill personnel use this as a source of gravel for roadbuilding or other construction needs. The Tuana Gravel is also exposed in some of the road cuts near Pickles Butte, and in a Jubilee Park parking area at the west end of Missouri Avenue. Holladay (1994) also describe a 30-foot-thick exposure along the eastern rim of Deadhorse Canyon. The filling of the canyon in this area by Phase 1 of the landfill has covered this exposure, leaving the aforementioned areas as the best exposures of Tuana Gravel on the property.

Holladay indicated that the exposure along the eastern rim of Deadhorse Canyon as being moderately well cemented by calcium carbonate. This level of cementation is not seen in the gravel pit area, nor in the road cuts and parking area near Pickles Butte. The gravel is generally subround, with some cobbles ranging in size up to approximately 6 inches, though much of the gravel is less than 3 inches in diameter. Interbedded lenses of sand and/or silt intervals may locally comprise substantial portions of the Tuana Gravel profile.

Tuana Gravel was encountered in the borings east of the landfill (PB-5, PB-6, PB-7, PB-9, and PB-10), in the three monitoring wells on the Pickles Butte ridge (PB-13, PB-14 and PB-15), and at monitoring wells PB-4 and PB-16 which are located in the south-central part of the landfill area. As noted above, the top of the Tuana Gravel was encountered in several test pits east of Phase 1 of the landfill. A comparison of the elevation of the top of the gravel at PB-13, the wells east of the landfill, and the 2016 test pits, shows that the upper surface of the gravel slopes to the northeast at 2.6 degrees. This correlates with observations made by Holladay where a slope of 3 degrees was reported.

Comparing the elevation of the bottom of the gravel shows a slightly smaller slope to the northeast of 1.7 degrees. This indicates a thinning of the layer from southwest to northeast. This agrees with the findings presented by Holladay, where they indicated the thickness of the unit ranges from less than 10 feet along parts of the northeastern rim of Deadhorse Canyon to nearly 100 feet on Pickles Butte.

More importantly, the bottom of gravel elevation, which ranges from approximately elevation 2988 to 2865 feet AMSL, is well above the base of the landfill cells in the expansion area. The Tuana Gravel therefore has no bearing on the hydrogeologic issues related to the landfill expansion.

2.4.4 Glenns Ferry Formation

Upper (younger) Glenns Ferry Formation soils are the majority of geologic materials exposed on the northern flank of Pickles Butte and in the walls of Deadhorse Canyon. This formation underlies the Tuana Gravel, where present. The Glenns Ferry Formation extends beneath the landfill beyond the total depths explored in the groundwater and geotechnical investigations conducted to date.

Malde (1972), in a study of the stratigraphy of the Western Snake River Plain, made extensive observations of the Glenns Ferry Formation. They indicate that it consists of lacustrine (lake deposited), fluvial (stream deposited) and flood plain facies that intertongue, often complexly. The lacustrine facies is the most dominant, both in terms of volume and extent. It consists mostly of massive layers of tan colored silt. The fluviatile facies is composed mainly of thick beds of pale brown-grey sand and silt. The flood plain facies is mostly thin beds of silt and clay with intermittent layers of shale and sand (Malde and Powers, 1972).

Information gathered during drilling for monitoring well installation indicates that the textural composition and physical properties of the Glenns Ferry Formation at the landfill site vary with depth. In general, the material becomes finer grained and more consolidated or indurated with increasing depth. The properties may also vary somewhat laterally, and the Holladay (1994) report mentioned that the lithification in correlating beds was seen to vary between borings.

The upper part of the Glenns Ferry Formation encountered in the borings is comprised primarily of sand and silt. DBS&A described the sand beds as ranging from poorly to well sorted, from very fine grained to coarse-grained, and having little or no consolidated structure to a well-lithified sandstone. The grain size of the upper Glenns Ferry Formation tends to decrease with depth, and in the lower depths explored the Glenns Ferry Formation consists primarily of siltstone or claystone. The change from upper portion of the Glenns Ferry Formation showing little consolidation to the more lithified sediments at depth is often abrupt, as described by DBS&A (2014a).

These lithified sediments are considered a hydraulic confining layer. This laterally extensive zone in the Glenns Ferry Formation has been found in all areas that have been explored to a sufficient depth at the landfill. The layer is usually described as a siltstone or claystone on the lithologic logs prepared by field geologists. The material is most often described as clay on lithologic logs prepared by drillers. Contained within this layer is a boundary at which the sediments below may have been deposited in an anoxic or oxygen deficient state. This condition gives them a characteristic blue green or blue grey color. This distinguishing characteristic is easily seen, so the layer is often referred to as the “blue clay,” and can be identified on boring logs and traced laterally across the entire landfill area. This anoxic layer is not limited to the areas explored at the landfill. Wood and Anderson (1981) indicate that the layer has been found as far to the west as Parma and as far to the east as Boise. The widespread presence of this layer is important for two reasons. First, its lateral continuity shows the uninterrupted nature of the middle Glenns Ferry Formation across the entire study area. This is consistent with the depositional environment proposed by Kimmel (1982) that the lower part of the Glenns Ferry Formation was formed in lacustrine setting across a large part of the Western Snake River Plain.

Secondly, the blue clay is postulated as acting as an impermeable or nearly impermeable layer limiting groundwater movement. Wood and Anderson (1981), as part of a geothermal investigation in southwestern Idaho, found significant temperature differences in wells completed above and below the blue clay indicating that it is acting as a cap above deeper warm water aquifers.

In their discussion of area groundwater conditions, Holladay (1994) compiled a table showing how the presence of blue clay can be traced throughout the general area. That table is designated as Table 1 in their report. They inspected the drillers logs for 72 wells located in the general vicinity of the landfill. The blue clay (or a lithologic feature that correlates to it) was identified on the majority of the logs. Those that did not specifically indicate blue colored clay nonetheless showed a thick sequence of clay and similar material (e.g. claystone, mudstone, siltstone) at depth in the area south of Lake Lowell. A recent search of well logs in the IDWR database found nine additional wells within approximately 1.3 miles of the landfill that have been installed since the Holladay research. Two of these (Stuart and Snell) are on property now owned by Canyon County Solid Waste. Four others (Helfrich, Lowry, Riggs, and Sevy) are downgradient of the expansion area based on the piezometric surface measured at the PBSL monitoring wells. These are summarized on **Table 5**.

Table 5: Area Domestic Wells, Post 1994

Well Owner	Address	Top of Redox Zone Elevation (AMSL)	Top of Confining Layer Elevation (AMSL)
Esther Helfrich	16666 Deer Flat	2372	2392
Chad Lowry	17626 Deer Flat	2270	2397
Lonnie Riggs	8018 Bale Lane	2402	2402
Daniel Sevy	17957 Deer Flat	2320	2355
David Snell[1]	16141 Deer Flat	2417	2587
Don Stuart[1]	16241 Deer Flat	>2264[2]	Unknown[2]

1 – Now owned by Canyon County.
2 – Only the portion of the log below 485 feet deep is available. The elevation of the top of the redox zone and confining layer cannot be determined.

Interpreting the lithology from driller's logs should be considered an approximation; in many cases there is no differentiation between silt and clay, or reliable information about the consolidation. Nonetheless, an inspection of the six1 logs for the wells shown in **Table 5** does provide useful information to show that that conditions encountered in monitoring wells at the landfill extend beneath and beyond the expansion area. The redox layer is discernible from each of the well logs. The elevation of the redox layer can be estimated by subtracting the depth to the redox layer from the approximate ground surface elevation at each well location. The elevation of the top of the claystone/siltstone can also be estimated, though there is more uncertainty because of the lack of descriptive information on the consolidation of the material.

Overall, the information shows that the fine-grained material (silt and clay) of the Glenns Ferry Formation is present at depth across the area, as is the redox boundary with a slight gradient to the northwest. The redox boundary in the western part of the landfill area ranges from an approximate elevation of 2417 feet at the Snell well to 2527 feet at monitoring well PB-13. The elevation of the redox boundary at the Daniel Sevy domestic well located approximately 1.3 miles west of the land fill is approximately 2320 feet. The redox boundary elevations at the Riggs and Helfrich wells (approximately 1 mile west-southwest and one-half mile west of the landfill) are approximately 2402 and 2372 feet respectively. Using these four points as a reference, the redox boundary slopes to the northwest with a gradient of 185 feet per mile.

As discussed above, the transition from less compacted sediments of the upper Glenns Ferry Formation to the more consolidated claystone and siltstone is not easily discernible from the driller's logs. Using a best interpretation of when the confining layer starts also shows a slope of that surface to the northwest though the gradient is slightly flatter than the redox boundary at approximately 155 feet per mile.

The northwestward slope in the top of the confining layer across the western part of the study area is consistent with the information shown on Cross Section A in **Appendix D**. This cross section is constructed roughly parallel to the apparent slope of the confining layer and shows a slope toward the northwest from the center of the landfill at PB-1 toward PB-11. In the northeastern part of the landfill, the apparent slope is toward the northeast, which is consistent the findings from the Holladay analysis.

2.5 AREA FAULTING

The USGS Quaternary Fault and Fold Database of the United States indicates that the Western Snake River Plain (WSRP) fault system is present in the general area, and a portion of an undifferentiated Quaternary-aged northeastdipping normal fault is mapped within the project boundaries. The mapped location shows it extending northwest through the proposed expansion area. It is labeled as a normal fault with an approximate slip rate of less than 0.2 mm/year. The approximate location, as indicated by USGS, is shown on Figure 1 in a Seismic Survey Report prepared by Tetra Tech in 2022. This report is included as **Appendix G**. The WSRP fault system consists of northwest-striking, northeast- and southwest-dipping normal faults. Most of these faults are described as having subdued expressions on the floor of the Snake River Plain. The USGS information indicates that detailed studies on the age of faulted deposits have not been published, but most fault traces are confined to older Quaternary deposits. The USGS thus assigns a Quaternary age to the faults until further detailed studies are conducted.

The fault locations are from various sources, mapped at scales ranging from 1:250,000-scale to 1:62,500-scale mapping. Mapping at the latter scale was conducted by Wood and Anderson (1981). The USGS information indicates that slip rates have not been described, but the weak geomorphic expression of these faults indicates very low rates of long-term slip (Personius 2003).

Two faults mapped by the sources listed above are on the northeast and southwest sides of the Pickles Butte ridge and are outside of the expansion area. Cross Section X-108 (**Appendix D**) shows the approximate location of the fault on the northeast side of the ridge. The USGS database shows one fault that is potentially present near the expansion area. It is shown with a length of approximately 3.4 miles with a strike of approximately North 37° West. The southern terminus is shown in the northern part of the active landfill area. A seismic survey was conducted in 2022 to collect more information on this fault. This is discussed in *Section 2.10* below.

2.6 SOIL AND ROCK PROPERTIES

Samples of soil and rock have been collected during three test pit investigations and during three of the drilling efforts to characterize the hydrogeologic and geotechnical properties of the materials. In some cases, the samples were collected and analyzed to use in slope stability calculations for landfill design. Other samples were specifically collected to provide data for use in Hydrologic Evaluation of Landfill Performance (HELP) and HYDRUS models. Still other samples were analyzed for various physical properties for final cover design purposes.

2.6.1 Test Pit Investigations

Holladay collected 25 soil samples from 13 test pits in 1994 to evaluate the material for a final landfill cover. The test pits were excavated in the area east of Phase 1 of the landfill. The sample depths ranged from 2 to 10.5 feet deep. Holladay had each of the samples tested for grain size analysis. Four of the samples were tested for Atterberg Limits. Holladay estimated ranges of field capacity and wilting point but did not provide specific values for individual samples.

Tetra Tech also collected soil samples during two investigations (April and July 2016) to evaluate near-surface soils for suitability as use for final cover material. Five test pits were excavated east and northeast of Phase 1 of the landfill in April. Thirteen additional test pits were excavated in July to provide additional spatial coverage of the area extending further south and west. The second round of testing pitting focused on identifying the upper and lower bounds of the silt loam layer target for the final cover. The maximum depth of the test pits was 13.5 feet. Nine samples were submitted for grain size analysis, two samples for Atterberg Limits, and four for Proctor compaction testing (moisture-density relationship). Four samples were also analyzed for permeability, field capacity, and wilting point. This data was presented as part of an alternative cover evaluation (Tetra Tech 2016) that was approved by SWDH on December 8, 2016 and by the DEQ on December 9, 2016. This document is provided in the supplemental material provided as part of the application.

Tetra Tech conducted another test pit investigation in October 2022 to evaluate soils in the County-owned property south of Missouri Avenue as part of the expansion investigation to ensure sufficient, suitable cover material would be available for closure. Fifteen samples were collected for grain size analysis and Proctor compaction tests, three samples were tested for permeability, eight samples were analyzed for field capacity, wilting point, and porosity, and five samples were tested for Atterberg limits. The soils from the 2022 investigation have lower permeability and would be better for final closure then the soils used for the alternative cover application. Therefore, the combination of the originally identified soils for final closure evaluated in 2016, as well as the additional soils available south of Missouri Avenue would ensure sufficient final cover material is available for closure of Phase 5.

Table 6 summarizes the results of the testing of the shallow soil samples from the three test pit investigations. Laboratory analysis included measurements of field capacity (holding capacity) that represents the amount of moisture the soil can hold against drainage by gravity. As the particle surface area (e.g. finer material) and organic matter increases, the moisture retention capacity of the soil increases resulting in a higher field capacity. The test pit samples from 2016 were collected to the east and northeast of the active landfill and showed an average field capacity of 13.8%. The test pit samples from 2022 collected south of Missouri Avenue had an average field capacity of 37.9%. The difference between these two areas is a higher sand content in the area east and northeast of the active landfill but may also reflect a higher organic content in the southern soils. The difference is field capacity is also reflected in the lower hydraulic conductivity for the southern soils. Given the low precipitation in the area around the landfill and the thickness of soil above the water bearing zone, there is more than sufficient capacity in the soil to retain the annual precipitation.

Table 6: Test Pit Data Summary

Sample No.	Depth (ft)	Soil Type (USCS)	Perme-ability (cm/s)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Passing No. 200	Field Capacity (% Moisture)	Wilting Point (% Moisture)	Porosity (%)	Atterberg Limits PL/LL/PI
1A	2	ML	-	-	-	55%	-	-	-	20/22/2
2A	2	ML	-	-	-	55%	-	-	-	-
2B	10	SM	-	-	-	15%	-	-	-	-
3A	2	SM	-	-	-	49%	-	-	-	-
3B	9	SM	-	-	-	20%	-	-	-	-
4A	2	ML	-	-	-	53%	-	-	-	18/20/2
4B	10	SM	-	-	-	29%	-	-	-	-
5A	2	SM	-	-	-	47%	-	-	-	-
5B	9.5	SM	-	-	-	18%	-	-	-	-
6A	2	ML	-	-	-	53%	-	-	-	-
6B	10	SM	-	-	-	23%	-	-	-	-
7A	2	ML	-	-	-	70%	-	-	-	NP/17/NP
7B	10	SM	-	-	-	16%	-	-	-	-
8A	2	ML	-	-	-	60%	-	-	-	NP/24/NP
8B	9.5	SM	-	-	-	26%	-	-	-	-
9A	2	ML	-	-	-	68%	-	-	-	15/20/5
9B	10.5	SM	-	-	-	15%	-	-	-	-
10A	2	SM	-	-	-	33%	-	-	-	-
10B	6.5	SM	-	-	-	22%	-	-	-	-
11A	2	ML	-	-	-	60%	-	-	-	NP/25/NP
11B	9	SM	-	-	-	36%	-	-	-	-
12A	2	SM	-	-	-	27%	-	-	-	-
12B	9	SM	-	-	-	12%	-	-	-	-
13A	2	SM	-	-	-	15%	-	-	-	-
13B	5	SM	-	-	-	20%	-	-	-	-

Sample No.	Depth (ft)	Soil Type (USCS)	Permeability (cm/s)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Passing No. 200	Field Capacity (% Moisture)	Wilting Point (% Moisture)	Porosity (%)	Atterberg Limits PL/LL/PI
14A	1-3	SM	-	-	-	34.0%	-	-	-	-
14B	4-5	SM	5.63E-04	109.5	15.0%	23.0%	13.13	8.59	-	NP/NP/NP
15A	1-4	ML	4.24E-05	111.0	13.9%	65.0%	17.84	4.37	-	NP/NP/NP
16A	0.5-1.5	ML	-	-	-	64.0%	-	-	-	-
16B	4-5.5	SM	1.74E-04	113.5	11.9%	41.0%	11.53	6.09	-	NP/NP/NP
16C	8.5-9.5	SM	-	-	-	36.0%	-	-	-	-
17A	1.5-5.5	ML	-	-	-	51.0%	12.80	6.52	-	-
18A	2-3.2	ML	-	-	-	67.0%	-	-	-	-
18B	7-8	SM	1.22E-05	86.0	31.8%	15.0%	-	-	-	42/64/22
TP-1	2-3	ML	-	98.4	18.6%	71.5%	-	-	-	NP/NP/NP
TP-2	6-8	ML	-	96.3	20.5%	67.3%	-	-	-	NP/NP/NP
TP-3	2-3	SM	2.11E-07	98.9	20.5%	40.2%	35.2	8.85	48.44	-
TP-4	4-5	ML	2.54E-07	91.8	14.4%	64.8%	37.6	6.1	51.82	-
TP-5	2-3	SM	-	102.9	17.3%	49.0%	-	-	-	NP/NP/NP
TP-6	2-3	ML	-	97.6	19.3%	71.6%	-	-	-	-
TP-7	4-5	ML	1.87E-07	99.2	19.0%	76.1%	36.3	6.6	48.67	-
TP-8	2-3	SM	-	98.5	18.7%	49.6%	-	-	-	-
TP-9	4-8	ML	-	90.9	22.4%	77.2%	41.6	7.15	53.14	-
TP-10	5-6	ML	-	94.1	21.4%	66.7%	37.4	8.43	49.85	-
TP-13B	0-1	SM	-	91.7	23.8%	32.6%	-	-	-	-
TP-14	4-5	ML	-	101.6	17.4%	78.4%	36.9	10.8	50.86	NP/NP/NP
TP-15	2-3	CL-ML	-	104.3	16.1%	77.1%	-	-	-	-
TP-16	4-8	ML	-	95.7	20.7%	75.3%	40.8	8.11	53.68	NP/NP/NP
TP-17	1-3	ML	-	97.0	19.9%	72.4%	37.7	6.96	51.15	-

Notes:
Samples 1A through 13B collected by Holladay Engineering Company, ca. 1994; Samples 14A through 18B collected by Tetra Tech, April 2016; Samples TP-1 through TP-17 collected by Tetra Tech, October 2022.
ML – silt; SM - silty sand; CL - lean clay
Atterberg Limits Abbreviations: PL=Plastic Limit, LL=Liquid Limit, PI=Plasticity Index, NP=Non-Plastic.
- Indicates test not conducted.

2.6.2 Drilling Investigations

Several subsurface investigations have been conducted since the early 1990s. These generally served one of two purposes: obtaining information on the hydrogeologic properties of the subsurface materials (usually during groundwater monitoring well installation) and collecting information for geotechnical engineering studies.

Holladay conducted a geotechnical investigation that included five borings drilled in November 1996. Over 80 samples from these borings were collected for various analyses; many of them were tested only for moisture content (Holladay, 1998). The results are included in Appendices E and F of the 1998 Holladay report.

Tetra Tech collected samples from 8 borings drilled in 2021 as part of a geotechnical slope stability evaluation. 21 samples were submitted to a geotechnical soils laboratory for Atterberg limits, 19 samples were submitted for grain size analysis, 9 samples were tested for Proctor compaction testing, 11 samples were tested for friction angle and cohesion, and 5 for unconfined compressive strength. 46 samples were also tested for natural moisture content.

The complete report for the slope stability evaluation is included **Appendix F**. The results of the testing are included in Appendix C of the Tetra Tech report.

Holladay collected 11 core samples from three of the monitoring well borings drilled in 1992. DBS&A collected 56 samples from the five monitoring well borings they drilled in 2011. Sample analysis from both of these investigations concentrated on physical properties of the soil and rock relative to hydrogeologic characteristics of the material.

The samples collected by Holladay ranged from 206 to 479 feet deep. These represent elevations ranging from approximately 2621 to 2320 feet AMSL. The lithologic descriptions of the materials included clayey silt, silty clay, and claystone. Table 5 of the 1994 Holladay report summarizes the pertinent tests and results from their testing. Saturated hydraulic conductivities in these samples ranged from 1.8×10^{-9} to 1.0×10^{-4} centimeters per second (cm/sec).

The 56 samples from the DBS&A drilling program in 2011 ranged from 39 to 750 feet deep. The elevations represented by the samples ranged from approximately 2793 to 2294 feet AMSL. The samples were analyzed for moisture content, dry bulk density, saturated hydraulic conductivity, moisture characteristics, grain size distribution, specific gravity, porosity (calculated), and Atterberg limits (DBS&A 2014a). The range of saturated hydraulic conductivity across all of the samples ranged from 4.29×10^{-9} to 7.24×10^{-4} cm/sec. These closely match the range from the 1992 Holladay investigation.

Table 7 presents a summary of the drilling investigations described above. These reports can be reviewed for additional details including the locations of the geotechnical borings.

Table 7: Summary of Drilling Investigations

Reference Report	Borings Drilled	Purpose	Number of Samples Collected	Tests Conducted
Holladay 1994	PB-2 through PB-8(A)	Monitoring Well Installation and Hydrogeologic Characterization	11	Moisture, bulk density, wilting point, specific retention, porosity, absolute and saturated hydraulic conductivity
Holladay 1998	GT-1 through GT-5(B)	Geotechnical Evaluation	87	Grain size distribution, Atterberg limits, bulk density, moisture, triaxial compression
DBS&A 2014a and 2014b	PB-11 through PB-15	Monitoring Well Installation and Hydrogeologic Characterization	56	Moisture, dry bulk density, saturated hydraulic conductivity, moisture characteristics, grain size distribution, specific gravity, porosity (calculated), Atterberg limits
Tetra Tech 2022	B2021-1 through B2021-8	Geotechnical Evaluation	68	Moisture, grain size distribution, Atterberg limits, friction angle/cohesion, Proctor compaction testing, consolidation, unconfined compressive strength

(A) Lab samples collected only from PB-2, PB-3, and PB-4
(B) Core samples saved from PB-2 were also submitted for analysis

2.7 GROUNDWATER

Groundwater beneath the Landfill has been comprehensively studied beginning with the Holladay investigations described above and listed in **Table 7**. The Holladay work was followed by investigations conducted by DBS&A, and then by Tetra Tech. **Figure 3** (**Appendix A**) shows the location of the site wells and other wells in the area. Fifteen monitoring wells and one former water supply well have provided information on the site's hydrogeology.

Canyon County has been granted a waiver for conducting groundwater monitoring at the Landfill, but has voluntarily conducted quarterly or semi-annual monitoring. This groundwater monitoring program has been used to study the groundwater flow characteristics and groundwater composition beneath and adjacent to the Landfill. Groundwater at the Landfill has a unique chemistry and is greater than 400 feet deep. The potential for impacts to groundwater from the Landfill are negligible because of the depth to groundwater and the geologic stratigraphy described above.

2.7.1 Groundwater Conditions

Holladay identified three water bearing zones during a literature review and their investigation and referred to them as the Upper Aquifer (UA), Middle Aquifer (MA), and Bottom Aquifer (BA). It should be noted that while these names may correspond to subsurface intervals that produce water, they are not necessarily considered aquifers because of low production rates or quality concerns. DBS&A acknowledged the naming convention used by Holladay and used similar reference names in their 2014 report (uppermost-unconfined aquifer or unconfined aquifer, middle confined aquifer, or confined aquifer, and bottom aquifer).

Monitoring wells have been constructed to characterize the first groundwater encountered at each location. Monitoring wells on the eastern part of the project area are completed in the Upper Aquifer in unconfined conditions. The remainder of the project wells are completed in the Middle Aquifer in confined conditions. The uppermost (unconfined) aquifer has been characterized at monitoring wells PB-5, PB-6, PB-7, PB-9, and PB-10. This water bearing zone is not present beneath the entire Landfill area; it is limited to the area at the northeast corner of the active Landfill and certification area; it is not present above the expansion area. The Middle Aquifer has been characterized by the former shop domestic well (PB-1), former monitoring wells PB-2, PB-3 and PB-4, and current monitoring wells PB-8 and PB-11 through PB-16. **Table 8** is a summary of monitoring well construction information.

Table 8: Summary of Monitoring Well Construction Information

Well Number	Groundwater Source	Screened Interval(s)	Depth to Top of Confining Layer	Total Depth Drilled	Approx. Depth First Water Encountered	Depth to Potentiometric Surface* (March 2023)
PB-1	Glenns Ferry Fm - Confining Layer	577-367	251	658	595	NA - Well Closed prior to 1997
PB-2	Glenns Ferry Fm - Confining Layer	407-420, 515-530	280	557	490	NA- Well Closed prior to 1997
PB-3	Glenns Ferry Fm - Confining Layer	340-350, 410-420, 520-530	263	860	410	NA - Well Closed June 2017
PB-4	Glenns Ferry Fm - Confining Layer	560 - 575, 605 - 620	422	640	565 - 630	NA- Well Closed September 2020
PB-5	Glenns Ferry Fm – unconfined	512.5 - 522.5	630	660	517	NA - Well Closed August 2021
PB-6	Glenns Ferry Fm – unconfined	487.5 - 497.5	620	700	490	NA - Well Closed August 2021
PB-7	Glenns Ferry Fm – unconfined	535 - 555	540	610	535	550.81
PB-8	Glenns Ferry Fm - Confining Layer	377 - 407	240	420	380	286.19
PB-9	Glenns Ferry Fm – unconfined	508 - 543	510**	544	Unknown	529.87
PB-10	Glenns Ferry Fm – unconfined	504 - 534	515**	560	Unknown	525.83

PB-11	Glenns Ferry Fm - Confining Layer	340 - 400	200	420	350 - 400	292.35
PB-12	Glenns Ferry Fm - Confining Layer	480 - 540	140	555	500 - 560	304.27
PB-13	Glenns Ferry Fm - Confining Layer	840 - 900	545	923	850 - 900	728.78
PB-14	Glenns Ferry Fm - Confining Layer	845 - 905	522	923	800 - 840	712.91
PB-15	Glenns Ferry Fm - Confining Layer	790 - 850	565	870	800 - 860	652.76
PB-16	Glenns Ferry Fm - Confining Layer	572-592	262	600	580 - 590	550.83
Measurements are in feet and referenced to ground surface except as noted						
*Referenced to top of casing, typically about 2 feet higher than ground surface						
**Based on interpretation from driller's log						

2.7.2 Upper Aquifer

The wells set in this zone encountered water between approximately 490 to 535 feet deep, or between elevations of about 2330 and 2400 feet AMSL. The saturated thickness of the Tuana Gravel at these locations is on the order of tens of feet, with groundwater present at depths ranging from about 500 to 550 feet deep. These depths have steadily increased over the duration of groundwater monitoring (groundwater elevations have decreased). The elevation at MW-7 was 2401.18 in September 1992; the elevation in September 2024 was 2388.02. This steady decrease of 0.4 foot per year has been echoed by the other four unconfined monitoring wells. This resulted in MW-5 and MW-6 becoming dry in 2003 and 2021 respectively. The wells were closed in August 2021. Groundwater in the unconfined aquifer flows to the northeast with a hydraulic gradient of approximately 0.05 to 0.06 feet per foot. As shown on **Cross Section A** in **Appendix D**, the confining layer on the eastern side of the landfill also has a downward slope toward to the east. DBS&A's analysis was that the slope of the upper surface of the groundwater in the unconfined aquifer is similar to the slope of the top of the confining layer (DBS&A 2014a).

2.7.3 Middle Confined Aquifer

The middle confined aquifer is located within the blue clay unit and appears to underlie the entire expansion area. Observations during the previous investigations indicated that water within the confining layer is present in deeper fractures within that unit. DBS&A's interpretation was that the material is harder and more brittle with depth and can support open fractures, while the shallower parts of it are more plastic and not able to support open fractures (DBS&A, 2014a).

The middle aquifer is currently characterized by current monitoring wells PB-8 and PB-11 through PB-16. The depth to the top of the piezometric surface ranges from less than 300 feet for wells installed in the lower parts of Wildhorse Canyon, to over 700 feet for wells on the Pickles Butte Ridge. Groundwater in the middle confined aquifer moves to the southwest with a gradient of approximately 0.03 to 0.04 feet per foot.

The depth to the water bearing zone in the middle confined aquifer wells ranges from over 300 feet to almost 900 feet BGS, which corresponds to elevations ranging from 2125 to 2340 feet AMSL. The potentiometric surface in these wells in March 2023 ranged from 2345 to 2423 feet AMSL, indicating the presence of a positive pressure head. This positive head is present at each of these wells, ranging from approximately 35 feet at PB-16 to approximately 225 feet at PB-12. This positive head exerts an upward pressure on the confining layer and would this inhibit the downward migration of fluids from above the confining layer and into the water bearing zone.

Monitoring wells PB-4 and PB-8 were installed in 1992 and provide the longest duration of piezometric levels in the middle confined aquifer. The elevations of the piezometric surface over the duration of the monitoring program at these two locations have had different trends. The piezometric surface elevation at PB-4 decreased by 0.5 foot per year through April 2007. After that time, the level stabilized. PB-16, installed as a replacement of PB-4, has also shown stable levels since it was installed in 2020. Contrasting this is the piezometric surface at PB-8 which has shown a steady increase in elevation over time, with an average increase of 0.4 foot per year. PB-11, located approximately 1200 feet west of PB-8 and also on the north side of Wildhorse Canyon, has shown a similar increase. Monitoring wells PB-12 through PB-15 are middle confined aquifer wells located on the south side of Wildhorse Canyon. These have shown a greater increase in the elevation of the piezometric surface over time, averaging over 0.7 foot per year.

Canyon County Solid Waste (CCSW) has acquired two properties on the south side of Deer Flat Road that were formerly used as homesteads or residences. Each of these has a domestic well that provides information on the middle confined aquifer north of the existing monitoring well network. Property formerly owned by Don and Shelly Stuart includes a well that is approximately 1800 feet north of monitoring well PB-8. The driller's log indicates that the top of the piezometric surface was 330 feet BGS. This equates to a piezometric surface elevation of approximately 2419 feet AMSL. Property formerly owned by David Snell includes a well that is approximately 1400 feet north of monitoring well PB-11. The driller's log indicates that the top of the piezometric surface was 370 feet BGS. This equates to a piezometric surface elevation of approximately 2367 feet AMSL. Both of these values agree with the groundwater depth and migration direction of water in the middle confined aquifer. The driller's logs for each of the wells show hundreds of feet of unsaturated clay or sandstone above the middle confined aquifer, which is also in agreement with observations from the previous investigations at the Landfill.

2.7.4 Lower Confined Aquifer

Holladay defined the lower confined aquifer as the next water bearing zone beneath the blue clay. Their interpretation appears to have been mainly based on inspection of area domestic and irrigation well logs showing deeper water-producing zones of sand or rock beneath the material described as blue clay on the drillers' logs. Holladay's review of the logs for wells that penetrate the lower confined aquifer showed that there is usually clay units that do not produce water between the middle confined aquifer and the lower confined aquifer (Holladay, 1994).

2.7.5 Aquifer Recharge

Recharge to the upper aquifer is postulated to be from surface sources, including Lake Lowell, surface irrigation, and possibly irrigation canals. Precipitation is not likely a significant contribution to recharge because of the low annual precipitation for the area. The middle confined water bearing zone is believed to be recharged by underflow from geothermal water that lies beneath the blue clay. Anderson and Wood (1981) theorized that recharge to the thermal system may be taking place slowly over a long period of time with little present day recharge. A depletion of heavy isotopes in the geothermal waters may indicate that recharge to this aquifer occurred more than 10,000 years ago.

2.7.6 Groundwater Quality

Groundwater monitoring has been conducted using the monitoring wells installed at the site since April 1995. Through 2016, groundwater samples were collected using stainless steel bailers. Beginning in December 2017, groundwater samples have been collected using dedicated pneumatic submersible pumps under a site specific groundwater monitoring plan that was last updated in November 2023 (Tetra Tech). Groundwater monitoring was conducted on a quarterly basis with the submersible pumps through September 2019. Since that time, groundwater monitoring has been conducted semi-annually, with monitoring events typically occurring in March and late August/early September. A monitoring report has been prepared summarizing the results of each sampling event. These reports have been submitted to the DEQ and should be referenced for the sampling results and statistical analysis.

2.7.7 Groundwater Quantity

The yield from the groundwater monitoring wells at the site is low to very low. Regionally, the production from wells in the upper unconfined aquifer vary spatially. Observations made at site monitoring wells completed in this unit (PB-5, PB-6, PB-7, PB-9 and PB-10) indicates that recharge takes place in a matter of hours to days. This is contrasted with monitoring wells completed in the middle-confined unit, where the time to recharge after bailing or pumping in monitoring wells PB-1, PB-2, and PB-3 was noted to take months (Holladay, 1994). The middle-confined unit is the first water bearing zone underlying the expansion area.

2.8 VERTICAL DISTANCE – WASTE TO WATER

The geologic cross sections in **Appendix D** includes information on the bottom of waste, the hydraulic head in the water bearing unit (referred to as the potentiometric surface), and the top of the confined aquifer (water bearing zone). This provides a visual of the distance between the bottom of waste and water bearing zone for compliance with §39-7409 (c)(i), Idaho Code - Standards for Design that requires solid waste should not be deposited within fifty (50) feet of the seasonal high ground water elevation in the uppermost aquifer. For example, on **Sheet X-104**, monitoring well PB-8 will be within the excavation and will be abandoned as discussed in *Section 3.4*. Based on data from September 2024, which is representative of historical data at the site, the potentiometric surface is 251 feet below the bottom of the waste and the water bearing confined aquifer is 357 feet below the bottom of waste. Similarly, on **Sheet X-108** that is near boring B2021-7, data from September 2024, the potentiometric surface is estimated to be 270 feet below the bottom of the waste and the water bearing confined aquifer is estimated to be near 375 feet below the bottom of waste. This demonstrates that the proposed expansion exceeds the requirements in §39-7409 (c)(i), Idaho Code for the distance between waste and the upper most water bearing zone.

2.9 GEOTECHNICAL EVALUATION

Tetra Tech conducted a geotechnical study in 2021 and 2022 to support the lateral expansion of the landfill. The investigation evaluated the proposed permanent excavation slopes that are planned to be on the order of 3H:1V to 4H:1V, with maximum cut depths on the order of 150 to 165 feet. The full report is available in **Appendix F**.

Tetra Tech previously completed a slope stability evaluation that included static and seismic stability evaluations for Phases 2 through 4 of the Canyon County Landfill (October 7, 2015). Tetra Tech also reviewed the previous evaluations conducted by Holladay in 1998 and conducted a seismic survey in December 2021. The survey was designed to image and delineate a suspected fault in support of the proposed expansion program at the PBSL. The seismic survey is discussed in *Section 2.10* and the report is available in **Appendix G**.

For the stability evaluation, Tetra Tech incorporated the following information: 1) the soils strength data available from previous analyses, 2) materials strength properties assigned based on the laboratory testing of the geotechnical samples collected in 2021 and also correlated from the Standard Penetration Testing (SPT) N-value (blow count) data collected during the geotechnical drilling and previous well installation reports.

Based on findings from this and former site investigations, the subsurface conditions beneath the areas of proposed landfill expansion are assumed to generally consist of silty and clayey sand, clay, and gravel overlying the Glenns Ferry Formation (300 to 950 feet thick), which includes younger lacustrine and fluvial sediments. The surrounding local geology includes an igneous basalt group of the Hat Butte-McElroy Butte type[1] that was not encountered in area of the proposed expansion.

Slope stability and pseudo-static analyses were performed using the computer program Slide2 (2020), developed by Rocscience, Inc., to determine the factors of safety (FS) of critical slip surfaces using both circular (rotational) and block failure analyses and vertical slice limit equilibrium methods. Circular failures can be viewed as a soil ‘slump’ with a remnant head ‘scarp’ or drop in elevation where the slide started, and a resultant ‘hump’ or bulge at

[1]Mancos - Macrostrat.org

the slide terminus. A block failure represents a large mass or 'chunk' of soil failing outwardly as a larger intact mass. Where the pseudo-static analysis indicated a factor of safety of equal to or less than 1.3 (industry standard for pseudo-static factor of safety for landfills), the internal slope of the landfill cell prior to waste emplacement was evaluated using the Newmark displacement analysis method to determine a range of potential seismic-induced deformations of the refuse mass.

Results of the slope stability evaluations indicate that the preliminary design for the expansion phases will meet the requirements of the Idaho Administrative Rules IDAPA 58.01.06 for the Idaho DEQ's administration of MSWLF. The analyses indicate static FS values on the order of 1.38 to 2.43, and 1.83 to 3.11 for circular and block failure respectively, while the pseudo-static FS values were on the order of 0.99 to 1.88, and 1.45 to 2.16 for circular and block failure, respectively. Subsequent seismic deformation analyses indicate maximum probable displacements on the order of 0.25 to 3.19 inches (0.5 to 8 cm) for the anticipated peak ground acceleration of 0.12g generated during the design seismic event at the project site. In general, the seismic displacement analyses indicate permanent seismic-induced displacements within the tolerances 6 to 12 inches (15 to 30 cm) that are typically considered acceptable for design of landfill systems with no liner.

Multiple slope angles were considered for Tetra Tech's slope analyses, ranging from 2.5:1 to 4:1 depending on the soil and bedrock types at each location. Based on Tetra Tech's analysis and the required FS's, the following two slope angles are recommended for the preliminary landfill site grading plans:

3H:1V: for the majority of the site slopes

4H:1V: where silt is encountered (Section F discussed below)

The 4:1 slope was analyzed and recommended for Section F because silt was interbedded between poorly-graded sand and fine sand and created a weakened soil profile. In areas where a high concentration of silt is predominant during construction, a slope of 4H:1V is recommended for cut areas. The soil profile within Section F was identified as having a high concentration of silt in the upper 135 feet of the proposed slope cut, thus decreasing the factor of safety. There are other areas where the silt was present; however, based on the analysis the proposed cut slope of 3H:1V was allowable for the silts as they were interbedded into stronger soil deposits. As the stratification is exposed during excavation of future cells, it is recommended that the soil conditions be reviewed to verify they match the design criteria.

The slope with compacted refuse were modeled to confirm the slope angles that were allowable during the backfilling process. Slopes of 3H:1V are recommended as a maximum angle for the backfill process. A steeper slope of 2.75H:1V was modeled as an iteration to confirm the recommendations, and in this situation the pseudo static conditions produced a factor of safety below 1.3 and is not recommended.

Portions of the soil profile were defined as claystone and have unconfined compression strengths higher than the site soils; however, the claystone had interbedded layers of softer soils, and for this reason Tetra tech has treated these areas as a soil rather than a rock and also recommends a slope cut of 3H:1V for the claystone zones.

2.10 SEISMIC STUDY

Tetra Tech conducted an active-source 3D seismic survey at the site in December 2021. The seismic survey was designed to image and delineate a suspected fault in support of the proposed landfill expansion. Seismic imaging over the suspected fault area was attained by using 3D seismic velocity tomography and reflection processing. The complete report is available in **Appendix G**.

The results from the 3D survey revise the location and structure of the USGS mapped NW-striking NE-dipping WSRP normal fault across the project site. The new 3D imaging of fault structure demonstrates that faulting along the USGS NW-striking NE-dipping WSRP normal fault is tapering to zero west of geotechnical boring B2021-5 and that residual fault deformation is distributed amongst a network of tip splay faults across the project site. Thus, primary normal fault slip is unlikely east of the west edges of the tip splay faults. Instead, any fault slip associated with earthquakes along the USGS mapped NW-striking NE-dipping WSRP normal fault will likely partition into attenuated fault slip among the splay faults within the project site. There may be additional limited extent (strike lengths < 200 feet) fault splays and relay fault within or outside of the 3D seismic volume extent. Distributed small stepover and relay faults commonly occur between large fault stepovers.

Typically, in highly weathered rock or in poorly consolidated sediments, fault slip transitions to distributed deformation or bedding flexure prior to reaching the ground surface. Tip splay faulting may decrease with decreasing depth above the water table and transition to flexure or distributed deformation. This is the most likely scenario for the PBSL project site. The projected intersection of the SW-dipping tip splay fault at a depth of 81 feet in geotechnical boring B2021-5 near the base of a zone of distributed broken clay deformation, suggests the fault has produced distributed deformation in the 66-86-foot depth interval of borehole B5. Since the age of this depth interval in geotechnical boring B2021-5 is probably much greater than the ~100 ka overlying unfaulted geologic strata used by Personius (2003) to constrain the most recent age of active faulting along the WSRP normal faults, this possible fault deformation observed in borehole B5 in the 66-86-foot depth interval is likely older than 100 ka.

The USGS NW-striking NE-dipping WSRP normal fault that is mapped as extending into the project site from the northwest does not appear to displace ~100ka age sedimentary units (Personius, 2003). From a probabilistic perspective there seems to be little possibility of significant shallow (< 200 feet) faulting within the project site southeast of the west edges of the mapped tip splay faults (negligible nonzero fault slip for annual exceedance probabilities greater than 0.01%). To best characterize the potential movement and absolute location of faulting would require geologic mapping during excavation of a future landfill cell. The current expansion application does not include the area identified for additional investigation and would need to be considered if future lateral expansions of the landfill are considered in the area of interest. This could be done when the area is excavated as a borrow source for cover material. At that time, geologic mapping of the fault could be conducted, with particular attention to identifying narrow fault zones with evidence of recent activity and areas of potential distributed deformation. Careful sampling can yield materials suitable to date the most recent age of fault activity to determine if any detected fault activity is recent (unlikely) or > 100 ka in age (most likely).

3.0 LATERAL EXPANSION DESIGN

The requested expansion design expands the current footprint of the landfill from 116.7 acres to 200.5 acres, which covers an additional 83.8 acres. The design primarily expands the landfill to the west of the current footprint, and then covers Phase 3 and incorporates the majority of Phase 4.

The excavation and final surface design was based on existing conditions and limitations, which included:

- Maintain the Phase 5 expansion within the existing landfill gas (LFG) header pipeline to the maximum extent possible;
- Maintain the current location of the condensate tank;
- Utilize the borrow source excavation area that provides daily cover for Phase 3;
- Maintain a 2.5% minimum floor slope along the bottom of the waste;
- Adhere to the geotechnical design criteria of 4H:1V for the outside slopes;
- Incorporate as much of the existing approved Phase 4 design;
- Maintain the peak height of the landfill at or below the level of Phase 4 to minimize visibility;
- Maintain access along Perch Road, which extends from the scale house to the lower areas for waste placement during both the excavation and waste placement portions of Phase 5 for as long as possible;
- Allow for wider operating floors for waste placement to improve operational efficiency and allow separation of commercial haulers and the public;
- Incorporate a stormwater pond inside the LFG header to minimize run-off;
- Incorporate a new road along the northside of the expansion for access as the lower portion if filled with waste; and
- Relocate the storage of white goods, concrete, clean wood, and green waste from the landfill face to reduce maintenance of the intermediate cover.

These criteria were used to determine the depth of excavation to the south, west, and northwest and determine the potential air space available from the expansion. **Appendix H** contains the design drawings. The excavation plan is shown on **Sheet C-101**, and the lowest elevation of waste is 2640 feet amsl. The excavation would include 1,806,895 cubic yards of soil. Although some of the soil would be utilized for daily the cover, the site is soil heavy and a significant portion of the soil would need to be relocated. The County owns the land south of Deer Flat Road and east of the portion of Perch Road that extends off Deer Flat Road. This area is a low spot and is currently used by OHV for recreation. The area was evaluated and has the capacity required for relocation of the clean soil.

Sheet C-102 (**Appendix H**) shows the Phase 5 top of waste plan with a maximum elevation of 3000 feet amsl. Intermediate and final cover would be placed on the surface shown. This results in an additional 24,360,554 cubic yards of air space. If approved, the landfill would modify operations to start filling Phase 5 on the western side of the landfill and build successive lifts going up to the east, which would delay the completion of Phase 3 and Phase 4. This would improve operational efficiency by allowing wider lifts, facilitate separation of commercial haulers and the public, and reduce modification to traffic flow. The northern and southern sections would be tied into the design based on elevation of the lifts. Based on past acceptance rate, if it is assumed that 370,000 tons of waste is accepted per year this will add approximately 36 years of operational life above the remaining capacity available in Phase 3 and Phase 4, assuming no growth. However, if it is assumed that the tonnage rate increases by 1% per year, Phase 5 would only provide approximately 29 additional years.

Sheets C-103 to **Sheet C-107** (**Appendix H**) shows various cross sections through the proposed expansion and differentiates between Phase 3 and Phase 4 that are already approved and Phase 5. **Sheet C-106** shows that with lateral expansion, the tie-in point for the top of the landfill would be further west then under the approved Phase 4 design and a small portion (114,264 cubic yards) of the Phase 4 design would not be utilized.

3.1 LANDFILL GAS SYSTEM

3.1.1 Regulatory Framework

State and Federal regulations require that landfill gas (LFG) generated by the facility be controlled to prevent migration beyond the property boundary, and to mitigate surface emissions beyond certain limits (500 parts per million by volume). PBSL operates under Air Quality Permit T1-2017.0049, dated March 1, 2018. PBSL is currently operating under a continuance of the permit. An application for renewal of the permit was submitted on August 30, 2022. A draft of the new permit (T1-2022.0038) was received from DEQ on September 24, 2024, and is currently being reviewed. The waste capacity of the PBSL is over 2.5 million megagrams (Mg), making it a major source of hazardous air pollutants (HAPS).

On June 21, 2021, the Federal Plan Requirements for Municipal Solid Waste Landfills that commenced construction on or before July 17, 2014, and have not been modified or reconstructed since July 17, 2014 (Federal Register, Volume 86, No. 97, page 27756, May 21, 2021) under authority of the Clean Air Act became effective. These rules reduced the allowable NMOC emissions rate for landfills before a landfill gas collection system must be installed from 50 Mg/year to 34 Mg/year.

In May 2019, the site specific NMOC concentration was 422 parts per million (ppm) based on the Tier 2 Testing for Non-Methane Organic Compounds (NMOC), which resulted in an annual NMOC emission rate of 37.3 Mg/year. Therefore, as a result of the change in regulations in 2021, PBSL was required to install a Gas Collection and Control System (GCCS).

3.1.2 Gas Collection and Control System

In response to the change in regulations, PBSL commissioned the design and installation of a GCCS that covers landfill operations through the completion of Phase 3. A permit to construct (PTC) was submitted to DEQ on December 23, 2022, for a candle stick flare associated with the GCCS design. The PTC for the GCCS system was approved on July 28, 2023. The GCCS consists of 14 vertical wells, 17 horizontal collectors (10 installed and 7 future), associated pipelines, control valves, and pneumatic condensate pumps that are tied in to a 10-inch header pipe that extends around the landfill perimeter. **Sheets C-101** and **C-102** (**Appendix H**) show the existing header pipe that is tied into a gas handling system consisting of 2 variable drive blowers, condensate knock-out vessel, and control panel. The vacuum blowers draw the LFG from the collection points to the control system. All LFG extracted by the GCCS is combusted by a 1,360 standard cubic feet per minute (scfm) flare. The system became operational on March 19, 2024.

3.1.3 Expansion of the Landfill Gas System

One objective of the expansion design was to stay within the existing landfill gas header pipeline and still be able to tie-in future horizontal LFG collectors. This would also minimize modification to the location of currently in-place manifolds, sumps, or highpoint valves. This was possible for the majority of the expansion, except in the southern area where it was necessary to tie into the hillside to promote positive drainage, minimize stormwater run-on from the radio antenna area, and improve capture of run-off water. In addition, it also provided additional waste capacity. **Sheet C-108** (**Appendix H**) shows how the landfill gas header line will need to be extended as part of the expansion design. The drawing shows the location of existing blind flanges where the extension would tie into to the existing header line. **Sheet C-108** also shows the location of an existing manifold for horizontal collectors from Phase 3 that will need to be relocated.

New vertical and horizontal collectors were not designed as part of the expansion design at this time. This is because the system just became operation in 2024, the current design extends through the life of Phase 3, and that the current flare only has an estimated lifespan of 10 years (Tetra Tech 2022). The current flare is capable of processing up to 1,360 scfm. In September 2024, the flare was operating at a flow rate of 950 scfm. The modeled peak flow rate of 1,148 scfm was calculated during the design phase and was estimated to occur in the year 2034 but is dependent on the tonnage and type of waste landfilled (i.e. organic versus C&D). As a result, as operational data is obtained, it will be necessary for the landfill to modify the Gas Collection and Control System (GCCS) system from a candlestick flare to a larger flare or develop a waste to energy program for the LFG (e.g. engines or liquified natural gas). Therefore, design of additional vertical and horizontal LFG collectors will be required as part of the system upgrade and will be designed based on operational data in the future. Any modifications to the GCCS system will be submitted to DEQ and any required air permits (e.g. Permit to Construct) will be submitted prior to any modifications to the existing system.

3.2 HYDROLOGIC ANALYSIS

PBSL reviews, and if required updates the stormwater plan approximately every five years based on the location of the active face and how well existing controls are working. The last update was completed on August 5, 2020, for Phase 3 (Tetra Tech 2020). The Phase 3 design was used as the starting point for the final Phase 5 design because the location of ponds and mechanism for drainage along bench roads are effective for the management of stormwater and minimizing erosion at the PBSL.

For the final Phase 5 design, Tetra Tech performed a detailed rainfall-run-off hydrologic analysis to estimate peak run-off rates and volumetric inflows for the 25-year, 24-hour design storm utilizing the National Resource Conservation Service (NRCS) Curve Number (CN) Method and the United States Army Corps of Engineers' (USACE) HEC-HMS software. The analysis presented in this section is the end point of the stormwater controls for Phase 5. Interim reviews and potential updates will be required between when the expansion design is approved, landfill operations are modified to more efficiently fill the landfill, and the design presented in this section is constructed. The parameters used for the hydrologic analysis and results used for the final hydraulic/stormwater management structures for the final Phase 5 design are provided in the following sections.

3.2.1 Rainfall Data

Site-specific rainfall data for the 25-year, 24-hour storm were obtained from the National Oceanic and Atmospheric Administration's (NOAA) Precipitation-Frequency Atlas of the Western United States, Atlas 2, Volume V-Idaho. Using the available isopluvial maps referenced in NOAA Atlas 2, Vol V-Idaho, point precipitation for the 25-year, 24-hour storm event is estimated to be 1.8 inches (NOAA, 1973).

3.2.2 Drainage Area Delineation

The drainage area for PBSL was delineated using client-provided survey data and aerial imagery from September 2023 and the final Phase 5 design utilizing AutoCAD Civil 3D software. The landfill conducts an aerial survey annually on September 30th of each year to reflect changes and modifications based on operations and to evaluate landfill performance. The entire drainage area reporting to existing or proposed stormwater structures encompasses approximately 247.9 acres. The drainage areas were delineated to account for the areas that report to the various existing or proposed stormwater management structures such as channels, culverts, and retention ponds.

Drainage area soils and land cover types were characterized by assigning Curve Numbers to each respective drainage area. Landfill cover material has been assigned a Curve Number of 70 and is assumed to be vegetated. The entrance to the landfill consists of paved/gravel roads, buildings, and other impermeable surfaces. As such, this area can be expected to have a higher run-off potential. For the site entrance area, a Curve Number of 85 has been assigned. Run-on areas upgradient of the landfill have been characterized as “Sagebrush with grass understory” and has been assigned a Curve Number of 51. A full list of assigned drainage areas, Curve Numbers, and landcover characteristics are provided in **Tables 1, 2, 3 and 4** of **Appendix I**. The drainage area delineations are shown on **Sheet C-109** and **C-110** (**Appendix H**).

3.2.3 Time of Concentration

The NRCS Watershed Lag Method was used to calculate concentration times for each individual subbasin. This method was developed for use in nonurban watersheds and accounts for a lag time for each subbasin that distributes the respective run-off hydrograph peaks as they occur naturally without reaching the design point simultaneously (NRCS 2010). The NRCS Watershed Lag Method calculates time of concentration using the following equation referenced from Part 630, Chapter 15, of the National Engineering Handbook (NRCS 2010):

$$T_c = \frac{L^{0.8}\,(S+1)^{0.7}}{1140Y^{0.5}}$$

T_c = Time of Concentration, hr

L = Flow Length, ft

S = Maximum Potential Retention, in

Y = Average Watershed Land Slope, %

Referring to the NEH Part 630, the relation between lag and time of concentration can be expressed as L (lag time) = $0.6T_c$, where the lag time is a factor of the time of concentration multiplied by 0.6. The lag time is defined as the delay between the time run-off begins until it reaches its peak.

3.2.4 Hydrologic Model (HEC-HMS)

Tetra Tech utilized the USACE’s HEC-HMS software to simulate the 25-year, 24-hour storm event using the NRCS CN Method. The hydraulic evaluation of the proposed stormwater management structures used an estimated Soil Conservation Service (SCS) Type II rainfall distribution, peak run-off rates and inflow volumes for the study area. The following parameters and assumptions were used to quantify the estimated peak flows and inflow volumes:

- Precipitation: Based on the site’s location, an SCS Type II Storm was selected as the synthetic rainfall distribution
- Run-off Volume: The SCS CN Method was used to model the estimated run-off
- Direct Run-off: The model used the SCS Unit Hydrograph transform method with a Standard (PRF 484) graph type

Results from the HEC-HMS hydrologic model is provided in **Appendix I**.

3.3 HYDRAULIC ANALYSIS

Based on the calculated peak flows and volumes that report to the proposed stormwater management structures, the capacities of each proposed structure were evaluated to confirm they can safely capture, convey, and store the estimated run-off from the 25-year, 24-hour design storm event.

3.3.1 Regulations

Stormwater discharges are regulated by the Environmental Protection Agency (EPA) under Code of Federal Regulations (CFR) Title 40 sections 122, 123, and 124 (National Pollutant Discharge Elimination System (NPDES) Permit Application Regulations for Storm Water Discharges). These rules have been in effect since December 17, 1990, and apply to landfills that are subject to regulation under subtitle D of Resource Conservation and Recovery Act (RCRA). The rules cover the discharge of stormwater that flows from a waste containing area of the facility to any offsite collection system. If the stormwater runoff from a waste containing areas is collected and treated on-site these regulations do not apply. The design collects stormwater runoff from the waste containing area of the Site and routes them to onsite retention basins through a series of benches, ditches, and culverts.

Per the e-CFR website, amended September 24, 2024, the following surface water requirements shall apply under Title 40, Chapter 1, Subchapter I, Part 258, Subpart C, §258.26:

Title 40: Protection of Environment

PART 258 – CRITERIA FOR MUNICIPAL SOLID WASTE LANDFILLS

Subpart C – Operating Criteria

§258.26: Run-on/run-off control systems.

(a) Owners or operators of all MSWLF units must design, construct, and maintain:

(1) A run-on control system to prevent flow onto the active portion of the landfill during the peak discharge from a 25-year storm;

(2) A run-off control system from the active portion of the landfill to collect and control at least the water volume resulting from a 24-hour, 25-year storm.

(b) Run-off from the active portion of the landfill unit must be handled in accordance with §258.27(a) of this part.

3.3.2 Stormwater Channels and Run-On Control

The lateral expansion of the PBSL will require additional stormwater controls to effectively manage stormwater run-off. Stormwater controls for the landfill expansion have been designed to capture most of the run-off except for a small area on the eastern slopes. Run-off from this eastern area will flow into the existing gravel pit that is within the controlled access for the site.

Four stormwater channels are proposed around the perimeter of the landfill expansion to capture and convey run-off. The location of the proposed channels are shown on **Sheet C-109** (**Appendix H**) and the channel dimensions are shown on **Sheet C-113** (**Appendix H**). The four channels are as follows:

- North Channel: Triangular shaped (V-ditch) stormwater channel that will capture stormwater run-off from the northern benches of the landfill and convey the captured run-off to the proposed Northwest Pond. This channel will parallel the outer edge of the existing landfill gas header road. A 1-ft tall earthen berm should be constructed along the outer edge of North Channel to prevent run-on from upgradient areas.
- East Channel: A trapezoidal shaped stormwater channel that captures stormwater from of portion of the landfill eastern slopes and conveys run-off to the proposed Eastern Pond. This channel will be constructed along the outside edge of the existing road.

- West Channel 1: A trapezoidal shaped stormwater that captures stormwater from the southern benches of the landfill in addition to some run-on from an upgradient area. This channel will convey captured run-off to an existing drainage west of the landfill within the controlled access area.
- West Channel 2: A triangular shaped (V-ditch) channel that runs parallel to the inside edge of the existing landfill gas header road. This conveyance channel with direct captured run-off from a small portion of the landfill southern benches and tie into a bench drainage channel network.

Additional internal stormwater channels that will be required include are shown on **Sheet C-109** (**Appendix H**) and include:

- Half Corrugated Metal Pipes (CMP): CMPs will be installed along the fill slopes of the landfill to drain water off the face of the landfill. The half CMPs should include a region at the bottom of each lift (i.e. distance between stormwater benches or approximately 40 ft) for transition to connect to the next adjoining earthen channels and/or half CMP down slope drainage channel. Energy dissipation devices (e.g. rock gabions or concrete splash walls) should be installed on the side opposite their respective adjoining earthen channels as required based on the length of the channel and if required to prevent erosion after a significant storm event. These features will serve to reduce channel velocity prior to their respective junctions.
- Bench Drainage Channel: Triangular shaped (V-ditch) conveyance channels along the benches of the landfill (**Sheet C-113**, **Appendix H**). These drainage channels convey captured run-off from the landfill slopes to the half-CMPs or the North and West Channels.

An existing series of culverts and drainage channels exist adjacent to the landfill entrance and scale house area. Run-off from the southeastern slopes of the landfill will be captured and conveyed via this existing stormwater conveyance system to the existing Southeast Pond that will be modified as part of the design. Except for West Channel 1, all stormwater is directed to on-site retention ponds.

Run-on to the landfill is expected to occur from an upgradient area west of the landfill and will be captured by West Channel 1 then diverted away from the landfill. Minor run-on along the northern perimeter of the landfill is expected. As such, a 1-ft tall run-on berm on the outside edge of the North Channel is proposed. Along the northwestern perimeter, downstream of West Channel 1, minor run-on may occur. A 1-ft tall run-on berm should be installed on the inside edge of the existing road, terminating at the proposed North Pond. Refer to **Sheet C-110** in **Appendix H** for the location of the run-on berms.

A hydraulic analysis using Bentley FlowMaster hydraulic calculator software and Manning's Equation for open-channel flow were used to determine minimum geometric and hydraulic properties for proposed channels. Hydraulic results for the channels are presented in **Table 9** below.

Table 9: Stormwater Channels Design Properties

Stormwater ID	Peak Discharge (cfs)	Length (ft)	Geometric Shape	Side Slopes (XH:1V)	Bottom Width (ft)	Minimum Design Depth (ft)
North Channel	0.2	3,125	Triangular	2	N/A	1
East Channel	1.2	680	Trapezoidal	2	1	1
West Channel 1	1.6	2,370	Trapezoidal	2	1	1
West Channel 2	0.2	710	Triangular	2	N/A	1
Bench Drainage Channel	0.8	Varies	Triangular	2	N/A	1

3.3.3 Stormwater Retention

Three retention ponds are proposed and designed to store captured runoff from the 25-year, 24-hour storm event. They are designated the Northwest Pond, East Pond, and Southeast Pond, and were designed to minimize discharge from the project site.

The Northwest Pond is designed to retain runoff from a large portion of the north and northwest landfill slopes. Through a series of channels and CMPs, runoff will be conveyed to the Northwest Pond. This pond is oversized and has more than ample capacity to store the estimated inflow volume.

The East Pond has been designed to store captured run-off from the eastern slopes of the landfill. Planned construction, existing roadways, and spatial constraints limit the East Channel from capturing all runoff from the eastern landfill slopes. Run-off from the eastern landfill slopes that would otherwise be directed to the East Pond will likely flow into the existing gravel pit.

The Southeast Pond is a modification of the existing pond near the site entrance. Specifically, the Southeast Pond will be deepened by excavating a foot from the existing pond bottom, the existing dam will require minor regrading and the addition of a foot of fill to raise the dam crest elevation. Although these modifications provide sufficient capacity for the 25-yr 24-hr storm event, an overflow pond was added downstream of the Southeast Pond to address concerns of neighboring landowners should the site experience sequential low-probably storms. The Southeast Pond and the overflow will be connected via a 3-ft wide x 2-ft deep trapezoidal overflow weir. Between the two ponds, there is more than ample capacity to store captured runoff from multiple 25-yr 24-hr storm events.

Stage-storage data for the four ponds are provided in **Tables 10, 11, 12** and **13** below. **Sheet C-111** and **Sheet-112** (**Appendix H**) show the pond cross sections.

Table 10: Northwest Pond Stage-Storage

Stage (ft)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
2616	0.99	0.00	0.00

Stage (ft)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
2617	1.05	1.02	1.02
2618	1.11	1.08	2.10
2619	1.17	1.14	3.24
2620	1.23	1.20	4.44
2621	1.29	1.26	5.70
2622	1.36	1.33	7.03
2623	1.43	1.39	8.42
2624	1.49	1.46	9.88
2625	1.56	1.53	11.41
2626	1.64	1.60	13.01
*Required capacity = 1.1 ac-ft			

Table 11: East Pond Stage-Storage

Stage (ft)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
2909	0.00	0.00	0.00
2910	0.12	0.06	0.06
2911	0.14	0.13	0.19
2912	0.15	0.14	0.34
2913	0.17	0.16	0.49
2914	0.18	0.17	0.67
*Required capacity = 0.3 ac-ft			

Table 12: Southeast Pond Stage-Storage

Stage (ft)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
2895	0.00	0.00	0.00
2896	0.01	0.00	0.00
2897	0.06	0.03	0.04
2898	0.09	0.07	0.11
2899	0.12	0.10	0.21
2900	0.14	0.13	0.34
2901	0.17	0.16	0.50
2902	0.20	0.18	0.68
2903[1]	0.23	0.21	0.89
2904	0.26	0.25	1.14
2905	0.31	0.29	1.43

*Required capacity = 1.1 ac-ft

1Overflow weir invert elevation

Table 13: Southeast Overflow Pond Stage Storage

Stage (ft)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
2892	0	0.00	0.00
2893	301	0.00	0.00
2894	925	0.01	0.02
2895	1,695	0.03	0.05
2896	2,693	0.05	0.10
2897	3,737	0.07	0.17
2898	4,965	0.10	0.27
2899	6,162	0.13	0.40

Stage (ft)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
2900	7,317	0.15	0.55
2901	8,457	0.18	0.74
2902	9,686	0.21	0.94
2903	10,989	0.24	1.18
2904	12,368	0.27	1.45
2905	14,566	0.31	1.76

3.3.4 Stormwater Culverts

The proposed North and East Channel alignments include road crossings and as a result a culvert for each of the channels is required to convey flow through these roads.

A hydraulic analysis using the U.S. Federal Highway Administration's HY-8 Culvert Hydraulic Analysis Program was completed to adequately size culverts which are to be used to convey captured runoff under existing and proposed roads.

The proposed culverts were analyzed as corrugated metal pipes with projecting inlet and outlet configurations. Culvert properties are presented in **Table 14** below. HY-8 culvert analysis results are provided in **Appendix I**.

Table 14: Stormwater Culvert Properties

Culvert ID	Material	Shape	Diameter (ft)	Length (ft)	Inlet Invert (ft)	Outlet Invert (ft)
North Culvert	Corrugated Metal Pipe	Circular	0.5	32	2,723.81	2,723.20
East Culvert	Corrugated Metal Pipe	Circular	1	22	2,926.82	2,926.38
*Culvert inverts are approximate and should be field verified upon installation						

3.4 GROUNDWATER MONITORING UPDATE

Monitoring well PB-8 is located within the footprint of the expansion area. This well will be properly abandoned before Phase 5 is excavated and filled in this area. Since this well in an upgradient background well, two new monitoring wells will be installed outside of the expansion area to provide groundwater quality data upgradient of the new cell. The proposed replacement wells are shown on **Sheet C-101** and **C-102** (**Appendix H**) and would be installed and monitored for 8 quarters before well PB-8 is abandoned. Existing monitoring wells PB-11 through PB-15 will provide downgradient coverage.

3.4.1 PB-8 Abandonment

PB-8 was installed in 1993 to a depth of 417 feet BGS using 4-inch diameter steel casing. It will be abandonment in accordance with IDWR regulations to prevent it from being a conduit of fluid or vapors to the subsurface. This well was installed with 8-inch diameter outer steel casing to a depth of 190 feet BGS. Outer casing was not placed between the 4-inch monitoring well casing and the walls of the boring below 190 feet where the consolidated formation allowed the boring to remain open. The annular space between the well casing and the walls of the boring was filled with bentonite chips. Over time, the bentonite can become desiccated enough to allow vapors to pass through it.

The submersible sampling pump, tubing, and cable will be removed from the well before the abandonment process. To help prevent the boring from being a conduit for vapors, the well casing will be perforated, and a pressure grouting technique will be used during the abandonment. The 4-inch diameter well casing will be perforated from 175 feet below ground surface to the bottom of the well (417 feet BGS). The perforations may be made with an air knife, mills knife or other appropriate method that creates an opening large enough to allow grout to be pushed into the space outside the well casing. At least four equally spaced perforations around the circumference of the casing spaced no greater than one foot apart vertically will be created to comply with IDWR rules.

The well will be pressure grouted with a cement/bentonite-based grout after the perforation is complete. A suitable packer or other seal will be placed near the surface to allow the grout to be injected into the casing under a minimum pressure of 20 psi at ground surface to force the grout out through the perforations into the filter pack, dried bentonite seal, or voids. The grout will be placed through a tremie pipe, from the bottom of the well up, for the full length of the well to approximately 6 feet below the future cut elevation. Additional neat cement grout will be added to match this elevation as needed following overnight settlement. The remainder of the well casing will be filled with hydrated bentonite chips and a temporary but secure cap will be placed on the well. Having bentonite chips instead of cement in the upper portion of the casing will allow CCSW personnel to periodically cut the top part of the casing as the excavation for Phase 5 progresses. Each time the casing is cut, water will be added as needed to maintain the hydration of the bentonite chips and the cap will be replaced.

After the final cut elevation is achieved in this area, an additional six feet of soil will be removed from around the well casing. The casing will be cut off at that level, and a steel cap will be welded on top of it as a permanent cap. The excavation around the casing will then be backfilled up to the bottom elevation of the Phase 5 cut so that the casing will be protected from damage.

3.4.2 New Upgradient Well Installation

The proposed locations of the two new upgradient monitoring wells are shown on **Sheets C-101** and **C-102** in **Appendix H**. These wells will be installed and monitored for at least 8 quarters prior to the abandonment of well PB-8. They will be incorporated into the sampling program upon completion, using the schedule described below. At least two sampling events will be conducted that includes PB-8 and the new wells prior to the abandonment of PB-8 to verify correlation of groundwater conditions.

One of the wells will be installed approximately 250 north-northwest of PB-8 and 1130 feet east of PB-11. The ground surface at this location is similar to that at PB-8 (approximately 2707 feet AMSL). The direction of groundwater flow at PB-8 is slightly south of west, so the piezometric surface at this location is also expected to be similar to that at PB-8. The piezometric surface elevation at PB-8 was 2426.37 feet AMSL when measured in early September 2024.

The second new monitoring well will be installed approximately 1000 feet east-northeast of PB-8 and 1000 feet west of former monitoring well PB-6. The first groundwater at this location should be beneath the confining layer, though it is possible that the unconfined conditions found at PB-6 extend this far to the west. At the location of PB-6, the current elevation of the unconfined groundwater is approximately 2370 feet AMSL, with groundwater moving to the east. The selected location is almost due north of former monitoring well PB-1, where the unconfined aquifer was not encountered. Groundwater at PB-1 was produced from sandy shale at an elevation beginning at approximately 2100 feet AMSL, beneath the confining layer. The piezometric surface after drilling was noted to be approximately 250 feet higher, or approximately 2350 feet AMSL. Holladay (1994) noted that the true piezometric may have been much higher had the well been allowed to recover longer. Extrapolating the groundwater levels from the recent monitoring events, the level of unconfined groundwater (if present) at the proposed location may be near 2440 feet AMSL, while the piezometric surface of the confined layer is expected to be near 2460 feet AMSL.

The well will be installed such that it is screened across the first water producing zone(s), but not spanning the confining layer. If the well is located in the area where confined conditions exist, it is expected that the piezometric surface will be in the range of 100 to 300 feet higher than the zone(s) that produce water.

Each of the two new wells will be installed with rotary drilling techniques. 8 or 10-inch diameter steel casing will be installed in the unconsolidated materials in the upper portion of the borings to prevent the walls of the borings from collapsing. The actual depths of the bottom of the casing will be determined during drilling. This casing will be permanently sealed to the walls of the boring with cement grout so that a conduit for subsurface vapor is not present.

The monitoring wells will be set once the final depth of the boring is established; this depth will be determined based on field observations and measurements of groundwater egress into the boring. The wells will be constructed with 4-inch diameter steel casing. The casing sections will be threaded together to provide smooth internal walls. The bottom of the casing will include an end cap connected to a five-foot section of blank (non-screened) casing to provide a sump at the bottom of the wells. Wire-wrapped screening casing will be placed above the sump spanning the zone(s) of water production. Blank casing will be placed above the screened section; the blank casing will then extend approximately two feet above the ground surface.

Each of the wells will be developed by surging and pumping or bailing. Once the installation and development for both wells is complete, dedicated, pneumatically powered submersible pumps will be installed into them. The pumps will be the same model or current equivalent to the pumps that are in the other PBSL monitoring wells (QED Environmental ST100PM). The new wells will be sampled within one week after pump installation. The wells will then be sampled on a quarterly basis until 8 sampling events have been conducted. Four of these events will coincide with the semi-annual monitoring program (March and late August or early September). The other four events will be conducted in June and December. After the 8 quarterly sampling events have been conducted, the wells will be sampled semi-annually. The sampling will be conducted in accordance with the Sampling and Analysis Plan that is current at the time of the sampling. The most current Plan is dated December 1, 2023 (Tetra Tech, 2023).

4.0 SUPPLEMENTAL REPORTS

In addition to the Geotechnical Investigation and Seismic Survey reports included in **Appendices F** and **G**, several other reports prepared for PBSL between 1998 and 2016 contain information to support this expansion application. These are being submitted electronically with this application in a separate folder. The following is a list of the reports including the year published, title, author, and relevant information.

- 1994 - Hydrogeologic Characterization, Ground Water Monitoring Plan, and Facility Design (Holladay). This report includes a description of the geologic and hydrogeologic conditions at the landfill, a summary of the installation and testing of monitoring wells PB-2 through PB-8, core sample collection and testing, HELP modeling to estimate travel times and support the non-lined arid design, cell design, surface water management, and cover design.
- 1998 – 1997 Landfill Status Report (Holladay). This report included a summary of the previous designs, an evaluation of project capacity, presented the groundwater monitoring plan, and discussed Title V compliance.
- 1998 – Geotechnical Evaluation (Holladay). The results of a geotechnical investigation were presented in this report. Samples were collected from borings GT-1 and GT-5 that were analyzed for various physical properties. Core samples previously collected from the boring for monitoring well PB-2 were also tested. Analysis of loading was conducted, and the potential for liquefaction and settlement was addressed.
- 2014 – Monitor Well Installation (DBSA). The installation of monitoring wells PB-11 through PB-15 is described in this report. The wells were completed between June and October 2011. The analysis of

laboratory samples for soil and rock is not discussed in this report. The laboratory results for the initial groundwater sampling event from these wells (collected April 2012) is included as an appendix.

- 2014 – Hydrogeologic Characterization Report, Volume 1 (DBSA). This report included the results of laboratory testing of core samples collected during the 2011 drilling program (Table 3 of the report). The results were used for infiltration modeling using Darcian flux calculations, the HYDRUS model, and the HELP model. The modeling results were summarized in Tables 5 through 10 and Table 14 of that report. The calculated travel times from the top of the confining layer to the groundwater beneath it ranged from 3,158 to over 52,000 years.
- 2014 – Hydrogeologic Characterization Report, Volume 2 (DBSA). This second volume of the report contained the laboratory results of the core samples collected during the 2011 drilling program. These results support the modeling that is described in Volume 1.
- 2015 – Landfill Status Report Update (Tetra Tech). The update of the PBSL status report included summaries of previous investigations, statistical analyses of the results of groundwater sampling, modeling for LFG emissions, a slope stability evaluation, stormwater controls, and cost estimates for closure and post-closure maintenance.
- 2016 – Alternative Final Cover System Equivalency Demonstration (Tetra Tech). This document proposed a cover system consisting of mulch for erosion control over an infiltration control layer, which would in turn be placed over an intermediate cover layer. This was proposed as an alternative to the EPA Subtitle D prescriptive final cover system. The report included a description of the borrow source investigation, laboratory testing of soil samples, infiltration modeling, and a grading plan. The DEQ approved the alternative cover design in a letter dated December 9, 2016.
- 2023 - Groundwater Sampling & Analysis Plan (Tetra Tech). This document outlines the procedures used to collect groundwater samples, the frequency, and the quality assurance requirements.

5.0 REFERENCES

40 CFR 258, 2024. https://www.ecfr.gov/current/title-40/chapter-I/subchapter-I/part-258/subpart-C/section-258.27#p-258.27(a), September 24, 2024.

Bentley Flow Master Hydraulic Modeling Software V81 (SELECT series 1) 2009, Bentley Systems Incorporated.

City of Nampa, 2024. Climate. https://www.cityofnampa.us/909/Climate accessed September 2024.

DBS&A (Daniel B. Stephens & Associates), 2010. Site Certification, Pickles Butte Sanitary Landfill. Report to Canyon County, June 17.

DBS&A. 2014a. Hydrogeologic Characterization Report, Pickles Butte Sanitary Landfill, Volumes 1 and 2. Report to Pickles Butte Sanitary Landfill, April 25.

DBS&A. 2014b. Monitor Well Installation, Pickles Butte Sanitary Landfill. Report to Pickles Butte Sanitary Landfill, April 25.

Digital Atlas of Idaho, 2023. Snake River Plan. Available on-line at https://digitalatlas.cose.isu.edu/geo/snkrvpln/snkrvpln.htm. Accessed December 2023.

Holladay (Holladay Engineering Company), 1994. Hydrogeologic Characterization, Groundwater Monitoring Plan, & Facility Design, Pickles Butte Sanitary Landfill, Canyon County Idaho, July 1994.

Holladay, 1998. Geotechnical Evaluation Pickles Butte Sanitary Landfill, Canyon County, Idaho. February 1998.

Hydrologic Modeling Software (HEC-HMS) 4.4 2020, US Army Corps of Engineers, Davis, CA.

Idaho Department of Water Resources (IDWR) 2024. Evapotranspiration and Consumptive Irrigation Water Requirements for Idaho, Nampa, Idaho https://et-idwr.idaho.gov/stninfo.py?station=2168 Accessed October 2024.

Kimmel, P.G., 1982, Stratigraphy, age, and tectonic setting of the Miocene-Pliocene lacustrine sediments of the western Snake River Plain, Oregon and Idaho, in Bill Bonnichsen and R.M. Breckenridge, eds., Cenozoic Geology of Idaho: Idaho Bureau of Mines and Geology Bulletin 26, p. 559-558.

Mabey, D.R. 1982. Geophysics and Tectonics of the Snake River Plain, Idaho. Cenozoic Geology of Idaho: Idaho Bureau of Mines and Geology Bulletin 26. Bill Bonnichsen and R.M. Breckenridge, editors. P. 139-153.

Malde, H.E., 1959, Fault zone along northern boundary of western Snake River Plain, Idaho: Science, v. 130, p. 272.

Malde, H.E., 1972. Stratigraphy of the Glenns Ferry Formation from Hammett to Hagerman, Idaho. United States Geological Survey Bulletin 1331-D.

Malde, H.E., and H.A. Powers, 1962, Upper Cenozoic stratigraphy of western Snake River Plain, Idaho: Geological Society of America Bulletin 73, p. 1197-1220.

Malde, H.E., and H.A. Powers, 1972, Geologic map of the Glenns Ferry-Hagerman area, west-central Snake River Plain, Idaho: U.S. Geological Survey Miscellaneous Investigations Map I-696, scale 1:48,000, 2 sheets.

Maley, Terry, 1987. Exploring Idaho Geology, Mineral Land Publications, Boise, ID.

Mayo, Alan L., Muller, Anthony B., and Mitchell, John C., 1984. Geochemical and Isotopic Investigations of Thermal Water Occurrences of the Boise Front area, Ada County, Idaho. In Idaho Department of Water Resources Bulleting No. 30, Geothermal Investigations In Idaho. December.

National Oceanic and Atmospheric Administration (NOAA) Precipitation-Frequency Atlas of the Western United States Atlas 2 Volume V - Idaho 1973. US Department of Commerce.

National Resources Conservation Service, United States Department of Agriculture (NRCS), 2010. National Engineering Handbook Part 630 Hydrology, Chapter 15, Time of Concentration. Washington D.C.

National Resources Conservation Service, United States Department of Agriculture (NRCS), 2023. Web Soil Survey. Available at: https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx. Accessed December 2023.

Personius, S.F., compiler, 2003, Fault number 635, Western Snake River Plain fault system, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, https://earthquakes.usgs.gov/hazards/qfaults, accessed December 2023.

Ruez, Dennis R., 2009. Framework for stratigraphic analysis of Pliocene fossiliferous deposits at Hagerman Fossil Beds National Monument, Idaho. Rocky Mountain Geology, v. 44, no. 1, p. 33–70, 10 figs., 2 appendices, May.

Savage, Carl N., 1968. Lexicon of Idaho Geologic Names, Idaho Bureau of Mines and Geology Information Circular #20. December.

Shervais, J.W., Gaurav Shroff, S.K. Vetter, Scott Matthews, B.B. Hanan, and J.J. McGee, 2002, Origin and evolution of the western Snake River Plain: Implications from stratigraphy, faulting, and the geochemistry of basalts near Mountain Home, Idaho, in Bill Bonnichsen, C.M. White, and Michael McCurry, eds., Tectonic and Magmatic Evolution of the Snake River Plain Volcanic Province: Idaho Geological Survey Bulletin 30, p. 343-361.

Swirydczuk, K., Larson G.P., and Smith, G.R., 1982. Volcanic Ash Beds as Stratigraphic Markers in the Glenns Ferry and Chalk Hills Formations from Adrian, Oregon to Bruneau, Idaho. Cenozoic Geology of Idaho: Idaho Bureau of Mines and Geology Bulletin 26. Bill Bonnichsen and R.M. Breckenridge, editors. P. 543-558.

Tetra Tech 2023. Groundwater Sampling & Analysis Plan, Pickles Butte Sanitary Landfill, December 1, 2023.

Tetra Tech, 2022. Pickles Butte Sanitary Landfill, Basis of Design Report, Nampa, Idaho. April 28, 2022.

Tetra Tech, 2020. Pickles Butte Sanitary Landfill, Phase 3 Stormwater Controls, Nampa, Idaho. August 4, 2020.

Tetra Tech, 2016. Alternative Final Cover System Equivalency Demonstration, Pickles Butte Sanitary Landfill. November 28, 2016.

U.S. BLM, undated. Geology of Southwest Idaho. Informational pamphlet prepared for Hulls Gulch National Recreation Trail in Idaho.

USGS, 1968. 7.5 Minute Series Topographic Map, Givens Hot Springs Quadrangle, Idaho – Canyon County.

USGS, 1968. 7.5 Minute Series Topographic Map, Lake Lowell Quadrangle, Idaho – Canyon County.

Wood, S.H., and J.E. Anderson, 1981, Chapter 2, Geology, in J.C. Mitchell, ed., Geothermal Investigations in Idaho, Part 11: Geological, Hydrological, Geochemical, and Geophysical Investigations of the Nampa-Caldwell and Adjacent Areas, Southwestern Idaho: Idaho Department of Water Resources, Water Information Bulletin 30, p. 9-31.

Wood, Spencer H., 1994. Seismic Expression and Geological Significance of a Lacustrine Delta in Neogene Deposits of the Western Snake River Plain, Idaho. APG Bulletin, Volume 78, No. 1, P. 102-121. January.

APPENDIX A : FIGURES

APPENDIX B: SITE CERTIFICATION

APPENDIX C: SOILS DATA

APPENDIX D: GEOLOGIC CROSS SECTIONS

APPENDIX E: WELL AND BORING LOGS

APPENDIX F: GEOTECHNICAL INVESTIGATION REPORT

APPENDIX G: SEISMIC INVESTIGATION REPORT

APPENDIX H: LANDFILL EXPANSION DESIGN DRAWINGS

APPENDIX I: HYDROLOGY AND HYDRAULIC CALCULATIONS