6NYCRR Part 363 - Engineering Design Report

Seneca Meadows Landfill

SMI Valley Infill

JULY 17, 2020

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PRESENTED TO

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REPORT CERTIFICATION

The material and data in this report were prepared under the supervision and direction of the undersigned.

7/17/20

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APPENDIX SECTIONS

1.0 INTRODUCTION

This Engineering Design Report (Report) for the proposed SMI Valley Infill development at Seneca Meadows Landfill (Landfill) describes the design criteria for the proposed facility as well as the considerations for the landfill construction. Landfill operations, maintenance and monitoring are presented in the Facility Manual submitted as an appendix to this Report.

1.1 PROJECT DESCRIPTION

The proposed modification of Seneca Meadows Landfill consists of a lateral and vertical development of the existing 330 +/- acre landfill footprint. The lateral development consists of approximately 47 +/- acres between the Western Expansion (WEX) and the Southeast Landfill (SELF). Construction of the lateral development will also support a vertical development of the landfill facility from an existing peak elevation of approximately 774 ft to a proposed peak elevation of 843.5 ft after the installation of the final cap.

The baseliner will be a geosynthetic, double-composite lining system with both primary and secondary leachate collection. A porewater collection system will be utilized to control groundwater level in the subgrade. The lining, leachate collection and porewater collection systems will be comparable to the systems currently approved and in place at the facility.

Leachate generated in the landfill area will be collected and conveyed into the existing on-site leachate storage and pumping systems. Temporary stormwater controls constructed within the landfill cells will optimize stormwater run-off and minimize the generation of leachate from stormwater running into areas of waste placement.

Waste placement operations will be performed by existing staff in accordance with established waste placement procedures.

Landfill gas generated within the landfill will be managed through the construction of new horizontal and vertical landfill gas collection wells. The landfill gas wells will be consistent and compatible with the existing landfill gas collection and control systems. Collected landfill gas will be managed by the existing landfill gas blowers and flare or electrical generation systems as appropriate.

Upon reaching final grades, the landfill will receive final cover. The final cover will include a network of sideslope diversion channels to collect surface water run-off and direct it into the on-site basins. Lined downchutes will be utilized to convey water directly down the final cover slope. The final cover will include a subsurface drainage system to remove infiltrated stormwater from above the geomembrane.

New infrastructure required for the construction and operation of the landfill development will be a relocated residential facility, maintenance facility, fuel island and wheel wash facility.

The design and function of the individual components of the landfill are discussed in greater detail in the following sections of this Report. This Report is for the proposed landfill and was prepared in conformance with the permit application requirements contained in the NYS Part 360 Regulations.

1.2 CONTENTS OF APPLICATION

The following documents constitute the submittal to the NYSDEC for the application for a permit to construct the proposed Landfill and are appended to this report:

- Engineering Drawings
- Hydrogeologic Investigation Report
- Construction Quality Assurance and Construction Quality Control Plan

- Facility Manual
- Drainage Report and Preliminary SWPPP

A Draft Environmental Impact Statement (DEIS) as well as a Gas Collection and Control System Plan (GCCS) will be submitted under separate cover.

1.3 BACKGROUND

Seneca Meadows, Inc. owns the Seneca Meadows Landfill Facility (Facility), located at 1786 Salcman Road, Waterloo, New York. The Facility is also maintained, managed, and operated by Seneca Meadows, Inc. (SMI), who hold an agreement with the Towns of Seneca Falls and Waterloo. Although the address of the Facility is in Waterloo, New York, the majority of the Facility property is located in Seneca Falls, New York, where landfilling of waste occurs.

The Facility is comprised of: the landfill, which includes closed and active units; the Citizens Drop-off Area (CDA); the Tire Processing Facility (TPF); the Landfill Gas Management System; the yard debris area; and Leachate Storage and Treatment Facility. In addition, the Facility maintains a secure container for regulated medical waste (RMW). RMW stored on site is disposed of at a permitted off-site location. Also, the Town sponsors a Household Hazardous Waste (HHW) Collection Day, which is held at the Facility. HHW collected is managed by the Town and no HHW is disposed of on site. Although located on Facility property, SMI is not under agreement to maintain, manage, nor operate the High BTU facility that is owned and operated by Seneca Energy II, LLC or the Landfill Gas-to-Energy (LFGTE) plant owned and operated by Seneca Energy, Inc.

The Facility (#50S08) is operated under New York State Department of Environmental Conservation (NYSDEC) Solid Waste Management Permit (Permit #8-4532-00023/00001). Currently, the Facility is permitted to accept up to 6,000 tons per day (tpd) of waste under the Permit. The 6,000 tpd is measured on an annual basis, with the equivalent of no more than 9,000 tpd accepted during a given quarter.

2.0 SITE ANALYSIS

2.1 SITE LOCATION AND DESCRIPTION

SMI's solid waste management facility is located within approximately 2,400 acres of property holdings owned by SMI. The facility is located in central New York State in the Towns of Waterloo and Seneca Falls, both of which are in Seneca County, New York. The Site is located in the northern portion of Seneca County, midway between the cities of Rochester to the west and Syracuse to the east. The Title Sheet of the Engineering Drawings provides a map that illustrates the location of the Site on a regional basis and the Facility Boundary Map/Survey Control Figure is included in Appendix G of this report. The Site is approximately equidistant between the northerly limits of Seneca Lake to the west and Cayuga Lake to the east. The Seneca-Cayuga Canal, which connects Seneca Lake with Cayuga Lake, is located approximately 1.8 miles south of the Facility. The Lake Ontario shoreline is approximately 25 miles north of the facility.

The facility is regionally accessible via the New York State Thruway and U.S. Routes 5 & 20. Entrance to the site is via Salcman Road, which intersects New York State Route 414 (also known as Mound Road) approximately 3.5 miles south of the New York State Thruway exit 41, and approximately 1 mile north of Route 414's intersection with U.S. Routes 5 & 20. The major east-west routes are well maintained and provide convenient all-weather access to New York State Route 414, a paved high volume arterial roadway which is the primary roadway used by vehicles traveling to the Site. In addition, vehicles originating from the local service area also utilize NYS Route 96 to access the facility. The existing local and regional transportation routes as described provide a superior system in support of the facility operation.

Seneca Meadows is currently the largest landfill in New York State, with a permitted waste capacity of 6,000 tons per day (tpd). The primary service area of the Facility is New York State, providing waste disposal services for communities and businesses state-wide. As shown in Table 1 (Appendix A of this report) in 2018, 89.8% of the waste (excluding BUD materials) accepted at SMI was generated in New York. The balance of the waste was generated in Massachusetts (8.3%), and other states (less than 2%).

The Landfill will be owned and operated as a private solid waste disposal facility. The primary service area of a private landfill is not limited by municipal boundaries in the same way that such boundaries may limit municipally owned disposal sites. The primary service area for the Landfill will be New York State. Service area boundaries for the proposed landfill are not intended to be permanent or exclusive. Competing solid waste collection and disposal facilities exist within the service area, and this competition benefits the residents and industries located in these areas.

2.1.1 Landfill Vicinity

Seneca Meadows is one of the most important solid waste disposal facilities in New York State. As illustrated by data available from NYSDEC in 2014, SMI provides disposal for over 20 percent of New York's in-State waste disposal needs.

Locally, SMI's importance as a disposal facility is much more pronounced. Most of the counties in the immediate vicinity of the SMI landfill do not have their own landfill disposal capacity.

These counties include the following:

- Cayuga
- Livingston
- Onondaga
- Schuyler

- Seneca
- Tioga
- Tompkins
- Wayne
- Yates

Seneca Meadows accepted over 276,735 tons of solid waste and BUD material from these counties in the year 2018. Moreover, Seneca Meadows accepts ash from the Onondaga County WTE facility for disposal, and thereby, helps in the provision of 350,000 tons of disposal capacity that the WTE facility accounts for.

Sheet 3 of the accompanying Engineering Drawings (Appendix B of this report) presents the Regional Vicinity Map, which encompasses an area of a minimum of one mile from the boundaries of the facility. The area within the Regional Vicinity Map boundaries includes portions of the Towns of Seneca Falls, Waterloo, Junius and Tyre, all in Seneca County. Two population centers are located proximate to the Site. The center of the SMI Site is approximately 2.1-miles northeast of the Village of Waterloo and approximately 2.6 miles northwest of the Village of Seneca Falls.

The zoning classifications for Seneca Falls and Waterloo are illustrated on the Property Holdings Map, Sheet 4 of the Engineering Drawings. The Facility is located primarily within the M-2 Refuse Disposal and Reclamation District of the Town of Seneca Falls, with landfilling identified as a permitted use. The M-1 Industrial zone designation applies to an area south of the M-2 zone, on SMI property located within the Town of Waterloo. This area primarily consists of the existing permitted Salcman Road mine. South of the M-1 zone and immediately west of the facility is designated primarily as R-1 Residential within the Town of Waterloo. The C-2 Highway Commercial zone designation has been applied to the area adjacent to NYS Route 414, and floodplain zone designation has been applied to the narrow strip adjacent to Black Brook within the C-2 zone, and through Agricultural zoning to the east.

East of Route 414, property owned by SMI includes that zoned as floodplain, agricultural, and industrial. Property zoned as floodplain is primarily associated with the Black Brook flood plain. Remaining zoning in the Site vicinity is predominantly classified A-1 or A-2 Agricultural, C-2 Highway Commercial, or M-1 Industrial.

Land use in the vicinity of the site is a mixture of residential, agricultural, with residential and commercial business present on the surrounding roadways. Residences located in the vicinity of the Site are indicated on the Site Vicinity Plan, and are primarily located along adjacent roadways, including Burgess Road, Strong Road, North Road and Route 414, as well as the Village of Waterloo.

2.1.2 Siting Criteria

6NYCRR Part 360 and 363-5 of New York State's solid waste management facility regulations establish siting restrictions for new and expanded land disposal facilities. In accordance with the New York State Solid Waste Management Plan (SWMP), the proposed action will provide added landfill capacity to the SMI Landfill in a timely fashion in order to provide critically needed solid waste disposal services locally and for the State. The SMI Landfill currently accepts the largest amount of waste in the State, and is of both local and statewide significance to the flow of waste in New York. The most recent NYSDEC solid waste management plan (titled Beyond Waste – a Sustainable Materials Management Strategy for New York State, adopted December 2012) recognizes the favorable features of continued landfill operations at existing sites because they use existing infrastructure, are more efficient and less expensive to design and construct, and generally impact fewer natural resources than new landfill sites.

In addition to the New York State SWMP the SMI Landfill is located in Seneca County, where there is no local solid waste management plan in effect. Seneca County was formerly part of the Western Finger Lakes Solid Waste Management Authority (WFLSWMA) Planning Unit that also included Wayne and Yates counties as members.

Ontario County was originally a part of the WFLSWMA, but subsequently withdrew. The WFLSWMA has since been disbanded and the planning unit no longer exists.

The following is a review of 6NYCRR Part 360 and 363-5 siting criteria, in relation to the proposed landfill.

2.1.2.1 Prohibited Siting

6NYCRR Part 360.8 requires that solid waste management facilities must not be located in the following areas:

Special Flood Hazard Areas

Part 360.8(a) states that persons must not construct a new facility or expand an existing facility, in a special flood hazard area, unless provisions acceptable to the Department have been made to prevent flooding of the facility and to prevent the constriction of floodwaters. The facility must not pose a significant hazard to human life, wildlife, fisheries, or land or water resources.

The proposed development does not lie within the current floodplain of Black Brook. As part of the ongoing expansion, Black Brook, which formerly traversed the site in an east-west alignment was relocated and realigned to the north of the site. The realignment of Black Brook which was completed from 2011 to 2014, along with the planned construction of the landfill perimeter berms on the west side of the site, resulted in a comprehensive re-definition of flood boundaries. A request for a Conditional Letter of Map Revision (CLOMR) was submitted to FEMA by the Town of Seneca Falls for the brook realignment in support of the realignment project. Until the final letter of map revision is submitted by the town (which can now be completed as a result of the recent completeion of the Stage 5/6 perimeter berms), the current flood plain mapping indicates that portions of the landfill site are within the 100-year flood plain limits. However, the CLOMR documentation cleary indicated the existing landfill footprint and that proposed as part of the SMI Valley Infill project will not be within the flood plain limits.

Threatened or Endangered Species

Part 360.8(b) states that persons must not construct a facility or laterally expand an existing one in a manner that causes or contributes to the taking of any endangered or threatened species or to the destruction or adverse modification of their critical habitat. The Draft Environmental Impact Statement (DEIS) submitted for the SMI Valley Infill project permit application addresses this regulation, and some discussion is provided in this Engineering Report as well.

The Project Area consists exclusively of land that has been previously disturbed by the construction and operation of the Facility and by the operation and remediation of the Tantalo Waste Disposal Area (Tantalo Site).

No natural vegetation communities will be disturbed in connection with the construction of the landfill development. The Project Area footprint consists of cleared bare land, developed industrial land, and closed or inactive landfill areas. Vegetation in these cleared and developed areas is very limited to occasional common weeds and grasses that are typical of disturbed areas. Vegetated areas that are part of the final cover system are periodically mowed to prevent the growth of trees or other deep-rooted vegetation. No State and/or federal-listed endangered, threatened, special concern, rare or exploitably vulnerable fauna are known to occur in the Project Area.

The Indiana bat has been found to occur in the general vicinity of the project, but this species roosts and forages in areas with deciduous trees. The proposed development area consists of cleared bare land, developed industrial land, and closed or inactive landfill areas that include no trees or natural vegetation that would represent potential roosting and/or foraging habitat for the Indiana bat. Accordingly, the Project will not adversely affect this species. A 2006 study by BHE Environmental, Inc. supported this conclusion.

No other rare, threatened or endangered species have been reported for the site by the NYNHP. There are two such species listed by NYSDEC in the general vicinity, the bald eagle and the osprey, but both are reported for areas in and immediately adjacent to the Montezuma National Wildlife Refuge. None were observed on or in the immediate vicinity of the Project Area, and none are likely to utilize the Project Area because no suitable nesting or feeding habitats for either of these species are present.

Federally Regulated Wetlands

Part 360.8(c) states that persons must not construct a new facility or laterally expand an existing one within the boundary of either state or federally regulated wetlands, unless required permits are obtained from the U.S. Army Corps of Engineers and/or the department.

The Project Area consists exclusively of land that has been previously disturbed by the construction and operation of the Facility and by the operation and remediation of the Tantalo Waste Disposal Area (Tantalo Site).

The landfill development will not be constructed on a state or federally regulated wetland. SMI currently maintains an Article 24 Freshwater Wetlands Permit from the NYSDEC in conjunction with a Federal Wetlands Permit from the U.S. Army Corp of Engineers as a result of the Wetlands Mitigation Project which developed approximately 585 acres into a wetlands mitigation area. Further discussion of the SMI Valley Infill project in relation to wetlands is provided in the Draft Environmental Impact Statement (DEIS).

Facilities at or near Sites Undergoing a Remedial Program

Part 360.16(f)(1) states that for facilities proposed to be located within 150 feet of the boundary of a site undergoing a remedial program, the applicant must submit a report that discusses the potential impacts of the facility on the remedial program for that site. Additionally, the proposed facility must not interfere significantly with potential, ongoing or completed remedial program.

The proposed landfill development will be located within the limits of the Tantalo Site and therefore is subject to the requirements of this part. The remediation of the Tantalo Site has been completed with ongoing monitoring as per NYSDEC requirements for the Class 4 site. In accordance with 6NYCRR Part 360.16 (h)(1), the potential impact of the proposed development on the Tantalo remedial program is discussed in Section 2.1.2.2 with respect to each operable unit. As detailed in Section 2.1.2.2, the proposed development would not interfere significantly with potential, ongoing or completed remedial actions associated with the Tantalo Waste Disposal Area.

2.1.2.2 Assessment of the Proposed Development on the AB Landfill and Tantalo Waste Disposal Area Remedial Actions

The former AB Landfill and Tantalo Waste Disposal Area represent Class 4 Inactive Hazardous Waste Disposal Sites for which remedial actions are either on-going or have been completed. The proposed SMI Valley Infill represents an overfill of the Tantalo Site and a portion of the former AB Landfill, and in accordance with 6NYCRR Part 360.16(h)(1), the following discusses the potential impact of the proposed development on the remedial programs. Notably, both of these areas were addressed as part of the 2006 application (See Appendix G of the 6NYCRR Part 360 Permit Application, Seneca Meadows Landfill Expansion, 1,9(g) Report) and with the exception of the need to abandon and replace some existing monitoring or injection well locations, the conclusions reached as part of the 1,9(g) report remain consistent. The report concluded that the proposed development will neither interfere significantly with potential, ongoing, or completed remedial program at the Tantalo Waste Disposal Area or former AB site, nor expose the environment or public health to a significantly increased threat or harm. Remedial actions and potential impacts associated with each of these areas are discussed below.

Former AB Landfill

The former AB Landfill is comprised of the southernmost area of the existing SMI Landfill and is located just north of the Tantalo Waste Disposal Area. Remedial actions associated with the former AB Landfill included the relocation of waste into lined areas of the landfill, placement of additional cover materials, and grading and construction of surface water diversion swales to address surface water drainage at that time. In addition, limited (areal extent) groundwater impacts were addressed through the construction and operation of the SM-11 drain system. These actions were completed during the 1993, 1994 and 1996 construction seasons and groundwater monitoring subsequent to these actions documented considerable improvement in groundwater quality south (downgradient) of the former AB Landfill.

The former AB Landfill was part of the 1998 Landfill Expansion area approval and has been overlain by a liner and leachate collection system for construction of the A/B Overfill. The former AB Landfill is thus isolated from the overlying waste and recharge by the liner system. Operation of the SMI-11 Drain system demonstrated that the drain is functioning as designed and as part of the 2006 application, continued operation of the western portion of the SM-11 Drain was proposed. The infrastructure associated with current/on-going operation of the SM-11 Drain System is under construction and will not be impacted by the proposed SMI Valley Infill.

As stated above, operation of the SM-11 Drain System will continue and will not be impacted by the proposed SMI Valley Infill. Therefore, the conditions and assessment presented in the 2006 1.9(g) report remain valid and the proposed SMI Valley Infill will not have an impact on the remedial program.

Tantalo Waste Disposal Area

The Tantalo Waste Disposal Area is located between the WEX and the SELF, and to the south of the existing SMI and former AB Landfill discussed above. The proposed SMI Valley Infill will directly overlie the Tantalo Waste Disposal Area which is being addressed under two Operable Units, as presented in the August 2019 Periodic Review Report and Combined Annual Report No.7 by Cornerstone. Operable Unit 1 (OU-1) deals with the waste disposal site area, the remedy for which included a cap, leachate/groundwater collection system, landfill gas venting system, cutoff wall, and waste consolidation along with other supporting activities (e.g., access road relocation, drainage controls, etc.), and deed restrictions. The NYSDEC deemed this work complete as of August 9, 2007.

OU-2 deals with bedrock groundwater and the selected remedy involves enhanced natural bioremediation by the injection of electron donor materials into the bedrock aquifer to enhance the naturally occurring biodegradation of TCE and its degradation products through reductive dechlorination. A performance monitoring plan is in place to document performance of the remedy and to monitor perimeter sentinel wells.

The site is currently in the operations, maintenance and monitoring phase for both operable units and annual Periodic Review Reports (PRRs) are submitted to NYSDEC documenting the performance of the remedial actions. The most recent PRR was submitted in August 2019 and documented that performance and effectiveness of the remedy are being maintained and that there were no recommended changes to the program. The potential impact of the proposed development on the remedial programs for each operable unit are discussed below.

Operable Unit 1

As noted above, OU-1 included, among other ancillary activities, a cap over the Tantalo waste mass, a leachate/groundwater collection system, a landfill gas venting system, and a shallow permeable barrier at the north end of the waste mass. Since completion, the PRRs have shown that the remedial actions have been effective in that leachate generation rates have been asymptotically declining, albeit with temporary increases due to adjacent construction activities that increased infiltration. Overfill of the Tantalo waste mass would further enhance the cap (i.e. a double composite liner and leachate collection system would be constructed over the existing cap) and would also eliminate the unlined areas surrounding the waste mass where most of the infiltration is occurring due to rainfall and associated stormwater. The construction of a double composite liner and leachate collection system over the Tantalo waste mass, the surrounding unlined areas, and up the slope of the adjacent solid waste management cells to the north, west and east, as detailed in the Engineering Plans, would thus further reduce infiltration and make both the existing cap and leachate/groundwater collection system associated with the Tantalo Site obsolete. Likewise, the shallow permeable barrier at the north end of the Tantalo Site that was completed as part of the remedy was intended to reduce surface water infiltration to the leachate/groundwater collection system. The significance of the barrier was reduced when Black Brook was relocated so that it no longer flows immediately north of the Tantalo Site, and its utility will be further reduced to obsolescence following construction of the double composite liner system for the Valley Infill as described above. In addition, as discussed within this hydrogeologic report and within the PRRs, groundwater flow through the overburden deposits is primarily vertical, and with the elimination of infiltration around the perimeter of the Tantalo waste mass, groundwater flow within the surrounding area will be significantly reduced.

The further reduction in infiltration and the vertical flow system also relates to the groundwater monitoring associated with the OU-1 remedial action. Specifically, this included the collection of groundwater samples from overburden wells T-1LL, T-1LT, T-1UT, T-15UL and T-16UTR. As presented in the August 2019 PRR, recent sampling of these wells has shown that the number of VOCs detected above NYSDEC Part 703 groundwater quality standards has declined from eight to two and there are numerous occurrences of non-detectable concentrations. This further supports the effectiveness of the cap and the conclusion that reduction in infiltration associated with construction of a double composite liner would be of added benefit.

In summary, the OU-1 remedial actions have been successful in reducing recharge through the Tantalo waste mass. Further reduction of infiltration, most significantly around the perimeter of the Tantalo Site, will result from the construction of the double composite liner and leachate collection system over the Tantalo waste mass and the surrounding unlined areas. The lining system will eliminate stormwater flow and associated infiltration in this area. The double composite liner would be constructed over the existing cap, thus increasing its effectiveness, and would eliminate the need for ongoing maintenance and monitoring of the existing cap. The double composite liner system would also be constructed over the currently unlined perimeter areas where the monitoring wells are located and would thus require abandonment of these monitoring wells. However, given the primarily vertical flow system within the overburden, these wells are already of limited value (see August 2010 PRR for additional discussion) and would be of even more limited value with further reduction in infiltration. Therefore, abandonment of these wells would not materially affect the monitoring of the OU-1 remedial action and groundwater monitoring can and will be conducted at the downgradient perimeter consistent with the surrounding SMI facility.

The development of the SMI facility over the Tantalo Waste Disposal Area would therefore have a positive impact on the OU-1 remedial actions and serve as an enhancement of the remedial actions previously completed and shown to be effective. It should be noted that construction activities will require removal of the current cap for subgrade development of the proposed SMI Valley Infill. The cap will be removed in phases for construction of the double composite liner system. As such, there will likely be a period of increased infiltration during construction activities, however this will be of limited duration and with anticipated minimal effects. Bedrock removal is discussed further in Section 3.4.2.2.

Operable Unit 2

As stated above, OU-2 deals with bedrock groundwater and involves enhanced natural bioremediation by the injection of electron donor materials into the bedrock aquifer and associated performance monitoring. Injection of electron donor materials are completed at injection wells (designated with the prefix “IN-“) located near the southern end of the Tantalo Site, with the last injection completed in 2013/2014. Performance monitoring is conducted at select injection and performance monitoring wells, a series of “inner” and “outer” tier monitoring wells, and at select Part 360 monitoring wells associated with the SMI facility as listed in Table 6-1 and shown in Figure 5-3 (Taken from PRR). Performance monitoring has and continues to demonstrate that injection of the electron donor material has had a beneficial impact on groundwater quality and the plume has retracted in comparison to historical mapping completed in 2005.

With the exception of the SE-100SBR and SE-100DBR well cluster, the inner and outer tier monitoring wells are beyond the limits of the proposed SMI Valley Infill area and would therefore not be impacted. These two wells would require that their casings be extended, but they would remain in place and be available for monitoring. However, some of the injection and performance monitoring wells are within the proposed development area and would therefore require abandonment and/or replacement as the proposed development progresses. Wells IN-4S, IN-4D and T-1SB are located within the first proposed phase and would therefore be the first to be abandoned (see Operations Plan sheets in the Engineering Report). Replacement of these locations would not be feasible. Wells IN-3S, IN-3D, IN-2D and IN-1 are located within the proposed berm to be constructed as part of the first phase and would thus require extension during the construction process or abandonment and replacement at a proximal location. The next well required for abandonment would be WEX-214SB. However, abandonment would not be required until construction of proposed Phase 4 and this location would remain in use until that time. Wells IN-6S, IN-6D and WEX-213SB would require abandonment upon construction of proposed Phase 5 and these locations would also remain in use until that time. Replacement of these locations would not be feasible. Finally, IN-5S and the office well would require abandonment during construction of the final proposed (Phase 6). Wells requiring abandonment in Phases 5 and 6 could be replaced with new locations constructed within the berm. In summary, three injection wells, IN-4S, IN-6S and IN-6D and two monitoring wells T-1SB and WEX-214SB would require abandonment and could not be replaced with a nearby location. Note that T-1SB and WEX-214SB are Part 360 monitoring wells that are not included in the OU-2 monitoring program. However, data from these locations are used to help define the Tantalo plume limits. The remaining injection and monitoring wells noted above could have their casing extended and remain in place or be abandoned and replaced with a nearby location.

Construction of the proposed SMI Valley Infill would thus impact the ability to conduct further injections at IN-4S and IN-4D. However, based on the on-going performance monitoring program, these wells have not been used since the initial injection event in 2007 and the data collected as recently as 2019 does not indicate the need for injection of additional material at these locations. The ability to conduct additional injections would also be impacted at locations IN-6S and IN-6D. However, these locations would not require abandonment until Phase 6 and would be available until that time. IN-6D has not been used since the initial injections in 2007 and IN-6S was last used during the third injection event in September 2013. The remaining injection wells, or their replacement, would remain accessible for additional injections and are located to the south of the Tantalo waste mass.

The loss of IN-4S and IN-6S would limit the ability to inject donor within the northern, upper portion of the bedrock high (bedrock knob) underlying the Tantalo Site. However, infiltration to this area will be significantly reduced by construction of the double composite liner system, thus reducing groundwater flow volumes and associated vertical and radial flow from this area. Groundwater flow from these areas will continue to migrate radially and vertically downward into the underlying deep bedrock water-bearing zone and then to the south where the additional injection and monitoring wells are located or to monitoring wells associated with the SMI SELF or WEX areas. While considered unlikely, additional injection wells could be installed at these locations in the future if deemed necessary.

As discussed in the PRR, one objective of the injections was to address the potential that implementation of the OU-1 remedy (i.e. capping of the waste mass) might reduce the volume of leachate from the Tantalo waste mass (ie. source area) that serves, or served, as a source of nutrients/electron donor to facilitate biological activity and promote reductive dechlorination in the underlying bedrock water-bearing zones. As discussed above, construction of a double composite liner system would further reduce infiltration and leaching from the waste mass. However, since the waste mass is already capped, the bulk of the reduction in infiltration would be associated with the unlined area surrounding the Tantalo Site where oxygen rich stormwater infiltrates. The result, therefore, would be to reduce the volume of oxygenated water infiltration, which would further promote reducing (i.e. low oxygen) conditions, which are conducive to reductive dechlorination.

In summary, while development of the proposed SMI Valley Infill would require the abandonment of four injection wells (IN-4S, IN-4D, IN-6S and IN-6D) potential impact on the OU-2 remedial program is not significant as existing injection wells within the deep bedrock water-bearing zone will remain in place and available, and additional injection wells could be installed to the west and east of the existing locations if needed. Similarly, while some monitoring wells will be abandoned, there are multiple existing monitoring wells already in place and currently monitored that can serve to document the performance and effectiveness of the OU-2 remedial action. Finally, construction of a double composite liner associated with the proposed development, and the subsequent reduction in fresh (i.e. oxygenated) water may further promote the desired reducing conditions.

The development of the SMI facility over the Tantalo Waste Disposal Area would therefore not have a significant impact on the OU-2 remedial action as currently implemented and monitored. Existing monitoring wells can be used for on-going monitoring and additional injection wells could be installed if the monitoring data indicates a need. Therefore, construction of the proposed development area would not impact the ability to meet the objectives of the OU-2 remedial action. It should be noted that construction activities will require removal of the current cap for subgrade development of the proposed SMI Valley Infill. The cap will be removed in phases for construction of the double composite liner system. As such, there will likely be a period of increased infiltration during construction activities, however this will be of limited duration and with anticipated minimal effects.

Part 375 – Change in Use

6NYCRR Part 375-1.11(d) requires that notification be submitted to the NYSDEC of a “Change of Use” that would explain how such a change, in this case, construction of the proposed SMI Valley Infill, may affect the sites proposed, ongoing or completed remedial programs. As discussed above, and as also supported by the 2006 1.9(g) Report, the proposed development will not negatively affect the ongoing or completed remedial programs. In fact, the construction of a double composite liner system over the top of the Tantalo Site will further reduce infiltration through the current cap and significantly reduce infiltration of oxygenated stormwater around the perimeter of the Tantalo Site. The SMI Valley Infill thus has the potential to enhance the OU-1 remedial actions and may also enhance the OU-2 remedial actions by reducing the influx of oxygenated water. Finally, the minor negative impact associated with the loss of current injection and monitoring well locations associated with OU-2 can be overcome through monitoring of other existing monitoring wells and the installation of additional injection wells, if needed.

Likewise, the proposed SMI Valley Infill will not affect continued operation of the SM-11 Drain System that is associated with the former AB Landfill. Therefore, this system can continue to operate as currently approved and the SMI Valley Infill will have no impact. A change in use has already been submitted and approved for the AB Landfill during the construction of the AB Overfill. As such no further notification for the AB Landfill will be required for this development.

2.1.2.3 Landfill Siting Restrictions

6NYCRR Part 363-5.1 requires landfill Development sites to comply with the following criteria enumerated below.

Groundwater Flow

Part 363-5.1(a) states that sites underlain by bedrock subject to rapid or unpredictable groundwater flow should be avoided unless it can be demonstrated that a containment failure would not result in contamination entering the bedrock groundwater system. Also, sites in proximity to mines, caves or other anomalous features that may alter groundwater flow are to be avoided.

The bedrock flow system at the SMI site is well understood as a result of the extensive quantity of data obtained through past investigations. Flow within the bedrock is predominantly vertical beneath the bedrock knob until being caught up in the regional horizontal flow direction to the south. The available hydraulic head data for the site do not suggest unpredictable groundwater flow patterns exist on-site. There is no convergence of head data, nor does reconnaissance of the site indicate the presence of insurgent streams or sinkholes that would be indicative of potential preferential conduit systems beneath the site. The Hydrogeologic Investigation Report includes the data to support these conclusions. As a result, these criteria do not represent a siting restriction for the development.

Character and Thickness of Unconsolidated Deposits

In accordance with Part 363-5.1(a)(2), the minimum separation distance of ten feet to bedrock will be maintained. Given that the overburden deposits marginally meet the minimum ten foot thickness criteria near the southern end of the Tantalo Site, and the need for subgrade development as part of the SMI Valley Infill, the natural overburden soils in this area will be excavated, bedrock will be removed, and recompacted soils will be placed to a minimum thickness of ten feet, consistent with the work in WEX Stages 5 & 6.

Agricultural Land

Part 363-5.1(c) states that new or expanded solid waste facilities must not be located on land that:

- Was, or is proposed to be, taken through the exercise of eminent domain;
- Consists primarily of agricultural soil group 1 or 2; and,
- Is within an agricultural district pursuant to Agriculture and Markets Law.

Since the property is not proposed to be taken through the exercise of eminent domain, this criterion does not apply. Furthermore, agricultural soil groups 1 or 2 are not present in the proposed facility development area, and the site does not lie within an agricultural district. As a result, this prohibition does not apply to the proposed landfill.

Primary Water Supply, and Principal Aquifers

Part 363-5.1(d)(1) states that no new landfill or development of an existing landfill can be constructed over a primary water supply aquifer, principal aquifer, within a public water supply stabilized cone of depression area or within a distance of 500 feet from surface waters that are actively used as sources of municipal water supply.

The property is not within a primary aquifer, principal aquifer or public water supply wellhead area. The absence of lateral flow within the overburden further supports the conclusion that the SMI site is not underlain by or tributary to a primary or principal aquifer and prior applications for expansion of the facility (i.e. the SELF and WEX development areas that surround the SMI Valley Infill development area) were approved based on this understanding. The closest surface water body is Black Brook. This surface water feature originates west of the site and flows in an easterly direction as it traverses SMI's property north of the existing landfill. Black Brook is a small, channelized, intermittent stream, which discharges to the Tschache Pool at the Montezuma Wildlife Refuge approximately seven miles downstream of the site. Black Brook is not used as a source of municipal water supply.

Horizontal Separation

Part 363-5.1(d)(2) states the required horizontal separation between deposited waste and primary water supply aquifers, principal aquifers, capture zones of public water supply stabilized cone of depression areas or surface waters that are actively used as sources of municipal drinking water supply must be sufficient to preclude contravention of groundwater standards in aquifer and surface water standards in waters that are currently used as a source of municipal drinking water supply.

The proposed limits of the development area will be greater than 1.8 miles from the closest water body, which is the Seneca-Cayuga Canal, the canal is not a known drinking water source, and the Seneca Falls water intake that is located in Cayuga Lake is greater than 6 miles downstream from the site. The site is not located over a primary or principal aquifer and the soils underlying the site do not yield sufficient quantities of groundwater to meet the definition of a primary or principal aquifer. Because of the above conditions, the horizontal separation distance between the proposed facilities and any controlled lake or reservoir does not represent an impediment to the project.

Aircraft Safety

The regulations contain various criteria regarding facility height and the proximity of putrescible waste landfills to airports. As identified by 6NYCRR Part 363-5.1(e) an airport is a facility open to the public without prior permission and without restrictions within the physical capabilities of available facilities; and an active military airfield. The aircraft safety criteria specified as follows:

I. A landfill or landfill subcell into which putrescible solid waste is to be disposed must be located no closer than 5,000 feet from any airport runway end used by piston-powered fixed-wing aircraft and no closer than 10,000 feet from any airport runway end used by turbine-powered fixed-wing aircraft.
II. A landfill or landfill subcell into which putrescible solid waste is to be disposed, which is located within five miles of any airport runway end, must not, in the opinion of the Federal Aviation Administration, pose a potential bird or obstruction hazard to aircraft.
III. The permittee of an existing landfill or landfill subcell that is authorized to dispose of putrescible solid waste and that is located less than 10,000 feet from any airport runway end used by turbine-powered fixed-wing aircraft or less than 5,000 feet from any airport runway end used only by piston-powered fixed-wing aircraft must provide in its permit renewal application documentation that the Federal Aviation Administration believes the landfill or landfill subcell does not pose a bird hazard to aircraft.
IV. Landfills containing only nonputrescible solid waste may be located less than 10,000 feet from any airport runway end used by turbine-powered fixed-wing aircraft or less than 5,000 feet from any airport end used by piston-powered fixed-wing aircraft, if the Federal Aviation Administration has determined that the landfill will not present a hazard to air traffic.
V. The final elevation of a new landfill or expansion of an existing landfill must not extend more than 200 feet above the highest elevation of the land surface that existed prior to landfill development, unless the Federal Aviation Administration believes that the proposed fill height in excess of 200 feet will not present a safety hazard to air traffic.

The FAA's Sectional Aeronautical Charts and local maps were reviewed to evaluate the minimum distance requirements specified by the regulations. A small landing strip identified east of Route 414 and the old Waterloo airfield in the easternmost portion of the Village of Waterloo do not pose siting issues since these airstrips are either inactive, not registered in the New York State Airport Directory and/or do not have air clearance with the FAA.

The nearest public use airport within five miles of the proposed facility is the Finger Lakes Regional Airport on Martin Road in the Town of Seneca Falls. On review of the USGS's Seneca Falls Quadrangle map, the end of the runway at the Finger Lakes Regional Airport is approximately 17,500 feet southeast of the site. This distance is greater than the minimum specified by the regulations as a prohibited area, and does not require that SMI obtain an opinion from the FAA regarding the potential hazards to aircraft. As well, the final elevation of the proposed landfill will extend more than 200 feet above the land surface at the site. In accordance with the requirements of 6NYCRR Part 363-5.1(e)(5), the Federal Aviation Administration (FAA) has reviewed the plans for the facility and made a determination that a fill height in excess of 200 feet will not pose a safety hazard to air traffic.

An electronic submittal was made to the FAA on November 14, 2019 in accordance with FAA requirements for the above determination regarding safety hazard. Based on this submittal, the FAA made a determination, on

January 10, 2020 (2019-AEA-13102-OE) that the proposed construction would not pose a hazard to air navigation. The FAA determination is subject to the condition that marking or red lighting be installed in accordance with FAA Advisory Circular 70/7460-1 L, Change 2. The FAA determination expires on July 10, 2021, but may be renewed. Additional correspondence with the FAA (also included in Appendix G of this report) via email, indicated that the proposed SMI Valley Infill project meets the requirements for siting facilities near airports as it relates to potential bird or wildlife hazard to aircraft.

The determination from the FAA including the submitted forms are included as part of Appendix G of this report.

Unstable Areas

Part 363-5.1(f) states that landfills must not be located in unstable areas where inadequate support for the structural components of the landfill exists. Additionally, a demonstration must be made that the landfill is properly engineered to ensure adequate support for the structural components and additional loads. The geotechnical properties of geologic media underlying the site have been well established through various subsurface investigations and laboratory analyses performed during past and recent investigations in support of this and other facility developments as well as specific project-related investigations. Conservative strength parameters were assigned to the subsurface soils for drained and undrained conditions based on laboratory triaxial compression shear testing for the on-site soils. The condition of these soils varies across the site so proposed remedies at certain parts of the facility are expected to be somewhat different. The Geotechnical analysis included in this report has taken this into consideration and describes how the facility layout provides the necessary factors of safety for facility construction and operation

The stability analysis was performed for short term and long term conditions to simulate operational and post closure periods. Stability analyses for representative landfill sections, including those that evaluated the effects of increased waste being placed over existing overfill liners, indicated failure surfaces associated with the developed landfill(s) showed factors of safety greater than 1.5.

A bearing capacity analysis shows factors of safety in excess of 2 for drained and undrained conditions for the landfill. The stability analysis concludes that the subsurface soils in the landfill development areas posses adequate strength to achieve acceptable factors of safety.

Monitorability/Remediability

Part 363-5.1(g) states that new landfills must not be located in areas where environmental monitoring and site remediation cannot be conducted.

The ability to perform monitoring and site remediation is based on a sufficient characterization of groundwater and surface water flow that allows the proper identification of upgradient and downgradient locations. In turn, this allows the proper placement of environmental monitoring points which will detect and define a release from the landfills, including the identification of source areas. This information then would form the basis of selecting an appropriate corrective action which would mitigate a release. Through prior development of the various phases of the existing site monitorability of this expansion has been established. Additionally, the remediation of the Tantalo Waste Disposal Area has adequately demonstrated the remediability of the site.

Fault Areas

Part 363-5.1(h) states that new landfills or lateral developments of existing landfills are not to be located within 200 feet of a fault that has experienced displacement in Holocene time, unless the owner can demonstrate an alternative setback distance of less than 200 feet will not result in damage to the structural integrity of the landfill and will be protective of human health and the environment. No faults are mapped on the map titled Geologic Map of New York, 1970, Hudson Mohawk Sheet within 200 feet of the project limits. There are no Quaternary faults (which include the Holocene time) mapped on the USGS interactive mapping tool.

Seismic Impact Zones

Part 363-5.1(i) states that new landfills and lateral developments are not permitted in seismic impact zones unless certain demonstrations regarding the structural integrity of the landfill are made.

A seismic impact zone is defined as an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years as delineated on the most current version of USGS Survey Map or as delineated on another approved source. Based on correspondence from the NYSDEC approving the use of the most current seismic hazard maps posted on the USGS seismic hazard mapping web page, a review of the USGS map entitled *Two-percent probability of exceedance in 50 years map of peak ground acceleration*, 2014 (which is accepted as equivalent to 10% in 250 years) and associated data indicates the site's maximum horizontal acceleration is 0.071g. As the mapped acceleration does not place the site in a seismic impact zone, the design does not require seismic analysis. The seismic impact zone map and lateral acceleration graph is included in Appendix G of this report.

2.1.3 Historical Landfill Development

The Site includes areas which have previously been utilized for waste management activities, other than those at SMI's currently permitted Landfill. These areas are discussed below.

2.1.3.1 Tantalo Waste Disposal Area

Waste disposal at the Site is reported to have begun as early as 1958 along the north side of Salcman Road, in the area now known as the Tantalo Waste Disposal Area. The Tantalo Waste Disposal Area is located at the southern end of the site as shown on Sheet 5 of the Engineering Drawings, and is listed by the NYSDEC as a Class 4 Inactive Hazardous Waste Disposal Site (No. 850004).

Disposal activity at the Tantalo Waste Disposal Area site began with the purchase of property by Dorothy and Dominick Tantalo, who subsequently entered into written agreements with Stanley Sell for the co-management of the waste disposal operation. Tantalo and Sell then entered into separate written agreements to accept municipal wastes from the Town and Village of Seneca Falls and the Village of Waterloo. In addition to the disposal of municipal solid waste from the local municipalities, private companies in these municipalities and the surrounding area also utilized the site for the disposal of industrial waste streams. Principal wastes alleged to have been placed in the Tantalo Waste Disposal Area include municipal and household solid wastes, sludges, foundry sand, surfactants, liquid wastes, other industrial waste, and canning wastes.

The Tantalo Waste Disposal Area is defined by the limits of waste or residue, as determined by a test pit investigation completed during the Supplemental Remedial Investigation in August, 1995. The disposal operation, as was common practice at the time, consisted primarily of end dumping the waste material on the ground, with subsequent burning to reduce volume. The dumping and burning operation commenced near the Salcman Road frontage and proceeded north toward Black Brook as time progressed. The burning of waste materials continued until sometime in 1967 or 1968, when the Seneca County Department of Health, in response to complaints from area residents, ordered the cessation of burning and the initiation of a "sanitary landfill operation". Waste disposal continued at the Tantalo Site until sometime in 1973 or 1974.

The Tantalo Waste Disposal Area can be generally described as an elongated and relatively flat mound, approximately 600 feet wide and 2,200 feet long, oriented with its long axis north to south. The main access road to the landfill previously traversed the north to south axis of the Tantalo Waste Disposal Area, however, remediation work for the site has included relocation of the access road to the east side of the Tantalo Site. The northerly limit of waste placement had previously existed approximately 100 feet south of the southerly bank of Black Brook; however, again as part of the remediation of the site, waste in the area has been pulled back and a geosynthetic clay liner cutoff wall has been installed. The waste disposal area is then bounded to the east by undeveloped land, to the south by Salcman Road and to the west by undeveloped land. The top of the mound had reached a maximum elevation of 494 feet MSL, approximately 14 feet above the surrounding grade, prior to remediation. As a part of the site closure, the disposal area was graded to promote runoff from the cap, and the final design grade for the area will be approximately elevation 513 feet MSL. Remediation of the site is complete with ongoing groundwater monitoring of the site. The southern end of the site is developed with buildings and a large paved area, including SMI's office, NYSDEC offices, the Equipment Maintenance Shop, a Storage Barn and the Scalehouse. A portion of the waste along the southern boundary was also pulled back as a part of the site remediation, and the areas within the developed/paved portion of the site will be capped with an enhanced asphalt cap.

SMI has completed Remedial Investigations (RI), and Feasibility Studies (FS) for the Tantalo Site which were approved by the NYSDEC. Based on the results of the RI and FS work, the NYSDEC issued Records of Decision (RODs) dated March 1998 and March 2001. Designs then followed for both Operable Unit 1 (OU-1), defined as the landfill and overburden groundwater, and OU-2, defined as bedrock groundwater, respectively. At that time, the Tantalo Site was considered a Class 2 Hazardous Waste Site presenting a threat to the public health.

The remedial actions for OU-1 are detailed in the Final Design Report for Operable Unit 1, Tantalo Waste Disposal Site, Eckenfelder Engineering PC, July 2003. The Remedial Design for OU-1 was accepted by NYSDEC in August, 2003, and the design included the following:

- Removal and consolidation of waste along the northern boundary of the Tantalo Site within approximately 200 feet of Black Brook;
- Consolidation of waste along the southern boundary and along the east and west to facilitate construction of the leachate/groundwater collection system;
- Installation of a hydraulic barrier between Black Brook and the Tantalo Site;
- Leachate/groundwater collection and disposal;
- Landfill gas collection and treatment;
- Placement of fill for grading of slopes;
- Construction of a final cover system;
- Installation of an enhanced asphalt cap over existing asphalt at the southern end of the landfill; and
- Relocation of the landfill access road.

The remedial construction was deemed complete as of August 9, 2007 after URS Corporation submitted a Certification Report regarding the OU-1 remedy. Continued operations and maintenance activities for the OU-1 remedy, including leachate management, gas collection, and final cover maintenance are ongoing.

The remedial actions for OU-2 are detailed in the Final Design Report, Tantalo Waste Disposal Site, Operable Unit No. 2 (OU-2), HydroQual Environmental Engineers and Scientists, P.C., December 2005 and subsequent letter of April 3, 2006. Remedial actions relative to OU-2 include the following:

- Installation of nine wells for the injection of both fast acting (e.g., lactic acid or similar material) and long acting (e.g., emulsified edible oil) to promote natural bioremediation of the VOCs within bedrock groundwater (i.e., enhanced, monitored natural attenuation);
- Installation of additional bedrock monitoring wells that will become part of the groundwater monitoring program;
- Implementation of an Operation, Maintenance and Monitoring (OM&M) Plan that will allow for monitoring the effectiveness of the remedy and continued monitoring of the nature and extent of the plume;
- Monitoring of sentinel wells beyond the limits of the plume;
- Development of a contingency plan for evaluation of alternative remedies in the event that monitoring indicates that the selected remedy does not adequately control the nature and extent of the plume;
- Establishment of a deed notice for affected SMI properties;

- Notification requirements to keep the public and local officials informed.
- Implementation of the OU-2 remedial action is completed.

In addition, Implementation of the OU-2 remedy began in 2006 and 2007. A Remedy Completion Report was completed in June 2008 and approved by NYSDEC in August 2008. At that time, the hazardous site was considered to be under control and under successful remediation thus far, and in May 2009 the Tantalo Site was reclassified to a Class 4 Inactive Hazardous Waste Site with continued monitoring and environmental controls. The last injection event for OU-2 occurred in 2014, with continued monitoring indicating evidence of a retracting groundwater plume and a decrease in overall contaminate concentrations. While the Tantalo Site is considered properly controlled, monitoring and maintenance operations for the Site continue.

2.1.3.2 Borden, Inc. Wastewater Lagoons

In the area east of the northerly third of the Tantalo Waste Disposal Area three surface lagoons were constructed, reportedly in 1970, for the purposes of treating food processing waste. This parcel was formerly owned by the Greenwood Cannery/Borden, Inc. with a local operation in the Village of Waterloo, and this waste treatment operation continued until approximately 1979. In the early 1980's, the northern most lagoon was filled in and the southern lagoons were used as agricultural ponds to irrigate adjacent farmlands. SMI petitioned the State to regrade and drain the ponds in 1989. General Testing Corporation of Rochester, New York obtained three individual and one composite sample from each pond for analysis of hazardous substance list (HSL) organics, acid extractables/base neutrals, pesticides, PCB's and select inorganic compounds. The results of the analysis indicated only the presence of trace concentrations of inorganic compounds. Based on this data, the State permitted the draining and regrading of the ponds, which was accomplished between August, 1990 and February, 1991.

These previously abandoned lagoons were located in the area of the Southeast Landfill and are believed to have been approximately five feet deep. As the design of the Southeast Landfill called for the excavation of soils in this area, the presence of the former lagoons had no meaningful effect on the engineering performance of the facility.

2.1.3.3 Comstock Food Processing Wastes

In 1968, food processing wastes were landspread at the site in the area subsequently used as a soil borrow area. This former borrow area is located adjacent to the northwest corner of the Tantalo Waste Disposal Area. Beet process wastes generated by Comstock Foods (Division of Curtis-Burns, Inc.) of Waterloo, New York were landspread from 1968 until about 1975 when a process change ended their production of this waste material. Lemon dust consisting of diatomaceous earth used as a filter material had taken the place of the beet waste; however, the lemon waste was incorporated into the landfill operation and was not land spread.

2.1.3.4 Land Disposal North of Black Brook

By 1974, the limit of waste placement south of Black Brook had reached approximately 100 feet from the south bank of the Brook. To continue the solid waste disposal operation, Tantalo Construction Corp personnel installed twin culverts in Black Brook to allow access to its northern bank. Operations in this initial area north of Black Brook reportedly consisted of excavation to a depth of 10 to 15 feet and filling with solid wastes, with the excavated soils used as cover material. During this time period, waste received at the site consisted primarily of municipal, household, and commercial wastes; however, industrial wastes and sludges were also accepted. Because of the alleged disposal of certain industrial wastes, on February 21, 1991, the NYSDEC notified Seneca Meadows, Inc. that the landfill was included on the Registry of Inactive Waste Disposal Sites as a Class 2a site.

In February of 1991 the NYSDEC also requested that additional wells be installed to enhance the landfill's groundwater monitoring well array. Monitoring wells SM-11UGL, SM-11SB, SM-12UGL, SM-12SB and SM-2SB were installed during June 1991 for the purposes of generating additional groundwater quality data and to address NYSDEC questions regarding monitorability. A review of the SM-11 series boring logs indicates that some of these monitoring wells were installed through a thin layer of waste approximately two feet thick. After installation and development, these wells were sampled, and the initial laboratory analyses demonstrated that landfill leachate indicator parameters were present in the samples obtained from monitoring well SM-11UGL.

2.1.3.5 SM-11 System

Based on the laboratory findings and in an effort to further identify the nature and extent of the impacts, DUNN Geoscience Engineering Company of Albany, New York prepared a workplan for investigations in the vicinity of SM-11UGL. The workplan called for the installation of additional monitoring wells to determine the extent of groundwater impacts, and test pit/trench excavations to determine the extent of the previously disposed waste mass. Monitoring wells SM-11AUGL, SM-11BUGL and SM-11CUGL were installed to define the extent of groundwater impacts. The field and laboratory investigations confirmed that groundwater contamination was largely confined to an area between monitoring wells SM-11AUGL and SM-11CUGL. Additionally, 26 test pits were excavated to define the extent of solid waste placed in this portion of the inactive landfill. Once the extent of waste was determined, the edge of the waste footprint was consolidated by excavating waste to a well defined line, and disposing of excess waste in the active landfill. After waste and soil was removed during October of 1992, virgin clayey soils were used to backfill the area, which was subsequently covered with topsoil, and seeded during the spring of 1993.

A remedial design was developed to limit groundwater impacts, and this design called for the construction of a groundwater drain to intercept contaminated groundwater in close proximity to the inactive area of the landfill. This groundwater drain is comprised of a gravel filled trench in the shallow overburden which includes a six inch diameter drain pipe. Cleanouts were constructed at a location approximately one half the entire length of the groundwater interceptor trench, and dewatering sumps four feet in diameter were placed at the east and west ends of the groundwater trench. The sumps were designed to temporarily contain liquid before being pumped out using an electric submersible pump, and they are referred to as "SM-11 East" and "SM-11 West". The SM-11 East sump is now approximately 31 feet deep having an invert elevation of 454.5 while the SM-11 West sump is currently 27 feet deep having an invert elevation of 455.4. From these sumps, the trenches extend toward their respective cleanouts at a slope of 0.5%. SM-11 East was not originally designed as an essential element of the SM-11 drain, but was designated as a contingency component of the system. However, SMI proactively installed both portions of the system.

2.1.3.6 SM-11 System Performance

The SM-11 drain became operational in September 1993. In May 1995, SMI petitioned the NYSDEC to de-list the site from the Registry (Request for a Record of Decision for the Seneca Meadows Landfill, May of 1995). In response to SMI's request, and as part of the remedial process, the NYSDEC prepared a Proposed Remedial Action Plan (PRAP) in February 1996 which specified no further action as the presumptive remedy for the inactive portion of the landfill. On March 27, 1996, the NYSDEC issued a record of decision (ROD), which specified no further remedial action was required. The ROD did require the long-term maintenance of the existing final cover as well as long-term operation and maintenance of the existing leachate, groundwater, and gas collection systems, and the continuation of groundwater and surface water monitoring. The site was also reclassified from a Class 3 to a Class 4, which indicates that the site is no longer a threat to human health or the environment and is properly closed, but requires continued management.

Monitoring of groundwater quality in the area south of the former AB Site has shown considerable improvement since the implementation of the remedial measures noted above. These improvements are consistent with the operation of the SM-11 drain and consolidation of waste from the area. Results of the ongoing monitoring program are submitted as part of the Quarterly Groundwater Monitoring Reports to NYSDEC and focus on water quality data collected from the SM-11 overburden wells, SM-23UGL, and to a lesser extent, SM-19UGL.

The layout of the SM-11 System is shown on Sheet 6 of the Engineering Drawings.

2.1.3.7 AB Site Inactive Area Upgrades

A test pit exploration program was also initiated by Seneca Meadows in July 1994 along the eastern perimeter of the active and inactive portions of the landfill. This program was implemented to obtain information regarding the lateral and vertical extent of the waste along the landfill perimeter. Based on the results of the test pit investigation, a perimeter trench drain was installed along the northern and eastern portions of the landfill. This drain, consisting of a six inch diameter corrugated HDPE pipe and NYSDOT number 2 washed stone was installed along an approximate 1500 foot length of the eastern perimeter of the limits of the active landfill at that time. The drain was designed and initially operated to gravity feed to existing leachate wells for collection by vacuum pump trucks. Observation pipes were installed adjacent to leachate wells 5 and 8.

SMI continued the test pit exploration program in June 1996 to define the cover and subsurface conditions along the perimeter of the inactive area. Twelve tests pits were excavated along the perimeter of the landfill at regular intervals. At each test pit location prior to backfilling, measurements were taken at three locations along the test pit to define the shape and vertical extent of the waste mass along the inactive landfill perimeter. The bottom of excavation, bottom of waste, and top of waste at each staked location along the test pit were measured and recorded on test pit logs.

During 1997 and 1998, SMI continued the construction of the perimeter leachate collection drain along the eastern and southern perimeter of the inactive landfill. The continuation of this drain was intended to provide enhanced leachate collection within the inactive portion of the Facility and is commonly referred to as the A/B drain. The horizontal alignment of this drain was based on the outside edge of waste.

One of the most important design objectives for the A/B drain along the southerly perimeter of the landfill was to prevent the migration of contaminants from the inactive area of the landfill to Black Brook. To accomplish this in an efficient manner, the drain was placed directly in contact with a minimum thickness of four feet of solid waste, with solid waste along the southerly side of the drain location removed and consolidated into the inactive area of the landfill. The depth of the drain was set at an elevation lower than the bottom of Black Brook, such that the A/B drain serves as the deepest groundwater sink in the shallow overburden. This, coupled with its close proximity to waste, would result in leachate from the inactive area preferentially flowing to the drain rather than the Brook. This design was also expected to result in a more expedient improvement in water quality in the SM-11 groundwater drain by forming a hydraulic barrier between the waste in the inactive area and the groundwater drain.

The A/B drain is comprised of six inch diameter Schedule 80 slotted PVC pipe and drainage stone exhibiting the gradation of a number 2 stone as specified in the NYSDOT Standard Specifications for Construction and Materials. The stone is comprised of clean, sound, durable stone free from organic material and was placed over the leachate collection pipe for a total thickness of 18 inches. A separation geotextile was placed between the drainage stone and the low permeability backfill to limit the migration of fines into the leachate collection drain and reduce clogging. The low permeability backfill material was placed in maximum one foot lifts and compacted.

The drain was designed to allow leachate to gravity feed to one of three manhole sumps installed at the low points of the system. These manholes are constructed of either HDPE or concrete, depending on their location along the drain. Manholes 1 and 2, due to their burial depth (32 ft.) and location in the perimeter access road are four foot inside diameter concrete manholes. A 60 mil chemically resistant coating was spray applied to the manholes to seal against leakage and prevent chemical degradation. Manhole number 3 is a four foot diameter HDPE manhole sump installed within the older landfill footprint outside the limits of the overfill liner installed over this area. In both cases, a bentonite/clay plug was placed between the perforated pipe and the solid wall pipe to prevent leachate infiltration. The locations of the A/B drain and manholes are shown on Sheet 6 of the Engineering Drawings.

2.1.3.8 A/B (Seneca Meadows Landfill) Hazardous Site Reclassification

In June of 1994 SMI made a formal request to the NYSDEC to remove the inactive A/B site from the list of inactive hazardous waste sites, pursuant to 6NYCRR Part 375. The NYSDEC declined to delist the area, but agreed it does not present a significant threat to the environment. It was decided a more appropriate means of resolving the regulatory status of the inactive area was to obtain a Record of Decision (ROD) pursuant to Part 375. On February 16, 1996 the NYSDEC released for public review and comment a Proposed Remedial Action Plan (PRAP), and a public meeting to explain the PRAP was held on March 7, 1996. Based upon the PRAP, the NYSDEC issued a ROD on March 28, 1996. The ROD outlined the completed components of the remedy to include an engineered final cover, continued operation of the groundwater controls, leachate collection and treatment, and operation of the landfill gas collection and treatment systems. The remedy also includes the long-term operation and maintenance of these systems and the groundwater and surface water/sediment monitoring program as required under the current Landfill operating permit. With issuance of the ROD, the inactive area was upgraded to a Class 4 site. Class 4 sites are described as those which do not present a significant threat to human health or the environment, have been properly closed and require continued management.

2.2 SOLID WASTE LANDFILL AND ANCILLARY FEATURES

2.2.1 Existing Landfill Cells

There are currently six landfill areas at the Facility: the Western Expansion Landfill (WEX), the SMI Landfill with AB overfill; the Southeast Landfill (SELF); the SELF Bump-out (SBO) located just south of the SELF; Stages 7 and 8 of the 2006 expansion, also referred to as the Northern Expansion (NEX); and the Tantalo Site.

The SMI Landfill includes the older portion of the landfill, primarily located along the northern half of the current footprint and the A/B overfill liner system installed in accordance with Part 360 regulations. The A/B overfill liner includes an approximate 51 acre overfill liner system and a 3 acre lateral expansion into a previously excavated borrow area. The A/B Overfill Landfill was constructed in four stages, commonly referred to as Areas 1, 2, 3, and 4. Construction of the A/B liner systems began in 1999 and was completed in 2002. The overfill liner systems provide two separately monitored cells. The first cell primarily consists of A/B overfill liner areas 1, 2, and 4. These individually constructed liner systems were designed and constructed to tie in together to form one contiguous operating cell.

Construction of the Southeast Landfill perimeter embankment was initiated in 2000. Construction of the double composite liner system was initiated in 2002. The Southeast Landfill liner system was constructed in two “cells”, referred to as Area 1 and Area 2. Each cell is similar in area, and is configured to drain leachate to a leachate collection sump. The liner system floor is a double composite liner system, with sideslopes constructed on 3H:1V slopes consisting of a primary geomembrane liner and a composite secondary liner system.

The most recently approved landfill construction (WEX, SBO, NEX) began in 2009. The current development liner system is being constructed in 9 Stages (Stages 1-9). Each stage is configured to drain leachate to a leachate collection sump. The liner system floor is a double composite liner system, with sideslopes constructed on 3H:1V slopes consisting of a primary geomembrane liner and a composite secondary liner system.

2.2.2 Ancillary Features

The site upon which the landfill facility sits incorporates a number of ancillary facilities necessary for landfill operation and to maintain environmental compliance. The existing ancillary facilities at SMI include the following:

- Administration Building
- Scale and Scale House

- Maintenance Building
- Fueling Island
- Wheel Wash Facility
- Citizens Drop-off Area
- Stormwater Controls
- Leachate Storage and Treatment Facility
- Landfill Gas Blowers
- Landfill Gas Flares

2.3 SITE TOPOGRAPHY

The Project Area is centrally located within the glacial lake plain area of Seneca County and is within the Erie-Ontario physiographic province. The glacial lake plain is a low-lying flat area. As a result of the glacial lake plain, the topography in the Project Area and surrounding area is generally flat, with a minor localized slope to the south of the site. Current elevations of the Project Area typically range from 480 feet to 490 feet.

The most pronounced topographic relief on the Project Area itself is provided by the disposal areas, primarily consisting of the Existing Landfill with AB overfill, Southeast Landfill (SELF), Southeast Burmpout (SBO), Western Expansion (WEX) and Northern Expansion (NEX). When completed, the Proposed Landfill will have a peak elevation of 843.5 feet above mean sea level (MSL). Most natural relief is provided by Black Brook which has been relocated north of the site, and a topographic high in the south-central portion of the site.

2.4 GEOLOGY AND HYDROGEOLOGY

The following section outlines the geological and hydrogeological setting of the landfill for the purposes of integrating these characteristics with the Engineering Design of the landfill. A more extensive exploration of the geological and hydrogeological conditions and suitability of the Site can be found in the Hydrogeologic Report, submitted as Appendix D of this report for the SMI Valley Infill project permit application.

2.4.1 Site Geology

The site lies within a glacial lake plain area typified by low permeability silt and clay deposits. This is evident by the relatively flat topography of the surrounding area and the prevalent silts and clays identified in the borings completed throughout the site. The geology of the Site consists of low permeability, unconsolidated, glacially derived deposits underlain by bedrock. The thickness of the overburden soils at SMI ranges from approximately 10 to 70 feet and are thinnest near the southern end of the proposed landfill area and the south central end of the western area. This area corresponds to a bedrock high commonly referred to as the “bedrock knob”, discussed further below. Throughout most of the proposed landfill area, the overburden thickness is on the order of 40 to 50 feet thick and there are no bedrock outcrops.

2.4.1.1 Overburden Geology

The individual stratigraphic units comprising the overburden are described individually below, in their order of occurrence from the ground surface to the top of bedrock.

- Upper Glaciolacustrine (UGL)
    - The uppermost stratigraphic unit at the site is the UGL. This unit is not present near the southern end of the Tantalo Site near Salcman Road. Elsewhere across the site, the thickness of the UGL ranges from 5 to 20 feet.
    - The upper-portion of the UGL can generally be described as a yellowish-brown silt and clay, in areas where the UGL is thicker than 10 feet, and as a reddish-brown varved lean clay with frequent silt partings at greater depth.

- Upper Glacial Till (UGT)
    - Underlying the UGL is the UGT, which is continuous across the site. The UGT ranges in thickness from approximately 3 to more than 35 feet. The UGT is generally a soft to stiff red-brown sandy silt and clay, containing a trace of fine to coarse gravel.

- Lower Glaciolacustrine (LGL)
    - This unit is absent beneath much of the Tantalo Site, along Salcman Road, and along a narrow band extending to the north under the Facility. The LGL is generally a thin unit over most of the site with thicknesses generally in the 0 to 10 foot range. However, this unit thickens significantly south of the historical intersection of Salcman Road and Route 414, where the LGL approaches 60 feet in thickness.
    - The LGL varies in character consisting of brown to gray-brown, very thinly to thinly varved lean clay, silt or silty sand throughout the majority of the site. Thicker layers of very fine sand or silty sand are present in the area south of the intersection of Salcman Road and Route 414, where the LGL thickens.

- Lower Glacial Till (LGT)
    - Immediately overlying the bedrock is a low permeability, lodgment till identified as the LGT. The LGT is absent near the southern portion of the Tantalo Site and the area immediately to the southwest. Elsewhere across the site, thicknesses of this unit generally range from 5 to 15 feet, with the thickest deposits near the north end of the Facility. The LGT is generally a dense to hard, sandy clay or clayey sand with gravel, and is brown to gray-brown in color.

2.4.1.2 Bedrock Geology

Bedrock elevations beneath the site range from approximately 410 to 484 feet. The highest elevations are encountered in the vicinity of monitoring well cluster T –22 southwest of the Tantalo Site along Salcman Road. This localized bedrock high is known as the “bedrock knob” and represents the highest elevation at the SMI site. The bedrock surface slopes radially downward from this high toward the lowest bedrock elevations observed to the southeast at location T-32 and to the northwest at location SM-13 (approximately 410 feet).

The bedrock beneath the site consists of the Onondaga Limestone and Bertie/Cobleskill Formations. Based on detailed analysis of the rock cores, local bedrock outcrops and literature reviews, the Onondaga Limestone is present as the uppermost bedrock unit in the southern portion of the site, particularly associated with the bedrock knob. Elsewhere across the site, the Bertie/Cobleskill Dolostone is the uppermost bedrock formation. While visual differentiation of these two units is difficult due to their similarity, this interpretation is consistent with the regional model that indicates the regional northerly terminus of the Onondaga Limestone occurs at or near the site.

Specifically, the location of the Onondaga Limestone, as found on the site, correlates well with the location of the northern areal limits of the Onondaga as reported by Richard and Fisher (1970).

The Onondaga Formation in New York State is a carbonate unit consisting of Middle Devonian marine limestones (Lindemann and Feldman, 1981). The Onondaga Formation consists of four members (University of Rochester, 1956; Lindemann and Feldman, 1981), two of which have been identified beneath the site. The lowermost is called the Edgecliff Member (Brett and Scatterday, 1994). It is a distinctive light gray, generally massive, very coarse crystalline limestone. The unit includes an abundant fauna of tabulate and rugose corals and crinoid columnals. This unit serves as a marker bed for bedrock identification as it is very distinctive when compared to the darker and finer-grained Bertie/Cobleskill Formation below and Nedrow Member of the Onondaga above. The Nedrow Member is a thin bedded, medium gray, very fine-grained argillaceous limestone (Brett and Scatterday, 1994). The occurrence of chert is very common (University of Rochester, 1956; Lindemann and Feldman, 1981). The core samples obtained from these units generally correlate with these descriptions.

The Cobleskill and Bertie Formations are combined because the contact is gradational at the site. The Cobleskill is a massive fine-grained brownish gray dolomite. The Bertie Formation consists primarily of units of massive dolostones, limestone beds, and shaley dolostone with wavey interbedded layers of argillaceous or siliceous material, rehealed fractures filled with gypsum, anhydride, and calcite or occasional pyrite crystals. Underlying the Bertie Cobleskill is the Camillus shale, which is described as a brownish gray dolostone. The Camillus shale was not encountered in the borings at the site.

Rock Quality Designation (RQD) recorded for some of the coring runs suggest that the rock quality, or frequency of fracturing does not appear to be a function of depth. The majority of the logged fractures are horizontal to sub horizontal (along bedding planes) with fractures, to some degree, in every boring. Fracture zones tend to be moderately weathered and are often associated with zones of lost core. Occasional higher angle fractures or breaks in the core were also observed.

Caliper logging, completed during the Supplemental RI of the Tantalo Site at four deep bedrock wells (T-23DB, T-26EB, T-30DB, and T-31DB) also indicates variability in the depth of fracturing. At T-23DB, the caliper log indicates a zone of relatively high fracturing at about 70 to 90 feet below ground surface (elevation 409-389), and again at about 115 and 120 feet (elevation 364-359). At T-26DB, the most evident fracturing was recorded at a depth of about 100 to 120 feet (elevation 397-377), with less fracturing above and below. The zone of highest fracturing in T-30DB was between 110 to 120 feet below ground surface corresponding to an elevation of 373-363, while at T-31DB an approximate 30-foot zone between approximately 75 to 105 feet below ground surface (elevation 413-383) was the most fractured. There was relatively minor fracturing above and below the zones in both boreholes. Finally, logging performed at the old SMI office well indicates that the highest fracturing was observed at approximately 50 to 58 feet below ground surface towards the bottom of the well.

A complete description of Site geology is presented in the Hydrogologic Investigation Report. The site geology as defined in detail therein, is conducive to landfill development. The low permeability glacial deposits provide a good level of natural containment to the existing and proposed facilities. As well, these materials are suitable for the construction of landfill liner systems, structural embankments and related components. With the exception of the bedrock knob, located within the southern limits of the SMI Valley Infill area, bedrock is sufficiently deep to provide for the required separation as defined by Part 360 Solid Waste Management Facility regulations, and helps provide a sound foundation for the proposed facilities. Within the southern end of the proposed landfill area, bedrock removal will be required to provide sufficient separation. Where bedrock is removed, it will be replaced by structural fill.

2.4.2 Site Hydrogeology

The regional groundwater framework for the site is characterized as predominantly bedrock controlled, superimposed on which are low permeability glacial till and glaciolacustrine soils, consisting predominantly of silt and clay. Given these low permeability soils and the relatively humid climate, these overburden soils are saturated close to the ground surface (i.e., high water table). The underlying bedrock is comprised of limestone and dolostone, which exhibit hydraulic conductivity values one or more orders of magnitude greater than those observed in the overburden. Given these conditions and the relative lack of topographic relief in the area, the bedrock serves as a groundwater sink or discharge point for overburden groundwater. The site-specific hydrogeologic characteristics, and the interaction between groundwater and surface water at the site, are discussed in the context of this regional framework below.

2.4.2.1 Surface Water

Surface water features at the site are generally man-made and range from the channel conveying Black Brook around the northern perimeter of the Site, to retention ponds associated with the active solid waste facility to the east south and west. Black Brook, the only flowing body of water, originates about 1 mile west of the site, and flows in an easterly direction until being diverted around the north side of the site before again flowing in an easterly direction. Approximately 1.5 miles east of the site, Black Brook turns northward and drains into the Montezuma Refuge located approximately 5 miles to the northeast.

Retention ponds associated with the existing operations are used to collect surface water runoff from the Facility. Retained water in the ponds is sampled to confirm that water quality meets discharge limits, and is then pumped into Black Brook.

The nature, elevation, and location of surface water features at the site are such that they do not serve as significant groundwater sinks. The hydraulic heads in the overburden and bedrock are generally lower than surface waters indicating that groundwater discharge to surface water is generally negligible. However, some groundwater discharge is expected from the UGL to the Retention Ponds when the water level in the ponds is pumped down.

2.4.2.2 Groundwater

Groundwater flow conditions have been studied and evaluated extensively over the years, primarily using head data obtained from monitoring wells and piezometers scattered throughout the Site. Groundwater occurs at the Site in the unconsolidated soils and bedrock, with the greatest increase in water levels during the spring months when greater recharge events (including precipitation and snowmelt) contribute water to the groundwater flow regime. Water elevation data obtained in the fall months reflect the seasonal low groundwater heads. Typically, the increases and decreases occur synchronously in the overburden and bedrock units, with some minor and short term exceptions. Groundwater flow is predominately vertically downward, from the overburden to the underlying bedrock.

Hydraulic Conductivity

The hydraulic properties of the overburden and bedrock geologic units underlying the site have been evaluated through slug testing at individual overburden and bedrock well locations, laboratory tests of Shelby tube samples collected from the overburden, packer testing and aquifer tests completed within the bedrock. Based on slug test results, the geometric mean hydraulic conductivity for the UGL, UGT, LGL and LGT were calculated at 1.8×10^{-5}, 2.7×10^{-6}, 2.4×10^{-5} and 1.4×10^{-6} cm/s respectively. Vertical hydraulic conductivity of the overburden soils has been estimated through laboratory testing of Shelby tubes. These data indicate a range of vertical hydraulic conductivity from 2×10^{-8} to 6×10^{-6} with the UGL deposits typically exhibiting a higher vertical permeability relative to the other overburden units.

Bedrock horizontal hydraulic conductivity has been estimated through performance of slug tests, packer testing, and aquifer testing and is generally in the 10^{-2} to 10^{-4} cm/sec range with a few values at 10^{-5} cms.sec. These data further indicate that while the horizontal permeability of the rock is generally consistent with depth, the vertical hydraulic conductivity of the bedrock above an elevation of approximately 420 to 430 feet msl exhibits a vertical anisotropy of approximately 1000, which equates to a vertical hydraulic conductivity of approximately 3×10^{-6} cm/sec. Conversely, the vertical anistropy of the bedrock below approximately 430 feet msl is negligible (i.e., approximately one, which equates to a vertical hydraulic conductivity of roughly 3×10^{-3} cm/sec). The lower vertical hydraulic conductivity of the bedrock above approximately 430 feet msl generally corresponds to the bedrock comprising the bedrock knob underlying the southern end of the Tantalo Site.

Collectively, the hydraulic conductivity data consistently demonstrates that the overburden soils are represented by low hydraulic conductivity values on the order of 10^{-5} to 10^{-6} cm/sec (on average) while the hydraulic conductivity of the bedrock is typically two to three orders of magnitude higher at approximately 10^{-3} cm/sec. As discussed in further detail below, this results in groundwater flow that is predominantly vertically downward through the overburden with significant horizontal flow restricted to the bedrock water-bearing zone.

Groundwater Flow

Overburden Groundwater

The uppermost water-bearing zone in the overburden is found within the UGL deposits or the UGT deposits where the UGL is absent. A review of the potentiometric surface for the UGL/UGT interval identified the presence of a depression in the water table surface beneath the Southeast Landfill (SELF) and Western Expansion Landfill (WEX) as a result of operating the porewater depression system. The areal extent to which these discharge points affects groundwater flow is limited as a result of the low hydraulic conductivity of the overburden deposits and the strong downward vertical gradients.

The strong vertical gradients within the overburden deposits, are a result of the low hydraulic conductivity of the overburden deposits, which overlie bedrock of higher hydraulic conductivity. As a result, groundwater flow within the overburden is predominantly vertical, with discharge to the underlying, higher permeability bedrock. Horizontal flow is primarily limited to those areas immediately adjacent to the localized discharge point represented by the porewater suppression system.

A localized horizontal flow component may also be present within the LGL deposits near the southeast corner of the proposed landfill area, based on the presence of slightly thicker LGL deposits and the highest reported hydraulic conductivity occurring in this area. Although the evidence suggests that these changes are localized, there is the possibility that some lateral flow may occur within this localized area.

Bedrock Groundwater

Within bedrock groundwater, a potentiometric high was observed within the bedrock knob area, with a relatively flat potentiometric surface beyond the limits of the knob. The groundwater high within the knob area is present as a result of increased recharge due to the thinner soil cover in this area and the lower vertical hydraulic conductivity, as discussed previously.

As a consequence of this increased recharge and lower vertical permeability, the vertical hydraulic gradients within the bedrock knob area (i.e., above an elevation of approximately 430 feet msl) are strongly downward and groundwater in this area is unconfined. Groundwater flow within the bedrock knob area is thus radial, away from the potentiometric high and vertically downward into the underlying lower bedrock water-bearing zone. Once in the lower bedrock, horizontal flow paths dominate and the groundwater is under confined conditions with flow consistent with the regional gradient to the south, with localized variation as a result of anisotropy.

Groundwater flow in the lower bedrock water-bearing zone, which essentially encompasses the area beyond the limits of the bedrock knob and below an elevation of approximately 430 feet msl, is characterized by low vertical and horizontal gradients. These conditions are indicative of the permeability and negligible vertical anisotropy observed during the PW-100 aquifer test, and result in predominantly horizontal flow paths to the south-southwest, consistent with the regional groundwater flow regime.

Collectively, the potentiometric surface maps and hydrogeologic cross sections illustrate a groundwater flow system dominated by downward vertical flow paths through the overburden, which behaves as an aquitard, with discharge to the underlying bedrock water-bearing zone, which behaves as a confined aquifer. Vertical flow paths are also present within the bedrock knob area to an elevation of approximately 430 feet msl, at which point the bedrock exhibits negligible anisotropy and the flow paths become predominantly horizontal with flow to the south, consistent with the regional groundwater flow regime. Localized variations in the groundwater flow direction are also present as a result of anisotropy within the bedrock water-bearing zone.

3.0 LANDFILL DESIGN

3.1 PROPOSED LANDFILL DEVELOPMENT

The proposed landfill is designed as both vertical and horizontal landfill development. The vertical development will increase the overall elevation by approximately 70 ft from the permitted maximum landfill elevation of 774 ft to an approximate elevation of 843.5 ft. The proposed lateral development includes approximately 47 +/- acres of solid waste facility property adjacent to the WEX and SELF footprint.

Most of the 47 +/- acre proposed landfill is presently occupied by the Tantalo Site and will be included in the baseliner/overliner design of the subgrade for the landfill. Additional infrastructure situated in the area includes a Citizens Drop-off Area, wheel wash areas, fueling islands, and multiple maintenance shops. These will be relocated within the facility boundary as necessary during site development.

The following sections address each element of the design for the proposed landfill.

3.2 LANDFILL CAPACITY

The proposed landfill provides an air space volume gain of approximately 46.6 million cubic yards for waste placement. By applying historical site density values of 0.85 Tons/cy (1,700 lbs/cy) to the anticipated volume gain, it is projected that the landfill will be capable of accepting up to an additional 40 million tons of waste.

3.2.1 Waste Characterization and Quantities

The Seneca Meadows Landfill, under its current operating permit, is permitted to accept an annualized average of 6,000 tons of solid waste per operating day. A review of 2018 waste stream data indicates the following general breakdown expressed in percentages of waste received at SMI:

- 86% Municipal Solid Waste
- 7% Contaminated Soils
- 3% Construction and Demolition Debris
- 2% Sludge, Grits and Screenings
- 1% Industrial Waste
- 1% Friable Asbestos
- < 1% Ash
- 0% Medical Wastes

Waste composition at the Facility is expected to remain generally consistent with the 2018 data. Seneca Meadows also uses materials that have received a BUD under 6 NYCRR 360.12. SMI's current permit and the Part 360 regulations define BUDs as those materials that should be considered solid waste if not beneficially reused. These materials are excluded from the current 6,000 tons per day waste acceptance limits. Specific materials such as compost and contaminated soils are considered BUDs provided they meet specified requirements. In addition, case-specific beneficial use determinations can be made by petitioning the NYSDEC in writing.

The proposed landfill facilities are proposed to operate at an approved design capacity of 6,000 tons per day, excluding BUD material and Alternative Operating Cover (AOC), and it is this rate that will control the rate of waste disposal at the facility. The approved design capacity for the Facility has been established as 6,000 tons per day, calculated on an annual basis. The volume during any calendar quarter is limited to the average daily tonnage of waste received not exceeding 9,000 tons per day. No increase in this acceptance rate is being proposed.

SMI receives several waste streams that require special handling. Procedures for handling these waste streams are provided in the Facility Manual. These waste streams include:

- Asbestos and Fine Materials;
- Treated Regulated Medical Waste; and
- Sludge.

3.3 ADDITIONAL LIFE EXPECTANCY

The SMI Valley Infill's landfilling life expectancy was evaluated using an annual rate of 3.04 million cubic yards for combined airspace consumption of both incoming waste and Alternative Operating Cover (AOC) materials (see Section 3.8). This rate was used based on the assumption that the maximum amount of waste (6,000 tpd) and AOC materials (up to 20% of waste acceptance tonnage) is taken in by the landfill each year, providing a conservative life expectancy estimate. The approximate 47 million cubic yards provided by the landfill will allow the landfill to continue to operate to the year 2040 or for 15 +/- years beyond the estimated final closure year of 2025 for the currently permitted landfill. The individual life expectancy for each phase of landfill development is outlined below:

Development Phase	Phase Volume (Cubic Yards)	Phase Life Expectancy (Years)
Phase 1	3,635,000	1.2
Phase 2	5,835,000	1.9
Phase 3	7,047,000	2.3
Phase 4	7,105,000	2.3
Phase 5	11,685,000	3.8
Phase 6	11,342,000	3.7
Total	46,650,000	15.3

3.4 BASELINER SYSTEM DESIGN

In accordance with 6NYCRR Part 363 requirements, the liner system design will provide for both primary and secondary collection and removal of leachate. The proposed liner system will meet, at a minimum, the requirements of 6NYCRR Part 363-6.6.

3.4.1 General

The dual composite liner systems for the proposed landfill area, including the overfill liner areas, are designed and will be constructed in accordance with the requirements of the 6NYCRR Part 360 regulations. The components of the liner systems on slopes of less than 10 percent, in ascending order, are as follows:

- Prepared subgrade;
- Porewater collection system, where subgrade is below the seasonal high groundwater table, consisting of the following:
  - Geotextile overlain by 24" stone mat with embedded pipe network in the development area for porewater drainage and construction stability (in areas where fine silty sand is encountered on sideslopes);
  - Geocomposite drain (GCD) with embedded pipe network in other areas;

- Two-foot thick secondary soil liner;
- 60 mil textured secondary HDPE geomembrane;
- Secondary GCD collection layer with embedded pipe network;
- One foot thick secondary leachate collection soil layer (only placed on slopes less than 10%);
- Geosynthetic clay liner (GCL);
- 60 mil textured primary HDPE geomembrane;
- Geotextile cushion;
- 12-inch thick natural aggregate (No.1 stone, minimum) leachate collection layer with embedded pipe network; and,
- 18-inch thick tire chip layer.

In accordance with 6NYCRR Part 363-6.6(a)(2), the primary liner system on slopes greater than 10 percent may be a single geomembrane overlying the secondary collection system for areas five vertical feet up the sideslope. Therefore, on slopes greater than 10 percent, the GCL will be installed only on the first five vertical feet. Consistent with current proposed regulations, SMI is proposing the use of a combined geosynthetic/sand layer for the secondary leachate collection system. Also, the one-foot secondary collection soil layer is not included in the design on the slopes greater than 10 percent. The components of the liner systems on slopes greater than 10 percent, in ascending order, are as follows:

- Prepared subgrade;
- Two foot thick secondary soil liner;
- 60 mil textured secondary HDPE geomembrane;
- Secondary GCD collection layer;
- Geosynthetic clay liner (GCL) (installed only on the first five vertical feet);
- 60 mil textured primary HDPE geomembrane;
- Geotextile cushion;
- 12-inch thick natural aggregate (No. 1 stone, minimum) leachate collection layer; and,
- 18-inch thick tire chip layer.

Typical liner configurations are shown on the Engineering Drawings, Sheets 20 & 22.

3.4.2 Subgrade

The base of the proposed landfill area will be founded in the unconsolidated deposits occurring naturally at the Site. The need for undercutting and removal of native subgrade soils is based on the occurrence of a bedrock knob at the southern portion of the landfill that will require removal and replacement with structural fill to meet bedrock separation requirements.

In addition, some unconsolidated "soft" soils in the north need to be improved to increase soil strengths. A soil improvement program has been performed during the construction of previous landfill cells using soil columns to increase the overall strength of subgrade soils. A phased filling approach that allows the soils to consolidate may also be implemented. Details are discussed in the slope stability section and mirror previous programs. A detailed improvement program will be explored and finalized for each phase of construction.

The engineering properties of site subgrade soils have been characterized through the geotechnical sampling and testing program performed as part of the geotechnical investigation. The details of the field and laboratory geotechnical testing program are contained in the Hydrogelogic Investigation Report.

The engineering properties of the site subgrade were used to establish the bearing capacity of the foundation soils, conduct stability analyses, and to estimate settlement. The estimated settlement was then used to define pre-settlement liner and collection pipe slopes that will result in minimum post-settlement slopes of two percent for liners and one percent for collection piping. The details of the bearing capacity, slope stability analyses and settlement calculations are contained in Appendix H of this report.

The subgrade will be proof rolled to assess whether pockets of unsuitable subgrade soils are present (e.g., soft materials that will not meet the foundation specifications). Unsuitable materials will be removed and replaced with compacted, structural fill or improved with rock columns as has been done in previous cell construction. Before placing porewater relief or liner materials over the subgrade, the exposed surface will be observed and tested in accordance with the requirements of the CQA/CQC Plan to confirm the suitability of the subgrade soils.

Prior to placement of geosynthetic materials above the subgrade, the surface will be prepared with a smooth drum compactor. The resultant surface will then be inspected for sharp objects that may damage the geosynthetic materials, and if objects are found they will be removed. The subgrade will also be checked for a uniform surface suitable for geosynthetics (e.g., no pockets that may cause bridging of geosynthetics) and will be reworked as needed to meet subgrade acceptance requirements.

3.4.2.1 Overfill Liner Subgrade

Subgrade preparation for the overfill liner will be similar to that for areas above native soils. The area of the Facility that will receive overfill liner has previously received final cover. As such, the waste subgrade would already have been made suitable for placement of a compacted low permeability soil cover (e.g., proof roll, address soft spots, etc.). Then for construction of the overfill liner, the subgrade preparation process will proceed as follows:

- Overlying layers of topsoil will be stripped and stockpiled for future use in the landfill operation.
- The existing final cover geomembrane will be inspected.
- The subgrade will be installed and proof rolled to prepare for placement of liner system.
- Overliner materials will be placed on the prepared subgrade.

The stability and settlement analyses for the overfill liner are also presented in the Appendix H of this report.

3.4.2.2 Bedrock Removal

As mentioned above, a bedrock knob is present along the southernmost phase of the proposed landfill area. Bedrock associated with this knob will require removal as part of the overall work to achieve practicable subgrade conditions for the liner installation.

Removal of the bedrock will entail the use of in-situ blasting followed by conventional excavation, and then followed up by further processing of the bedrock to develop a usable product. The limits of bedrock removal are depicted on Sheet 7 & 12 of the Engineering Drawings. Following removal of the bedrock, structural fill will be placed in the excavation to achieve the required 10’ of separation between the bottom of the liner system and top of bedrock.

3.4.3 Porewater Drainage System

To maintain heads below the double composite liner system, a porewater drainage system will be installed on the prepared subgrade of the areas where the seasonal high groundwater table elevations are above the bottom of the liner system elevations. The extent of the porewater drainage system is shown on the Engineering Drawings, Sheet 7. The porewater drainage system will be dewatered until such time as the head above the liner system is equalized by the liner system and overlying waste, and will help provide a firm, stable foundation upon which the liner system will be constructed.

The proposed porewater drainage system for the Landfill are consistent with the existing and permitted infrastructure. The proposed and existing porewater drainage system consists of the following components:

- A geocomposite drain (GCD) or a stone mat as a blanket drain over the area in which porewater collection is necessary.
- Piping along the low points of the porewater drainage system to collect the groundwater and convey it to a sump.
- A sump and submersible pump from which collected porewater will be pumped to the site-wide stormwater system.

Each of these components of the porewater drainage system is discussed in more detail below.

3.4.3.1 Drainage Layer

The drainage layer or blanket drain portion of the porewater drainage system will be comprised of either a GCD or a stone mat. The GCD will be placed to lines and grades illustrated on the subgrade plan, Sheet 7 of the Engineering Drawings. Installation of the GCD will proceed as described above for subgrade preparation for receiving a geosynthetic material.

The geotextile portion of the GCD is designed to control the movement of soil particles that could potentially obstruct the geonet, and it must be sufficiently permeable to allow groundwater to enter the drainage net.

The 6NYCRR Part 363 regulations establish the following criteria for design of a geotextile filter:

$K_f > 10K_s$ (permeability criterion) where:

K_f is the geotextile permeability, and

K_s is the overlying soil permeability

Geotextile AOS (apparent opening size) < (3)d_{85} (retention criteria)

Where d_{85} is the 85% finer soil particle size

The GCD will include an HDPE drainage net with heat bonded 6-8 ounce/square yard non-woven needle-punched geotextile above and below the net. The porewater drain will be underlain by the prepared subgrade and will underlie the secondary soil liner. However, where limited undercuts (e.g., <2 feet) are required to remove soft or sandy soil, the GCD may be placed on the undercut prior to placement of the subgrade fill, provided adequate drainage to the low points of the system are maintained. Calculations for the proposed drainage layer are addressed in the table below.

Geotextile Permeability; K_f (cm/sec)	Soil Permeability; K_s (cm/sec)	Geotextile AOS #70 (mm)	d_{85} (mm)
0.3	1.7×10^{-3}	0.15	0.085

A geotextile with #70 AOS meets the requirements for both permeability and retention.

3.4.3.2 Porewater Drain System Piping

Once the blanket drain has conveyed groundwater to the pipe drains placed in the low points of the baseliner configuration the pipe maintains significant capacity to convey flow to the sumps. The lateral and header pipe for the porewater drain system will consist of perforated PVC pipe. The required flow capacity of the porewater collection piping was determined based on the largest calculated seepage flow rate at a slope of one percent (minimum post settlement slope along pipe alignments).

The flexible pipe/soil system proposed for use has been designed to withstand the physical stresses imposed during construction, normal waste placement operations and over the post closure period. The PVC pipes for the porewater drain system were designed in accordance with Section 3.7.3.1. Pipe loading calculations for worst case loading conditions are presented in Appendix H of this report.

To prevent clogging of the drain pipe by the surrounding bedding material, typical pipe perforation size has also been calculated and is presented in Appendix H of this report. Based on typical No. 2 stone bedding material, the typical perforation size is 10-11 mm, or approximately 3/8". A typical perforation arrangement is illustrated on the Engineering Drawings, Sheet 27.

Use of both PVC pipe and HDPE pipe is proposed for the new landfill development. PVC piping will be utilized in areas underneath wastes and soils which need to be able to bear significant loads. In addition to the long-term loading considerations associated with waste filling, PVC piping is designed to resist stresses during baseliner construction. Equipment traffic above the pipe will however be limited to small and/or low ground pressure equipment as approved by the Engineer. HDPE piping will be utilized on the slopes of the landfill coming out of sumps and cleanouts, where loading will be much less significant and the use of HDPE will make for easier piping transitions and design at sump areas. Engineering plans will clearly indicate which type of piping will be used in each part of the design.

3.4.4 Soil Liner

The secondary soil liner has been designed and will be placed in accordance with the requirements of 6NYCRR Part 363-6.6(a)(1)(ii). Soil liner will be placed directly above the prepared subgrade and beneath the secondary HDPE geomembrane. Soil liner will exhibit a maximum hydraulic conductivity of 1×10^{-7} cm/sec. The compacted thickness of the secondary soil liner will be a minimum of 24 inches.

3.4.5 Secondary Geomembrane

The secondary geomembrane will be 60 mil textured high density polyethylene (HDPE) placed in accordance with the requirements of 6NYCRR Part 363-6.6(a)(1)(ii). The proposed material will be textured on both sides and will provide adequate interface shear strength to support landfill stability.

3.4.6 Secondary Collection System

A secondary leachate collection and removal system will be placed between the primary and secondary liners and will be designed and installed in accordance with the requirements of 6NYCRR Part 363-6.6(a)(4).

The secondary leachate collection system will consist of a geosynthetic drainage composite (GDC), 12 inches of overlying gravel, and a perforated pipe system designed to convey liquid from above the secondary composite liner system to the secondary sumps for removal. The secondary leachate collection and removal system must contain a minimum of 1 foot of drainage media (gravel) with a hydraulic conductivity of 0.1 cm/s or greater in areas with slopes of less than 10%. On slopes greater than 10%, the secondary leachate collection system may be constructed of a geosynthetic drainage layer system that can maintain a leachate head less than the thickness of the confined drainage layer itself. In accordance with 6NYCRR Part 363-6.10(a), the drainage layer must have less than 15 percent calcium carbonate equivalent and less than 5 percent material by weight that can pass the No. 200 seive after placement.

Flow from the secondary system will be measured and averaged over 30-day periods to determine the leakage rate for the primary liner system. The secondary leachate collection system will be discontinuous between operational cells to allow separate and distinct monitoring of the primary liner system between subcells.

The GDC will consist of a geonet core with a non-woven needle-punched geotextile heat bonded to both sides consistent with the specifications. The geocomposite will be placed on top of the textured secondary membrane liner, and below a 12-inch drainage layer. See Appendix H of this report for GDC calculations.

3.4.7 Geosynthetic Clay Liner

The primary geosynthetic clay liner (GCL) will be a manufactured liner system consisting of a layer of sodium montmorillonite (bentonite) sandwiched between two layers of geotextile. The layers of geotextile will be bound together by needle punching the fibers from one layer through to the other layer. The GCL will be placed in accordance with the requirements of 6NYCRR Part 363-6.6(a)(i). The GCL will have a permeability of approximately $5x10^{-9}$ cm/sec and with the other layers of the landfill baseliner system will provide adequate shear resistance to support the stability of the landfill.

3.4.8 Primary Geomembrane

The primary geomembrane will be 60 mil textured high density polyethylene (HDPE) placed directly above the Primary GCL in accordance with the requirements of 6NYCRR Part 363-6.6(a)(1)(i). The proposed material will be textured on both sides and will provide adequate interface shear strength to support landfill stability.

3.4.9 Primary Leachate Collection System

A primary leachate collection and removal system will be placed above the primary geomembrane and will be designed and installed in accordance with the requirements of 6NYCRR Part 363-6.6(a)(3).

The primary drainage layer consists of drainage aggregate and geotextile. The lower portion of the primary leachate collection layer will consist of 12 inches of No. 1 rounded stone. The upper portion of the primary leachate collection layer will consist of 18 inches of shredded tires, or tire “chips”. Depending on the slope and intended loading of the design area, one or two layers of 16 oz/sy geotextile will be used beneath the drainage aggregate as a cushion between the stone and the geomembrane. Calculations for the geotextile cushioning are provided in Appendix H of this report.

The primary leachate collection system must have a hydraulic conductivity of 1 cm/s or greater on slopes less than or equal to 10%. Alternately, the upper 18” of the primary system may have a permeability of 0.1 cm/s or greater if the lower 12” (or more) has a hydraulic conductivity of 1 cm/s or greater. On slopes greater than 10%, hydraulic conductivity must be 0.1 cm/s or greater. Materials used in the drainage layer must meet the carbonate and maximum fines requirements from Section 3.4.6.

3.4.10 Tire Chips/Shreds

6NYCRR Part 363-6.21 allows for the landfill design to include alternative materials to those specified in the regulations, pending a demonstration of the alternative material's ability to perform in the same manner. SMI's landfill design proposes the continued use of shredded tires in the upper portion of the primary leachate collection system, in lieu of granular soil material identified by 6NYCRR Part 360-6.21(b)(1).

SMI has successfully employed shredded tires in its operation since 1999. Prior to the use of drainage material in the construction of the facility, the quality control testing as required by the CQA/CQC Plan will be performed to demonstrate the suitability of the materials. For SMI's application, three inch nominal tire chips are proposed for continued use, and the use of tires in this manner will be based on availability, and is consistent with the Beneficial Use provisions of 6NYCRR Part 360.

Three inch nominal tire chips are durable and lightweight, typically exhibiting an uncompressed density of approximately 28 pounds per cubic foot (pcf). The tire chips proposed for use in the baseliner system will consist of shreds of whole tires, whose gradation is equivalent to that of uniformly graded coarse gravel. Tire chips have been demonstrated to exhibit thermal conductivities up to eight times lower than common soils, which makes them a good insulator. In cold climates they have been used to limit frost penetration and reduce the effects of frost heave. The material to be used will be free of organic matter and have less than five percent passing the No. 200 sieve after placement. Steel belts in whole tires result in short wire protrusions from the chips, and to protect the primary membrane liner from possible damage, tire chips are specified only within the upper portion of the leachate collection system.

Prior to including the shredded tires in the leachate collection system, the material will be subject to the testing program prescribed for the coarse aggregate used in the lower 12 inch thickness. This testing includes gradation and permeability testing as specified in the CQA/CQC Plan, and will be certified by the project engineer.

3.5 SETTLEMENT AND STABILITY ANALYSIS

A geotechnical analysis of the native soils, design subgrade, and design final grades of the proposed landfill has been completed to address the suitability of the design to maintain the integrity of the solid waste containment systems. The geotechnical analysis includes slope stability, subgrade settlement and bearing capacity. The stability analyses encompass evaluations of landfill cell grading, berm fill, soil subgrade under both waste and soil fill, operational conditions, baseliner (veneer), and final cover (veneer). Settlement calculations involve assessment of anticipated subgrade settlement, post-settlement baseliner slope, post-settlement leachate collection system slope, and baseliner strain.

3.5.1 Subsurface Information

3.5.1.1 Geologic Units and Engineering Properties

The geotechnical information regarding the landfill site was obtained from geotechnical and hydrogeologic data obtained and reduced specifically for this project, subsurface geotechnical explorations performed to specifically address the geotechnical properties of soil units, and from previous hydrogeologic and geotechnical studies performed at the facility. A review of site hydrogeological and geotechnical data showed that the proposed landfill development areas are generally underlain by five soil layers. The stratigraphy at the site consists of the following, (see Section 2.5.1 for further details):

- Glaciolacustrine Silt and Clay
- Glacial Till
- Bedrock
- Recent Fill Materials
- Municipal Solid Waste

Further details of the soil index properties and modeled parameters can be found in Section 2.4.

3.5.1.2 Field Investigations

The landfill has been extensively studied through each phase of prior development. The current proposed landfill development fills over top of existing landfill areas and as such, the existing investigations provide adequate data and documentation for this design. No new investigation was undertaken to demonstrate the performance of the design.

Subsurface field investigations involving soil borings and groundwater monitoring wells have been conducted to characterize the site of both existing and proposed landfill facilities on the site. See Appendix D of this report for the locations of applicable geotechnical investigation locations.

3.5.1.3 Geotechnical Parameters of Fills

Two instances of fills are anticipated on site: municipal solid waste (MSW) and compacted structural fill.

Municipal Solid Waste

Engineering properties of waste can vary with waste composition. Waste is generally comprised of household debris, mechanically compacted, and mixed with a thin lift of daily cover. The total in-place unit weight of waste, which includes imported waste, Beneficial Use Determination (BUD) material, and cover sand, has historically been about 70 pcf at the SMI Landfill. Shear strength of municipal solid waste has been modeled using a shear strength-normal strength function, as described in Appendix H of this report. Though mechanical compaction leads to some degree of overconsolidation in the waste, a conservative assumption of normal consolidation of waste is made in the calculations.

Structural Fill

Strutural fill, or common fill, is primarily comprised of soil obtained from on-site or off-site sources that has historically been placed for general construction and for the building of access roads. The unit weight of existing common fill at the site is estimated to be 120 pcf based on representative blow counts during historical subsurface exploration programs. Due to low observed moisture content and high gravel/sand content in the common fill material, the common fill appears to demonstrate a drained behavior. The appropriately conservatively low effective stress parameters for the unclassified common/structural fill are a friction angle of 28 degrees and no cohesion. Higher friction angles are possible and are dependent on the fill source.

3.5.2 Stability Analysis

3.5.2.1 Approach

A stability analysis has been performed in accordance with 6NYCRR Part 363-4.3(c) to demonstrate the structural integrity and overall stability of the landfill site, the subgrade, each component of the liner, leachate collection and removal system, and final cover system. The failure modes that have been evaluated include:

- Global Stability – Static
- Global Stability – Seismic
- Baseliner Stability – Short Term
- Veneer Cover Stability

The procedure for evaluating landfill stability, along with acceptable factors of safety, are described herein.

3.5.2.2 Acceptable Factors of Safety for Slope Stability

The following factors of safety were considered acceptable while evaluating the results of the slope stability analyses:

Global Stability

For deeper failure surfaces that pass through the subgrade, a factor of safety 1.50 was required for all configurations where the stability of waste was potentially impacted. This requirement is consistent with 6 NYCRR 363-4.3(c)(1)(i). Configurations without potential for impacting waste are considered under construction stability. A factor of safety of 1.25 was considered acceptable for veneer residual strength baseliner stability.

Seismic Stability

The site is not within a Seismic Impact Zone as defined in 6NYCRR Part 363-4.3(d) and the USGS Unified Hazard Tool for Peak Ground Acceleration. Therefore seismic analysis is not necessary and has not been performed.

Baseliner Stability

A factor of safety of 1.50 was considered acceptable for peak strength baseliner stability, in accordance with 6NYCRR Part 363-4.3(c)(1)(ii). A factor of safety of 1.25 was considered acceptable for residual strength baseliner stability. This stability is the stability of the liner system on slopes that are 3H:1V.

Veneer Cover Stability

A factor of safety of 1.50 was considered acceptable for veneer cover stability, in accordance with 6NYCRR Part 363-4.3(c)(1)(iv). A factor of safety of 1.25 was considered acceptable for veneer residual strength cover stability. The veneer cover stability is for the final cover system and considers the impact of water above the liner and landfill gas below the liner.

3.5.2.3 Landfill Stability Analysis Procedure

Global, liner, and cover stability analyses were performed using the computer program SLOPE/W. The program uses the Morgenstern-Price Method, an iterative approach that tests for moment and force equilibrium, for computing factors of safety. The program includes options for circular, or block specified failure surfaces to allow users to specify and optimize critical failure surfaces. Stability analyses were performed for static and appropriate dynamic conditions.

3.5.2.4 Short-term Analysis Versus Long-term Analysis

Short-term analyses are performed using the total stress (undrained) strength parameters. Long-term analyses are performed using the effective stress (drained) strength parameters. The type of analysis that is critical for design depends on the subsurface conditions and the nature of loading. Since the only appreciable difference between short-term conditions and long-term conditions is a likely strength gain in cohesive soils, long-term conditions have been used for the final configuration. Short-term undrained conditions are considered for post-construction and operational slopes.

For soils where there is limited possibility for pore pressure development during loading or unloading (i.e., granular soils or soils with a sizable proportion of coarse-grained particles and low moisture content) drained soil conditions are applicable. This is represented by the use of a Mohr-Coulomb failure criterion, modeled with a friction angle and cohesion. Conversely, filling at sites that are underlain by saturated cohesive soils can result in development of excess pore pressure; therefore, undrained conditions are assumed to exist under the stress envelope of proposed waste and fill loading. Undrained conditions presume that the soils do not exhibit frictional behavior ($\phi = 0$), and therefore have a shear strength independent of applied load. After the completion of final cover, excess pore pressures will likely dissipate and the undrained shear strength of the soil under consideration increases.

In areas of undrained soils under existing waste loads, the soil strength has been modeled using the Stress History And Normalized Soil Engineering Properties (SHANSEP) approach proposed by Ladd and Foott (1974). This strength gain is dependent and governed by a tau/sigma ratio of the undrained soil in its undisturbed state. To further support the application of the SHANSEP approach on previously loaded undrained soil deposits, the amount of time required to realize 70% consolidation of the in-situ undrained soil below existing waste loads has been considered (see Appendix H of this report). A ratio of 0.30 was considerd for normally consolidated soil based on previous reports. This computation found that approximately one year is required to realize 70% consolidation. Existing waste loads on top of undrained material has been applying loads for 10 to 30 years.

3.5.2.5 Selection of Strength Parameters

Strength parameters were assigned to each of the layers and components of the proposed landfill, the existing landfill, the landfill lining systems, and landfill subgrade. The value, and basis for selection, are presented below.

Soil Strength Parameter Selection

The in-situ clay units were assigned undrained shear strength parameters. Placed fills, in-situ tills, and wastes were selected to exibit drained behaviors. The selected strength parameters are supported by laboratory test data and are presented as follows:

Layer Name	Unit Weight (pcf)	Cohesion	Phi
Compacted Fill	130	0	30
LGL	130	0	34
LGT	135	0	36
Geomembrane	110	0	15
Solid Waste	70	0	0
UGL	120	0	30
Undrained UGL/UGT	120	600 psf min undrained*	NA
UGT	130	0	34

The short-term shear strength of soft zone of UGL and UGT are discussed in the next section.

The short-term strength for soft UGL and UGT were based on consolidated undrained strengths are were determined based on the SHANSEP method discussed previously. A soil strength calculator, included in Appendix H of this report, was developed to calculate the strength based on load and percent consolidation. Based on testing values to determine the coefficient of consolidation, the soft soil can consolidate to 70 percent of full consolidated strength within one year. Within that one year the shear strength is approximately 0.35 times the effective overburden load for the typically slightly overconsolidated soft soils. Soft soils were modeled with the following general changes of undrained shear strength.

Overburden	Shear Strength
Current overburden from existing small berm or soil cover	600 psf
Tantalo waste overburden	800 psf
Generally 15 feet of soil or 25 feet of additional waste	1200 psf
Generally 25 feet of additional soil or 40 feet of addtitional waste	1600 psf
Generally over 30 feet of additional soil or over 50 feet of waste	2000 psf
Maximum value used for at least 75 feet of waste or soil over soft soils	2400 psf

Actual values computed are included in Appendix H of this report.

Geosynthetic Parameter Selection

Sections 3.4 and 3.8 involve the specification of required interface strengths such that the proposed geosynthetic systems achieve a satisfactory factor of safety. These requirements apply to all geosynthetic-geosynthetic and soil-geosynthetic interfaces in the proposed baseliner and final cover systems.

Once appropriate materials have been selected, laboratory testing shall be performed to confirm that the interfaces described in Sections 3.4 and 3.8 will meet or exceed these requirements. The geosynthetic interface data included in the calculations (Appendix H of this report) are generally attainable with materials that are available for these particular applications. It should be noted that residual conditions only apply to the interface with the lowest peak strength, and therefore, not all interfaces will be required to have the residual strengths reported herein. Likewise, the specified strengths assume no cohesion. Once testing has been completed, further analysis can show that interfaces that would otherwise not meet the specifications can, when consideration for cohesion is determined to be appropriate, be acceptable under specific loading conditions. Based on experience, suitable materials can be obtained which possess the required material properties.

3.5.2.6 Stability Analyses for the Proposed Landfill

The locations for analysis were chosen with consideration for either the existing soil conditions, as documented by field and laboratory tests, or the proposed landfill geometry. Special attention was given to areas where soils demonstrating lower shear strengths had been identified, and where maximum loading is proposed. Slope stability section locations are shown in Appendix H of this report and the applicability of the sections are summarized below:

Section A – Final waste sloper configuration and stability of berm

Section B – Construction fill slope on north end; short term operation.

Section C – Stability of existing waste slope over cell excavation

Section D – Interim stability of Phase 4

Section E – Interim stability of Phase 1

The results of the stability analysis are summarized in the table below.

Section	Failure Mode	Factor of Safety
A	Block at Peak	1.61
A	Block with Residual	1.25
A	Circular	1.99
B	Circular	1.46*
C	Circular	2.09
D	Block at Peak	1.77
D	Block with Residual	1.44
E	Block at Peak	1.59
E	Block with Residual	1.35
E	Circular	1.71

**Section B includes improving section of the soil to an undrained shear strength of 2000 psf*

For Section B the soft soils are the toe of slope would have to be strengthened in an approach to soil improvements used previously at the site. The specific extent of the improvements would be refined before construction. Another option would be to develop the fill in stages and to allow for the soft soils to consolidate under each stage. Each stage would take approximately one year. The staging is summarized below and included in Appendix H of this report.

Stage	Consolidated Undrained Shear Strengths	Factor of Safety
Fill to Elev 500	600-800 psf based on existing load	1.54
Fill to Elev 515	800-900 psf based on fill to Elev. 500	1.51
Fill to Elev 530	1200 psf based on fill to Elev. 515	1.54
Fill to completion (about Elev 543)	1200-1600 psf based on fill to Elev. 530	1.55

Section D is an interim slope stability section through Phase 4. From the previous table, there are no issues with slope stability for failures through the liner system. However, the soft soils that may underlay the Tantalo Waste in this area required more extensive analysis. These soft soils will gain strength as the soil fill, new liner system, and subsequently new waste is placed. Using the current undrained shear strength for evaluating the full interim waste grade does not achieve adequate factors of safety but also disregards the increased in shear strength over time. To more completely evaluate the stability, the Phase development was considered and the slope stability was evaluated at the various stages of development as summarized below. The interim waste slopes in this Phase would have to be no steeper than 4:1 to achieve the minimum factor of safety of 1.5.

Stage	Consolidated Undrained Shear Strengths	Factor of Safety
Waste fill to Elev. 630	800 psf based existing strength under Tantalo waste load	1.51
Waste fill to Elev. 780	1200, 1600, 2000, and 2400 psf depending on location and overburden load; strength developed from compacted fill load which will have been in place for more than one year	1.51
Waste to Full height for Phase 4	1200, 1600, 2400 and 2400 psf; allows for small amount of consolidation under first two lifts of waste which will have been in place for over one year	1.51

3.5.2.7 Global Stability

Global stability analyses assess the stability of the final landfill configuration. While this represents the final developed configuration, undrained, short-term conditions have been applied with strength gain only where existing waste has been in place for 10 to 30 years. This is conservatively reasonable based on an estimate of one year for cohesive soils to achieve 70% consolidation under an increased loading condition. Where the existing waste is shown over undrained soil materials, a tau/sigma strength function was modeled for undrained material.

Undrained conditions result in a factor of safety greater than 1.5, as shown in the analyses provided in Appendix H of this report.

3.5.2.8 Baseliner Stability

Stability calculations have been performed for the proposed geosynthetic baseliner to confirm that the baseliner will be both constructable and stable on a short-term basis before waste placement is performed. The construction of the baseliner system will result in various geosynthetic and soil interfaces. Minimum strength requirements for these interfaces have been developed that will provide an acceptable factor of safety and are provided herein. It should be noted that all interfaces do not necessarily exist at all locations, the interface with the lowest peak shear strength (i.e., the one that would be the first to shear and be governed by residual shear strengths) is taken as the critical interface for residual shear strength factor of safety consideration. Baseliner slope stability calculations are presented in Appendix H of this report.

Design Requirements

As required in 6NYCRR Part 363-4.3(c) the factor of safety against sliding for the baseliner system must be at least 1.5. Therefore, the required factor of safety for interface stability is 1.5. The baseliner system will be installed on slopes with maximum approximate final design grade of 18.4°, which is equivalent to a 3 horizontal to 1 vertical (3H:1V, 33%) slope. In areas where the slope is 10% or less, 6NYCRR Part 363-6.6(a)(1) specifies the use of a double composite liner system; this results in additional interfaces for analysis in areas exhibiting these shallower slopes. However the required factor of safety of 1.5 or greater remains the same for these reduced slopes.

Although not addressed in the regulations, the variations between peak and residual shear strengths of the interfaces have been considered. Although there is not an industry ‘standard’, it is accepted practice to design interfaces with a minimum factor of safety of 1.5 using peak shear strengths and a minimum factor of safety of 1.25 using residual shear strengths in areas where significant deformation could occur. Due to construction of liner on existing waste, it is possible for interfaces to be strained enough such that the residual shear strength governs. As this situation is usually localized and can be minimized by good construction practice, a safety factor lower than 1.5 using residual shear strengths is considered appropriate. Calculations for both the peak and residual conditions are presented in Appendix H of this report.

It is assumed that the Geosynthetic Composite Drainage (GCD) layer will have a similar geotextile cover on both the top and the bottom, and will be utilized within the Secondary Leachate Collection and Recovery System (SLCRS) as well as the Porewater Conveyance GCD. It is recommended that interface testing be performed with the representative soils and geosynthetics (coarse sand, secondary soil liner, and natural subgrade) to determine compliance to the interface requirements of the CQA/CQC Plan and Technical Specificaions provided in Appendix F of this report.

The porewater GCD must also be capable of transmitting water that enters the porewater collection system, as the porewater management system must provide for the effective removal of porewater from below the liner system and eliminates the buildup of excess pore pressures along the GCD-soil interface. Anticipated groundwater flux is discussed in Section 2.4.2. Development of appropriate requirements for GCD transmissivity are discussed in Section 3.4.3.1.

Baseliner Analysis

Veneer stability of the landfill baseliner prior to waste placement was assessed. The results of the analysis have been compared to published interface friction data at the appropriate load. This assessment has shown that the required values are attainable with materials that are currently available for the specified application. Minimum required interface strength parameters are as follows and will be verified prior to installation. Additional calculations are provided in Appendix H of this report.

- Interface 1 (Soil against geosynthetic) - Degree of Internal Friction Angle (φ) is 24.5 degrees
- Interface 2 (Geosythetic against geosynthetic) - Degree of Internal Friction Angle (φ) is 26.5 degrees

The minimum interface shear strengths shall be determined by laboratory testing of the actual liner materials used in construction. Materials that exhibit friction angles equal to greater than the values listed above are acceptable. However, depending upon the amount of cohesion (y-intercept) presented in the shear testing reports, the actual allowable friction could be lower than the values listed. If there is physical justification, such as textured geomembranes or geosynthetics with physical interlocking of soil having cohesions, the cohesion can be taken into account into assessment of the failure envelope. A detailed assessment of the shear strength laboratory results and approval by the Engineer is required prior to acceptance of material for use at the site.

Placement Reqirements

Construction methods used when placing liner soils is also critical in maintaining the stability of the baseliner system. It is recommended that the placement of the baseliner soil materials on 3H:1V slopes be completed as follows:

- By placing the baseliner soils from the bottom of the slope upward, a passive stabilizing soil wedge is established at the toe of the slope prior to placement of the soil higher on the slope. The operation of construction equipment over this lower wedge tends to compact and strengthen the wedge.
- Placement from the side of a baseliner cell area is acceptable provided the lower wedge is in-place prior to placement equipment running on the baseliner and provided the baseliner soils are placed in accordance with the other placement recommendations presented herein.
- Relatively small wide-track dozers (i.e., low ground pressure dozers) are recommended for placing the baseliner soil material. This type of equipment limits both the dynamic force imparted to the soils and geosynthetics during acceleration and braking and sharp turns and the tractive force applied through the dozer tracks.

Down-slope dynamic forces can be limited further by limiting the dozer speed on the slope and by instructing the dozer operator to avoid hard braking, particularly when backing down-slope.

3.5.2.9 Final Cover Slope Stability

Veneer stability calculations have been performed for the proposed geosynthetic cover. The proposed cover system will be installed over portions of the landfill that have been filled to final grade. The construction of the cover system incorporate multiple geosynthetic and soil interfaces. Minimum strength requirements for these interfaces have been developed that will provide an acceptable factor of safety and are provided herein. Stability analysis calculations are presented in Appendix H of this report.

Design Requirements

As required in 6NYCRR Part 363-4.3(c) the factor of safety against sliding for the final cover system must be at least 1.5. Therefore, the required factor of safety for interface stability is 1.5. The cover system will be installed on the waste with an approximate final design grade of 18.4°, which is equivalent to a 3 horizontal to 1 vertical (3H:1V) slope.

Although the majority of the cover system will consist of a relatively consistent thickness of soil placed on a consistent slope, some portions of the final cover system may be constructed with geosynthetics or soils being placed at a slope steeper than 3H:1V. These cases would be exterior slopes of landfill access roads and/or side slope drainage swales and would not impact the overall 3H:1V final waste grade.

Although not addressed in the regulations, the variations between peak and residual shear strengths of the interfaces have been considered. Although there is no industry 'standard', it is accepted practice to design interfaces with a minimum factor of safety of 1.5 using peak shear strengths and a minimum factor of safety of 1.25 using residual shear strengths. In addition to peak and residual factors of safety the final cover system was also designed to mainatain a factor of safety of 1.2 under seepage conditions and 1.3 when considering landfill gas pressure under the liner. Due to construction traffic on the slopes and settlement of the waste, it is possible the interfaces could be sheared or strained enough such that the residual shear strength governs. As this situation is usually localized and can be minimized by good construction practice, a safety factor lower than 1.5 using residual shear strengths is considered appropriate. The interface with the lowest peak shear strength (i.e., the one that would be the first to shear and be governed by residual shear strengths) is taken as the critical interface for residual shear strength factor of safety consideration. Calculations for both the peak and residual conditions are presented in Appendix H of this report.

The following critical interfaces have been reviewed:

- Barrier Protection Layer (BPL)/Drainage GCD
- Drainage GCD/Geomembrane
- Geomembrane/Landfill Gas Venting GCD
- Landfill Gas Venting GCD/Intermediate Cover

It was assumed that the drainage GCD and the landfill gas venting GCD will be a similar product with a similar geotextile cover on both sides. Therefore, the interfaces between both GCD layers and the geomembrane (texturing on both sides) will be the same. However, it is likely that the soil used for intermediate cover and barrier protection layer soil may be from various sources. Therefore, the soil to GCD interfaces should be tested using the actual soil source used at the time of construction.

The cover system must also be capable of transmitting water that infiltrates the BPL, and landfill gas that can exert upward pressure on the underside of the geomembrane, such that interface stability is not impacted. Development of appropriate transmission rates to accomplish these objectives is shown in the Final Cover Stability Calculation in Appendix H of this report.

Cover Analysis Parameters

For the purpose of performing veneer stability calculations, final cover is assumed to be constructed with a final slope of 3H:1V or 18.4°.

Due to the potential for pore water build-up in the soil layers above the impermeable geomembrane, a GCD layer will be installed directly above the geomembrane to drain stormwater impingement. In addition, a 60 foot by 60 foot grid of 4 inch pipes will be installed in the cap drainage layer to relieve porewater buildup. The cap drainage system will daylight at the toe of slope and be conveyed into the stormwater drainage system. This cap drainage system is shown in the Liner/Cap Drainage Calculation in Appendix H of this report. Without this layer, water would fill the pores within the cover soils resulting in a reduction of soil-to-soil or soil-to-geosynthetic normal forces and an increase in the driving force. The reduction in normal force is due to the soil mass becoming buoyant. The impact to the factor of safety is based on parallel seepage along the top surface of the geomembrane and can be seen in the calculations (Appendix H of this report). The result of this seepage is a reduction to the factor of safety. Therefore, the transmissivity of the GCD must be such that water passing through the vegetative cover and barrier protection layer will be adequate to drain the cover soils and prevent excessive build-up of pore pressures at the interfaces. The GCD should pass appropriate volumes of water without becoming full. The interface calculations presented herein assume this will be accounted for in the cover system design.

Similarly, on the underside of the relatively impermeable geomembrane, potential for the build-up of landfill gas pressure exists. Therefore, a GCD (gas venting layer) has been proposed to transmit gas from the underside of the geomembrane to landfill gas collection lines and wells. Typically, GCD's will provide a much higher degree of transmissivity for relief of gas pressures than that associated with the NYSDEC-required landfill gas-venting layer (12 inches of sand exhibiting a minimum permeability of $1x10^{-3}$ cm/s). The GCD will be tied to the active and passive system in accordance with the NYSDEC regulations. During active landfill gas collection (during operations and during a portion of the post-closure period) the landfill gas collection system will be operated in a manner that will provide a negative pressure below the geomembrane, minimizing the potential for positive gas pressure to reach the underside of the cover system. The system has risers located at regular intervals that extend down into the waste to collect/convey the landfill gas and are also tied to the landfill gas-venting layer. Once active landfill gas collection is discontinued, the landfill gas-venting layer will transmit landfill gas to a series of passive landfill gas vents. The amount of landfill gas pressure that is applied to the underside of the cover system during passive landfill gas collection is a function of the landfill gas generation rate, the spacing of the vents and the transmissivity of the venting layer. The calculations have assumed a maximum landfill gas pressure of 15.6 pounds per square foot (psf) or 3 inches of water column.

It is assumed that soil-to-soil interfaces will have the strength of the weakest soil. That is, due to intermixing of the layers during compaction there will be no defined 'interface' between the two soils and therefore the strength of the weakest soil will govern. Furthermore, it should be noted that the soil strengths discussed herein are peak strengths only, as movement of the soil (i.e., residual condition) is not anticipated to be the critical mode of instability. Based on historical soil property data at the site (both on-site and imported soils) a minimum angle of internal friction for the vegetative cover, BPL and intermediate cover layers of 26.5° has been assumed for these calculations.

Final Cover Analysis

The results of this analysis have been compared to published interface friction data at the appropriate load. This assessment has shown that the required values are attainable with materials that are currently available for the specified application. The results of the analysis are provided in the table below. Additional calculations are provided in Appendix H of this report.

Interface	Minimum Friction Angle
1- Soil against uppermost geosynthetic layer	32.2
2- Between two geosynthetic layers	27.7
3- Soil against the lowest geosynthetic layer	26.5

The minimum required interface strength parameters will need to be field verified prior to installation.

Placement Requirements

Construction methods used when placing cover soils is also critical in maintaining the stability of the cover system. It is recommended that the placement of the cover soil materials be completed as follows:

- By placing the cover soils from the bottom of the slope upward, a passive stabilizing soil wedge is established at the toe of the slope prior to placement of the soil higher on the slope. The operation of construction equipment over this lower wedge tends to compact and strengthen the wedge.
- Placement from the side of a cap area is acceptable provided the lower wedge is in-place prior to placement equipment running on the cover and provided the cover soils are placed in accordance the other placement recommendations presented herein.
- Relatively small wide-track dozers (i.e., low ground pressure dozers) are recommended for placing the soil cover material. This type of equipment limits both the dynamic force imparted to the soils and geosynthetics during acceleration and braking and sharp turns and the tractive force applied through the dozer tracks.
- Down-slope dynamic forces can be limited further by limiting the dozer speed on the slope and by instructing the dozer operator to avoid hard braking, particularly when backing down-slope.

Analyses of specific equipment outside of the general recommendations (i.e., small pad foot rollers) provided herein can be performed if requested. Upon completion of the cover system the system should be regularly inspected and maintained to minimize the impact of subsidence and erosion on the stability of the system.

3.5.2.10 Stability of Swale Embankments

It is assumed that the waste under the cover system will generally not be steeper than a 3H:1V slope for the final condition and that the cover system will act as a veneer on that waste at that 3H:1V slope. However, there will be cases where soils above the waste are placed steeper than 3H:1V. The primary example of this condition is the placement of side slope swales above the liner.

The side slope swales will generally consist of a 2-foot-high low permeable soil berm placed at a 2H:1V slope on the top of the cover system. The limited fills required to create these swales will increase the driving forces on the liner system and slightly reduce the factor of safety for the cover system directly below the swale. However, in has been Cornerstone’s experience that swales 2.5 feet in height or less have minimal impact on overall stability of the final cover system (i.e., longer length of slopes).

Stability of the proposed swales has been calculated with a factor of safety of 1.5 using the Slope/W program, the resultant calculation is included in Appendix H of this report.

The swales will be lined to minimize potential for infiltration of collected stormwater into the barrier protection layer and avoid a concentrated feed of moisture to the GCD.

3.5.3 Settlement Analysis

3.5.3.1 General

The intent of this settlement analysis is to support the design of the leachate collection and baseliner systems and to verify minimally required post-settlement slopes specified in 6NYCRR Part 363-4.3(b). Specifically, the requirements are that post-settlement slopes for leachate collection piping are at least 1% and, for leachate collection systems (as well as liner and subgrade), are at least 2%.

3.5.3.2 Selection of Critical Settlement Analysis Locations

The sections where the post-settlement slope was evaluated, as well as the points analyzed along the sections, have all been chosen according to the same criteria. These include:

- geometry - local and global subgrade minimum, local and global subgrade maximum, change in subgrade slope, sump locations, etc.
- geology - significant changes in underlying soil type, thickness of specific units, or total overburden thickness

The landfill geometry was determined based on existing landfill features and standard landfill geometry allowed by NYSDEC regulation. The location and structure of the existing landfill footprint are based on record drawings and surveys. The soil conditions are based upon historical site investigation. The locations of selected sections are presented in the Settlement Calculation in Appendix H of this report.

As the result of the total predicted settlement is greater than 1.0 foot, a settlement monitoring plan is required in order to meet the requirements of 6NYCRR Part 363-4.3(b)(3). More specifically the regulations require demonstration that the leachate collection and removal system is functioning as designed and the integrity of key collection appurtenances is maintained. The functionality and integrity of these features will be demonstrated through the regular inspection and monitoring of the leachate collection system as outlined in the Facility Manual. The functionality of the various leachate collection systems is verified as part of the regular daily and monthly site inspections and leachate flows to the collection sumps (primary and secondary) are monitored daily. Routine maintenance will consist, at a minimum, of annual jetting or flushing of laterals and headers in the primary leachate collection system. The collection system (primary and secondary) will also be video inspected every other year. These activities will be documented and evidence of clogging or system impact will be assessed and action plans developed as needed.

3.5.3.3 Analysis Procedure

Post-settlement elevations, employed in calculation of post-settlement slope, were determined by subtracting the expected subgrade settlement from the proposed subgrade elevation at points along critical sections. The calculation of anticipated settlement was implemented according to the generalized equation for settlement, which can be written as:

$$\Delta H = \Delta H_i + \Delta H_c + \Delta H_s$$

Where:

ΔH = total settlement
ΔH_i = immediate (elastic) settlement
ΔH_c = consolidation settlement
ΔH_s = secondary (consolidation) settlement

Either the elastic or consolidation settlement tends to dominate the settlement based on the properties of the overburden found on the site. For coarse-grained materials that easily dissipate excess pore pressures, immediate elastic settlement comprises the majority of settlement. Conversely fine-grained soils normally require a significantly greater length of time to dissipate excess pore water pressures created by additional loading. This renders the soil nearly inelastic as initial loads are, as exhibited by sustained increases in pore pressures, borne almost exclusively by water, which is relatively incompressible. Laboratory tests generally indicate that the Glaciolacustrine units, as well as the soils used for compacted fills generally behave as finer-grained soils and therefore consolidation settlement will define the subgrade displacement for these soils. These soils will also undergo secondary consolidation, indicating the long- term reorganization of soil particles under the applied stress. The settlement in solid waste is also described well using the consolidation settlement method. The one exception is the LGT layer which has consistently shown lower plasticity, lower void volumes, and higher content of coarse-grained particles. For this reason, the elastic (immediate) settlement analysis is most appropriate for the lowest soil layer. A determination of the method applicable to each soil type and the equations used for each case are found in Appendix H of this report.

In accordance with standard engineering practice, a factor of safety of 2.0 for the landfill subgrade analyses was applied by doubling the unit weight for waste in the analyses.

3.5.3.4 Selection of Settlement Parameters

Primary settlement parameters were derived from historical settlement analysis data as summarized below and as seen in Appendix H of this report. Secondary consolidation, for the layers to which it applies, was generally equal to 10% of primary settlement.

Due to the non-cohesive nature of this soil, settlement in the Lower Glacial Till is considered to be completely elastic (immediate). The soil parameter necessary to calculate the settlement is the stress-strain modulus, which has been determined from literature correlations to the moisture content and Atterberg limits tested on the LGT. This correlation results in a modulus of 8.24×10^5 psf. The LGT is not considered to have secondary settlement.

The parameters for the Upper and Lower Glaciolacustrine clay as well as the Upper Glacial Till were conservatively chosen consistent with previous settlement analysis data. The values for the Glaciocustrine and Upper Glacial layers are as follows:

Material Name	Initial Void Ratio, e_0	Consol. Index, C_c	Reconsol. Index, C_r	Moisture Unit Weight [pcf]	OCR	Secondary Consol. Index, C_a
UGL	0.7	0.4	0.004	120	1.8	0.02
UGT	0.3	0.07	0.02	140	1.8	0.0035
LGL	0.7	0.12	0.02	130	1.8	0.006

The compacted fill is placed in a controlled manner with specified lift thicknesses and a consistent compaction procedure. The consolidation index (0.08), reconsolidation index (0.02), and secondary consolidation index (0.004) are based on the properties of a remolded clay. The initial void ratio of 0.5 is suggested for fill with liquid and plastic limits resembling typical fill. As the fill is placed in lifts of controlled thickness and compacted uniformly, it is more appropriate to employ a preconsolidation pressure instead of an overconsolidation ratio. The preconsolidation pressure was calculated according to general practices and properties of equipment used for soil compaction. The preconsolidation pressure of 15,000 psf is based on an expected value within a standard twelve (12) inch soil lift.

As portions of the project will include overfilling an existing landfill, consolidation properties for existing waste must also be considered. It is generally presumed that previous compaction of the waste has prepared immediate settlement to be negligible in comparison to delayed settlement. The average consolidation ratio for the existing waste is taken as approximately 0.20 as indicated in Appendix H of this report. As this is the consolidation ratio (also known as modified consolidation index) the void ratio is not necessary. Furthermore, it is appropriate and conservative to assume that the waste will be normally consolidated and will therefore not require a reconsolidation index. Decompositional settlement is generally modeled as secondary settlement for Municipal Solid Waste. A modified secondary consolidation index (C_a) of 0.04 was used.

3.5.3.5 Baseliner and Leachate Collection Pipe Settlement Analysis

Using the above methods and parameters, the subgrade settlement was evaluated along nineteen sections located within the surrounding existing landfill and along five sections within the proposed development. The location of each section may be found in Appendix H of this report. The twenty four critical sections were analyzed in the surrounding landfill areas to assess the settlement associated with the placement of additional fill over these areas. The sections were assessed for post-settlement slope of the longest drainage path in their respective cells. Sections W-2, W-3, W-5, 6-2, 7-2, S-2, S-4, and S-1-3 show that post-settlement slopes along the floor of the surrounding areas will equal or exceed 2.0%.

The remaining sections, W-1, W-4, W-6, 6-1, 6-3, 5-1, L-1, L-2, 7-1, 7-3, S-1, S-3, S-2-4, S-3-3, S-4-3, and S-5-4 were chosen to determine post-settlement slope of representative leachate collection pipes. These sections show that the leachate collection pipes will have a post-settlement slope exceeding 1.0%. The calculation is available in Appendix H of this report.

3.5.3.6 Baseliner Strain

Additional analysis related to the anticipated subgrade settlement was performed to assess the elongation strain in the proposed baseliner system geosynthetics. A generally accepted industry standard is that the elongation strain should be limited to 5% or less. To determine the maximum potential baseliner strain, baseliner intervals from the Settlement Calculation were evaluated for baseliner strain.

The maximum anticipated strain, according to the calculations included in Appendix H of this report, will be 0.36%. This represents the greatest value found from the settlement section, and results in an anticipated strain less than the industry accepted 5%.

3.5.4 Bearing Capacity Analysis

The bearing capacity analysis was conducted to determine the factor of safety against failure of the subgrade due to subsidence induced by the expected loading. The analysis was performed at the location with the greatest waste thickness over the most subsidence-prone soils. The calculation method employed in the Bearing Capacity Calculation in Appendix H of this report indicates that bearing capacity failure is primarily dependent on soil properties, while landfill height has a smaller influence on the bearing capacity factor of safety. Thus an acceptable factor of safety in areas with low-strength soils indicates that waste over stronger soils should not have an equal or greater factor of safety, even with greater waste thicknesses.

The proposed waste thickness at the point chosen for analysis is approximately 370 feet, and the calculation was conducted using the assumption that the footing area in question will be a 800 feet wide by 2700 feet long under the waste. Neglecting the slight decrease in landfill height over the adjacent unit squares, the weight of the counteracting soil wedge ($\overline{q}$) will be approximately equal to the stress induced by the waste directly over the unit square being analyzed.

The required factor of safety against bearing capacity failure is 2.0 for all configurations and all cases. This requirement is in accordance with 6NYCRR Part 363-4.3(c)(3) regulations. The factor of safety against drained bearing capacity failure is 80.6 and the factor of safety against undrained bearing capacity is 2.22. Therefore, the regulatory minimum is satisfied.

3.6 SOIL BALANCE ANALYSIS

Construction of the landfill will require excavation and fill to form the proposed landfill grading. Construction of the proposed SMI Valley Infill will require a gross cut of approximately 560,000 +/- cy and gross fill of approximately 5,300,000 +/- cy. The summary calculations are presented in Appendix H of this report. Overall the landfill construction will yield a net fill on the order of approximately 5,000,000 +/- cy; however, when utilizing on-site borrow areas (Meadow View Mine) the required off site fill required will be reduced to approximately 1,700,000 +/- cy. Based on the estimated site life for the landfill development the required fill materials are estimated to be hauled in from off site sources at a rate of approximately 113,000 +/- cy per year which is a reduction of approximately 91,000 cy per year under SMI's current operating conditions.

Excavated Soils will be used for construction and operational soils. To the extent the excavated soils are suitable for landfill construction, they will be incorporated into the project. Excess soil or soil that is not suitable for landfill construction will be stockpiled for future use as landfill operational and cover soils.

3.7 LEACHATE MANAGEMENT SYSTEM DESIGN

3.7.1 General

One of the key aspects of managing a sanitary landfill is leachate management. The landfill design includes various measures to minimize and contain leachate and then collect and manage the liquid that contacts waste and is collected in the landfill containment system. The purpose of a low permeability liner system is to restrict the migration of liquids to or from the contained waste. Intermediate cover placement offers a reduction in the generation of leachate during operations, while a low permeability final cover is applied to even further restrict the generation of leachate after closure of the landfill. The leachate collection system, in part, is designed to effectively prevent the buildup of liquid on the low permeability liner system below. The minimization of this "head" on the liner system, by limiting the infiltration of liquids through operational practices and by providing efficient removal, will help to protect the environment. The following sections describe the various practices for leachate management that are used in the currently permitted landfill operation.

3.7.2 Leachate Generation

Leachate generation was estimated using existing leachate generation data in conjunction with the Hydrologic Evaluation of Landfill Performace (HELP) Model, Version 3 (HELP-3).

3.7.2.1 Existing Leachate Generation Data

Actual leachate collection rates from the WEX and NEX areas as well as the SELF bumpout provide useful information regarding leachate generation. Seneca Meadows compiled leachate data over a period of approximately nine years (2009 to 2018), from the representative expansion areas listed above. Utilizing a memorandum by the engineer for Seneca Meadows, Inc. to the NYSDEC dated April 30, 2003, an analysis of the most recent landfilled areas was performed. The details of this analysis as well as the reference memorandum are presented in Appendix H of this report.

Initial operations are defined as the time period when minimal waste thickness exists over a constructed liner area. During this time period the presence of a limited quantity of waste limits the moisture holding capacity available to reduce leachate generation rates and portions of the porous leachate collection/tire chip layer where stormwater cannot be effectively diverted are directly exposed to precipitation. Therefore, initial operations is the time period during which leachate generation rates would be at their maximum.

Active operations represent conditions where waste covers the entire area of baseliner, the moisture holding capacity of the waste reduces net infiltration to the primary leachate collection system, and only daily cover exists on the landfill operating areas. This condition yields leachate generation rates considerably less than initial operations.

The safety factor of 2.0 is applied to account for variability in actual leachate generation rates and potential influences of leachate recirculation so that the collection system can be designed to maintain the required maximum head buildup. The safety factor adjusted leachate generation rates are, therefore, used to design leachate conveyance systems, which must consider short term peaks. However, the 95% of the leachate generation values are representative of attenuated peaks and are used for longer-term estimates of leachate generation (e.g., to estimate storage requirements). The historic leachate generation is used primarily for leachate storage calculations where accurate monthly generation is important and the peak daily flow is not required, The historic leachate generation rates are summarized below with additional calculations provided in Appendix H of this report:

Cell Condition	Leachate Generation (gpad)
Initial Cell Start-up	2300
Active Waste Placement/Daily Cover	700
Intermediate Cover	530
Geomembane Cover	20
Clay Cover	100

In addition to the above table a HELP model has been developed and a calculation of the liquid generated in an open cell from the design storm has been conducted.

3.7.2.2 HELP Model

The HELP Model was developed at the U.S. Army Corps of Engineers Waterways Experiment Station (WES) in Vicksburg, Mississippi specifically for the evaluation of landfill designs and operations. The model incorporates a routine for generating a 25-year climatological database based on daily, monthly and yearly values for mean temperature, solar radiation, precipitation and evapotranspiration. This routine was used to generate synthetic climatological data representing these four parameters, based on the Syracuse default data. The data sets are representative of a 25-year period.

As the historical leachate generation data provides a more site specific estimate of leachate generation the historic values were used to design the leachate storage system and estimate disposal quantities. For the sizing of the leachate collection system and to check the head on the primary liner the HELP model was used for providing a peak value. The model was run using normal operating conditions with active waste placement and 6 inches of daily cover.

The peak daily leachate generation from the HELP model was 0.147 inches (3980 gpad) which was used to size the leachate collection system. The annual generation was calculated to be 8.55 inches (232,000 gallons) which is not significantly lower than the historical data at 255,500 gallons and shows the correlation between the historical data and the model. Because the HELP model can evaluate the peak daily impact, the HELP model was used for determining the peak daily head and verifying that the head above the primary liner was not greater than 12 inches. The results show the peak head is 1.16 inches which satisfies the requirements.

The HELP model was also used to analyze the impact of the 25-year 24-hour storm. For this evaluation one year was run instead of 25-years. That daily precipitation rate generated by the model was then modified by changing the day with the highest precipitation (day 194 with 1.11 inches) to 3.92 inches which is the 25-year, 24-hour storm. The model was run with only 10 feet of waste in place. The peak daily generation rate was 0.153 inches (4150 gpad) and the annual generation was 9.25 inches (251,000 gallons); values that are not significantly higher than average which shows how even 10 feet of waste can dampen out peaks do to evaporation, run-off and absorption. The peak daily head was 1.21 inches which was less than the maximum allowable 12 inches.The results of the HELP Model analysis are presented in Appendix H of this report.

A similar analysis using the 500-year 24-storm would indicate that the head on the liner would not be more than 12 inches and the leachate could be fully contained within the lined area.

3.7.2.3 Proposed Landfill Leachate Generation Estimate

A leachate generation estimate has been developed for the proposed expansion to evaluate the logistics of off-site leachate management and the sizing of storage facilities. The leachate generation estimate is based on leachate generation estimates presented in the preceding section.

The leachate generation estimates for site operations have been prepared by applying the previously described percolation rates over the initial operations areas, active operations areas, intermediate cover areas, and final cover areas. It was determined that the greatest leachate generation will occur during Phase 2 of the landfill construction with a daily leachate generation of 124,000 gallons per day.

As the HELP model was only run under active waste placement conditions the leachate generation from a newly opened cell was also calculated. The largest phase (Phase 1) was analyzed under 25-year 24-hour storm conditions (3.92”) assuming the entire phase (13.8 acres) had received the first placement of waste with zero runoff. Under this scenario 1,468,000 gallons of leachate would be produced. Additionally the leachate produced after 1 lift, 10 feet, of waste placement was calculated. Under this scenario 55,000 gallons of leachate would be produced. These values were used to size the peak removal capacity for the leachate pumping systems to remove the volume of leachate in less than the 7 days required by the regulations. The site currently maintains adequate storage for the 1,468,000 gallons of leachate as described in Section 3.7.6.

The estimates of leachate generation rates during the life of the landfill and at closure are presented in Appendix H of this report.

3.7.3 Leachate Collection Pipes

Once the primary collection system drainage layer (i.e., No. 1 stone) has conveyed leachate to the pipe drains placed in the low points of the baseliner configuration, they convey flow to the sumps. The lateral and header pipe for the primary leachate collection system will consist of eight-inch diameter perforated PVC pipe. An eight-inch diameter perforated PVC pipe is proposed for the secondary leachate collection system.

The purpose of the leachate collection piping system is to transfer leachate from the collection system (e.g., drainage media) into the leachate conveyance systems. The porewater/leachate piping systems convey liquids to the respective collection sumps within each cell. Leachate collection pipes are also installed along the toe of slopes located on side or down gradient locations of the liner system and at flow convergence locations on the liner floor. The required leachate collection piping spacing is a function of leachate impingement rates, the flow path, and system permeability. The landfill liner systems have been designed with higher permeability leachate collection media (e.g., No. 1 stone in primary systems, use of GCD in secondary leachate collection systems), allowing for minimization of leachate collection piping along the floor.

The leachate collection pipes were sized using a pre determined spacing to develop a drainage area. A minimum primary pipe diameter of 8 inches was chosen to allow for inspections and to meet regulations. A minimum secondary pipe diameter of 8 inches was chosen to comply with regulations and to provide for the required camera inspections. The factor of safety for the pipe reaching capacity is 7.5 and 25.8 for the primary and secondary systems, respectively. Additional calculations are provided in Appendix H of this report. The leachate head will be maintained below a maximum of 12 inches with collection pipes located only along the perimeter areas of the liner system.

Leachate collection pipe cleanouts will be connected to the leachate collection pipes located on the floor of the liner system to facilitate inspection and cleaning of the pipes. Cleanouts will consist of similar diameter piping installed on the sideslopes of the leachate collection systems. Primary and secondary leachate collection pipe cleanout locations are depicted on Sheets 8 & 9 of the Engineering Drawings.

3.7.3.1 Leachate Collection Pipe Loading

The leachate collection pipe has been assessed to confirm that it will retain its structural and functional integrity throughout the lifetime of the landfill. Two conditions have been assessed. These conditions include:

- the newly proposed leachate colletion pipe that will be installed as a component of the landfill development, and
- the existing leachate collection pipes in landfill cells that will be overlined and/or overfilled.

The flexible pipe/soil system proposed for use has been designed to withstand the physical stresses imposed during construction, normal waste placement operations and over the post closure period. The criteria used to evaluate the physical characteristics necessary for the PVC pipe/soil system include:

Ring Deflection

Ring deflection in flexible PVC Pipe is nearly elliptical and the vertical and horizontal deflections are nearly equal for small deflections. By transposing the Modified Iowa Deflection Equation the ring deflection can be calculated as follows:

$$\frac{\Delta Y}{D_m} = \frac{L_f K P_T R^3}{E_p I + 0.061 E_s R^3}$$

Where:

ΔY = Vertical Pipe Deflection
D_m = Mean Pipe Diameter
L_f = Deflection Lag Factor
K = Bedding Constant
R = Mean Pipe Radius
P_T = Total Pipe Load
E_p = Pipe Modulus of Elasticity
E_s = Modulus of Soil Reaction
I = Moment of Inertia

Pipe Buckling

Pipe buckling is determined by comparing the critical buckling pressure of the pipe to the total load on the pipe. The critical buckling pressure can be calculated as follows:

$$P_{cr} = 0.55\left[E_S \frac{E_P I}{0.149 R^3}\right]^{1/2}$$

Where:

P_{cr} = Critical Buckling Pressure
E_p = Pipe Modulus of Elasticity
E_s = Modulus of Soil Reaction
I = Moment of Inertia

Wall Crushing

Wall crushing is determined by comparing the axial stress applied to the pipe with the intial and long term compressive yield strength. The long term compressive yield value was determined for a 50 year service life. The applied axial stress can be calculated as follows:

$$P_{SA} = \frac{P_T D_m}{2A_W}$$

Where:

P_{sa} = Axial Stress
P_T = Total Pipe Load
A_w = Area of Pipe Wall
D_m = Mean Pipe Diameter

PVC pipe is proposed for the new landfill development. In addition to the long-term considerations associated with waste filling, the piping is designed to resist stresses during baseliner construction. Equipment traffic above the pipe will however be limited to small and/or low ground pressure equipment as approved by the Engineer.

Proposed Leachate Collection Pipe

The proposed leachate collection pipes for the landfill development were analyzed under the proposed loading conditions. The ranges for the calculated long term (50 years) and short term factors of safety of the pipes are as follows:

	Ring Deflection	Pipe Buckling	Wall Crushing
Calculated Initial Factor of Safety	4.3 – 14.2	5.6 – 20.7	3.4 - 21.7
Calculated Long-term Factor of Safety	3.5 – 9.8	3.2 - 11.7	1.7 – 8.4

The pipe critera for each of the pipes analyzed meets the factor of safey of 1.25, the industry standard for pipe performance.

Existing Leachate Collection Pipes

The existing leachate collection pipes for the Tantalo Site and the Part 360 compliant SMI Cells were analyzed under the proposed development loading conditions. The ranges for the calculated long term (50 years) and short term factors of safety of the pipes are as follows:

	Ring Deflection	Pipe Buckling	Wall Crushing
Calculated Initial Factor of Safety	3.9 - 5.2	5.1 - 7.6	6.2 - 7.9
Calculated Long-term Factor of Safety	3.2 - 3.6	2.9 - 4.3	2.4 - 3.1

The pipe critera for each of the pipes analyzed meets the factor of safey of 1.25, the industry standard for pipe performance.

Some small portion of the leachate from the perimeter sideslopes will be directed to existing cells. An evaluation was performed to check that these cells could handle additional leachate. Cell 6 has the longest potential leachate travel distance so that cell was used for the evaluation. We calculated that approximately 0.73 acres of the expansion perimeter sideslope contributes to Cell 6.

The analysis included a reevaluation of the same calculations used for Cell 6 with the additional contributory area and includes:

- The capacity of the secondary system to handle another 1000 gal per acre per day for the additional sideslope
- The travel time with the additional length from the sideslope
- The capacity of the primary sideslope sand layer which is now extended
- The capacity of the primary floor with additional leachate

The analysis assumes leachate generation from the sideslope is 4,600 gpad which is representative of a first lift of waste and assume leachate generation over the floor of 1,400 gpad since significant waste is already placed in Cell 6. The transmissivities and permeabilities for the completed Cell 6 are based on actual test values for the materials.

The analyses concludes that all aspects meet the design criteria, specifically:

- Using the same geocomposite over the sideslope portion of the infill, the infill/Cell 6 sideslope secondary system has a maximum head of 3.7 mm which is less than the 5 mm thickness of the geocomposite.
- On the Cell 6 floor the maximum head in the secondary system is 0.0312 inches which is less than the maximum 1 inch criteria
- The maximum time of travel from the top of sidelslope in the infill to the Cell 6 sump is 18.8 hours; less than the allowable maximum of 24 hours.
- The maximum head on the primary sideslope is 0.24 feet; less than the 1 foot maximum criteria
- The maximum head on the primary floor is 0.26 feet; less than the 1 foot maximum criteria.

Additional calculations are provided as in Appendix H of this report.

3.7.3.2 Leachate Conveyance System

Currently, force mains exist that transfer leachate from landfill leachate collection wells and the existing landfill areas. The entire system is comprised of heat fused HDPE pipe, double-contained wherever it exits the landfill containment system. To facilitate force main access for routine maintenance and cleaning, manholes are located along the existing landfill conveyance system pipeline. These manholes are prefabricated concrete structures equipped with lightweight, lockable hatches and ladders for ease of access.

Outside the landfill footprint, leak detection risers are located along the conveyance pipe to detect the presence of liquid in the HDPE containment pipe. The dual containment pipe is sloped so that liquid present in the containment pipe will gravity feed to the leak detection risers located at low points of the system. Routine inspections of these risers will be performed as presented in the Facility Manual.

The proposed landfill areas are designed to convey leachate to the Leachate Storage and Treatment facility using the existing systems noted above. The design of the proposed force mains parallels that of the existing systems. The proposed systems include heat-fused HDPE, dual contained pipe, manholes, and leak detection risers. The conveyance system plan view, profiles, and details are illustrated on the Engineering Drawings, Sheets 8, 9, 23, and 25 to 29.

3.7.4 Stormwater Diversion

SMI's leachate management plan outlines procedures for minimizing leachate generation. As illustrated in the design, stormwater runoff that has not contacted waste will be diverted away from the leachate collection facilities to the maximum extent practicable. Further, SMI, as a component of its daily operations, will place intermediate cover over inactive fill areas. This intermediate cover placement not only reduces leachate infiltration but assists in controlling odors. Final cover will be placed over areas as soon as practicable after they have reached final grade.

Consistent with SMI's current operating practices, up to approximately 20 acres would typically be under daily cover conditions (i.e., the active operating area), with the remainder of the lined area having intermediate or final cover with interim or permanent stormwater diversion structures in place.

3.7.5 Collection Sumps and Siderisers

The proposed landfill areas are divided into a total of three separate drainage sub-areas from which leachate generation and secondary collection system monitoring can be performed separately. The leachate collection pipes placed within the primary leachate collection layer and the secondary collection layer respectively, will convey collected liquids to one of the two sumps for removal along the southern perimeter berm, one sump for Phases 1 and 2 and the other sump for Phases 3 through 6. At each sump location, leachate collection pipes will discharge into a leachate collection tee, that will be connected to the associated primary and secondary leachate collection sideriser pipes, consisting of 24-inch diameter HDPE pipe. The sideriser pipes provide access to the sumps from the sideriser station. Each sump will contain a submersible pump to convey leachate through a discharge hose into the respective perimeter conveyance system or to recirculation, as detailed in the Facility Manual (Appendix E of this report). The pump systems will be equipped with level controllers to enable and disable the pumps, and a high level alarm controller to indicate possible pump failure. The primary and secondary sideriser pipes at each sump location will daylight into a sideriser station at the top of the perimeter embankments.

The porewater drain system sumps will be located at the same low points as the primary and secondary systems. These porewater sumps, located beneath the primary and secondary leachate collection sumps, will consist of a PVC tee connected to a sideriser pipe that will connect the sump to the sideriser station. The porewater sump will be fitted with a submersible pump for groundwater removal. Groundwater will be discharged into the perimeter stormwater system. Historical groundwater quality data has shown that the operation of the landfill is not contributing to the introduction of contaminants that will be manifested in the porewater system. Absent the introduction of contaminants, the porewater would reasonably be discharged to the surface water drainage features, which will in turn be routed to retention basins where water quality would be tested.. If, however, monitoring data suggest that contaminants may be introduced to the porewater system, the porewater would be routed to the leachate management system for disposal, and the piping arrangements are designed for this potentiality.

The locations of the sumps and details of construction are illustrated on the Engineering Drawings, Sheets 8, 9 and 25 to 27.

3.7.5.1 Sideriser Stations

Each sideriser station serves to house the monitoring equipment necessary to conform to the requirements of 6NYCRR Part 363-4.6(f)(8)(iii). This citation requires that the landfill's primary and secondary leachate collection and removal systems include leachate flow monitoring devices and controls to effectively quantify flow rates.

Flow conveyed by each of the primary and secondary collection sumps will be independently measured within the sideriser station prior to being combined into the site-wide leachate conveyance system. Flow sensors will transmit flow information through a relay located within each station, and the data will be continuously recorded. At sump locations where a porewater drain is present, the flow monitoring and recording capability will also be provided for the porewater system. This system will also record the data required to compute the rate of leakage into the secondary leachate collection system for comparison with the allowable leakage rate. Sample ports will be provided to permit independent sampling of the liquids removed from the primary, secondary and porewater systems.

During routine operations, pump on/off liquid level controllers and high level alarms for each of the primary and secondary leachate collection sumps will provide automatic evacuation of collected liquids. Manual "hand/off/auto" (HOA) switches will also be provided within the sideriser stations. Level and flow controls will also be installed for the porewater drain sumps. Leachate and groundwater conveyance will be controlled by valving located within the individual sideriser stations. The locations of the sideriser stations and the details of construction are shown on the Engineering Drawings, Sheets 28 and 29.

3.7.6 Leachate Storage and Removal Systems

The Leachate Storage and Treatment Facility consist of two nominal 500,000 gallon steel storage tanks, a 1,772,000 gallon steel storage tank, and 110,000 gallon steel storage tank. Each 500,000 gallon leachate storage tank has a usable capacity of 485,000 gallons, for a total usable capacity of 970,000 gallons. The usable volume allows for an additional capacity equivalent to one foot of freeboard. The 1,772,000 gallon tank has two feet of freeboard as does the 110,000 gallon tank. All the tanks are covered and the freeboard should not be impacted by rainfall events. The three smaller tanks are located within a secondary containment berm capable of holding 110 percent of the tank’s total capacity. The larger tank is ringed by a second steel tank that is open to the atmosphere and has a capacity of 2,051,000 million gallons, or approximately 116 percent of the primary tank’s total capacity. The 1,772,000 and 110,000 gallon tanks will be dedicated to temporary storage of leachate concentrate generated by the on-site reverse osmosis treatment (RO) system.

The existing Leachate Storage and Treatment Facility allows leachate to be transferred in one of two ways. Leachate can be transferred from the conveyance lines to the storage tanks and then from the storage tank to a leachate truck or to the Seneca Falls sewer line. This process is controlled using a series of actuated valves linked to a control panel at the facility. In addition SMI has the ability to transfer liquid between the storage tanks. Leachate concentrate is conveyed from the RO system directly to the one of the dedicated storage tanks and then is transferred to the loadout facility as needed for trucked disposal. A flow meter is located within the facility for recording leachate disposal quantities.

The containment pad for the tank trailer staging area is positioned adjacent to the leachate transfer building, and consists of a six inch thick curbed concrete structure sloping to a center drain. Liquid collected in the center drain flows into a collection sump in the transfer building, where the liquid is automatically pumped into the storage tanks. The sump is equipped with liquid level controls to enable and disable the pump, and a high level alarm to warn of potential pump failure.

The Leachate Storage and Treatment Facility piping arrangement is configured to allow for routine maintenance of the storage tanks while continuing to collect and transport leachate off site for treatment. The concentrate tank will be taken off line for inspections as needed with accommodations for managing the concentrate through temporary storage in the other tanks, management of the reverse osmosis production rates, or increased short term trucking.

The leachate storage tanks are equipped with a sonic level sensor mounted on the top of the tank, manways (permit confined space) to facilitate access for inspection and cleanout. The three smaller tanks are equipped with a leak detection riser connected to a drain within the tank foundations. Visual observations of the secondary containment area are performed for the larger tank. Inspections of the leachate storage tanks will be carried out weekly and the results are documented.

Precipitation within the tank's secondary containment will be managed in accordance with the Spill Prevention Control and Countermeasures (SPCC) Plan. This precipitation may be in the form of rainfall or snow. The secondary containment will be inspected weekly and following precipitation events. Since the secondary containment will be drained following appreciable rainfall events, the buildup of ice is not expected to impact the containment capacity. The removal of snow will occur as soon as practical but not more than 24 hours after the event has occurred. Onsite equipment and labor including front-end loaders/skid steer type units will be used to remove snow from the leachate storage tank secondary containment.

3.7.6.1 Leachate Transportation and Disposal

Leachate will be transported off site using 8,000 gallon tank transfer trailers. Seneca Meadows currently holds agreements with the Village of Seneca Falls Wastewater Treatment Plant (sewer connection) and several other POTWs to accept leachate from the facility. SMI has arranged disposal agreements with multiple facilities to provide both routine and contingency disposal capacity that is greater than the expected leachate generation rates.

SMI maintains contracts with local hauler and service providers and also owns a vac truck to manage the needed leachate pumping and hauling services at the site. In the event that there is a need for additional hauling capacity, SMI may utilize its 4,000-gallon vacuum truck or contact additional service providers. Contract negotiations will be initiated 60 days prior to their expiration to allow the identification of alternate haulers, if necessary.

3.7.6.2 Leachate Treatment Facility

Up to 120,000 gallons per day of pumped leachate that is brought to the on-site storage tanks can be treated by an on-site reverse osmosis treatment system operated by SMI. Two byproducts result from the reverse osmosis treatment system: the permeate; and the concentrate. The permeate is generally discharged into the local POTW forcemain. The concentrate is temporarily stored then recirculated through the active working face or trucked off-site to surrounding POTW facilities. The leachate concentrate is stored as noted in Section 3.7.6

3.7.6.3 Leachate Storage Design

Based on the calculations presented in Appendix H of this report for future leachate generation quantities, the maximum estimated daily leachate generation rate, including accounting for initial operations conditions, is 124,000 gallons per day. This value was calculated using impingement rates based upon historical data and applied to various operation stages of landfill development. This is below the contracted capacity that SMI has established for leachate disposal. Furthermore, these leachate generation quantities are largely based on operating data from the existing Facility, not on estimates from other facilities or reasonable tools for projection, as would be the case for a new facility without an operating history. Therefore, the degree of certainty in the estimates is enhanced.

The storage tanks provide more of a staging area than storage due to the relatively rapid turnaround time between collection and disposal. The 6NYCRR Part 360 regulations state that the minimum design capacity must be based on the proposed leachate generation rate and must be capable of containing a minimum of three months combined flow during the initial start-up condition of the landfill unless otherwise approved by the Department. However, for the reasons stated above and further discussions presented below, SMI is proposing the use of the existing leachate storage tanks at the facility.

SMI's leachate management plan outlines contingencies for leachate storage, transportation and disposal, with the ability to use any or all of the disposal facilities and contingency trucking capability also available, all as described in the Facility Manual, Appendix E of this report.

In the event of a contingency where leachate disposal was interrupted, the two 500,000 gallon and 1,722,000 gallon leachate tanks (2,692,000 gallons operating capacity) will be utilized to provide contingency storage of twenty one days at the maximum expected leachate generation rate. The storage time afforded by the tanks will allow SMI ample time to locate additional services, if necessary.

3.7.7 Leachate Recirculation

Leachate recirculation is the reintroduction of landfill leachate into the lined landfill cells. Leachate recirculation provides a means of optimizing environmental conditions within the landfill to provide enhanced stabilization of landfill waste, as well as treatment of moisture moving through the waste mass.

Seneca Meadows will potentially use leachate recirculation during the Landfill Development consistent with the current Facility Manual. Operationally, recirculation by the two basic methods will be performed as follows:

- Horizontal recirculation lines will be dosed initially at a rate equal to the storage volume of the pipe and surrounding stone media. Pressure will also be monitored as close to the recirculation line as practicable. If the pressure during an initial or subsequent doses exceeds a nominal value in the range of approximately five psi, dosing will be discontinued until the pressure dissipates. Gravity discharges will be performed in a manner that allows the operator to visually monitor components (e.g., the tanker, discharge line, and connections to the recirculation line) during dosing. The discharge rate will be metered into the line at a rate that limits buildup of head within the line. Dose volumes will be adjusted from the initial volume based on operating experience (i.e., it is likely lines will ultimately plug increasing the time between dosing).
- Spray and surface pit recirculation will be accomplished from tanker trucks that will cycle back and forth between the operational areas and the leachate storage facilities. The operator will observe the recirculation process for signs that the application rate may be exceeding the hydraulic capacity of the waste (e.g., ponding or overland flow), and dose rates will be decreased as appropriate to avoid such conditions.

Newly constructed landfill cells for the SMI Valley Infill project are not currently planned to receive horizontal recirculation lines. The actual rates of recirculation and specific locations of recirculation are unknown at this time. A geotechnical assessment evaluating the effect of leachate recirculation on the structural integrity and stability of the landfill's liner system, leachate collection and removal systems, and waste mass will be performed at the time these variables are determined.

3.8 COVER SYSTEM DESIGNS

3.8.1 Daily Cover

In accordance with the regulation, six inches of daily cover, or approved AOC, will be applied to the landfill working face at the close of each day. Seneca Meadows currently obtains daily cover material from NYSDEC permitted local borrow pits in close proximity to the landfill. Soils excavated from those locations are hauled to the site using ten-wheel dump trucks. In addition to the soils excavated from local borrow pits, SMI will also utilize approved Alternative Operating Cover (AOC) material as well as a portion of the soil excavated from subgrade. Soil or AOC material used for daily cover should consist of a sandy-type material that can readily drain standing water. The use of as sandy-type soil also maximizes the interconnection between lifts of waste, reducing the potential for formation of perched leachate zones within the cover system. Additional detail regarding the specific materials used and placement procedures is provided in the Facility Manual.

3.8.2 Intermediate Cover

Intermediate cover consists of non-contaminated soils that are not readily eroded by stormwater runoff or wind. Clayey soils are normally used and are applied to achieve a compacted (minimum) one-foot thickness over waste where additional waste will not be applied within 30 calendar days. Intermediate cover is stripped to the maximum extent practical immediately prior to the resumption of landfilling within an area. The stripping of low permeability soils prior to waste placement reduces the opportunity for perching of leachate to occur. Additional detail regarding the specific materials used and placement procedures is provided in the Facility Manual.

3.8.3 Final Cover System and Final Grades

3.8.3.1 General

The final cover design provides stable, long-term protection against exposure of waste, and is intended to limit infiltration and support vegetative growth. The proposed final cover system consists of the following permitted major components, from bottom to top:

- Intermediate cover serving as subgrade;
- Landfill gas venting layer, consisting of a geocomposite drainage layer;
- Linear low-density polyethylene geomembrane liner;
- Drainage geocomposite layer;
- 24 inches of barrier protection layer;
- 6 inch layer of soil suitable to maintain vegetative growth; and,
- Vegetative cover.

On the four percent sloped plateau, final cover will consist of the layers that comprise the 3H:1V slopes, along with a GCL between the landfill gas venting layer and the geomembrane.

The above final cover system is consistent with 6NYCRR Part 360 design requirements. A GCL is proposed for use below the geomembrane in the composite lined areas, and a geocomposite LFG venting layer is proposed as required by 6NYCRR Part 363-6.15.

Permanent cap drainage structures, namely diversion swales, downdrains, and culverts, will be constructed as part of the final cap system and will be consistent with currently permitted designs. Solid and slotted polyethylene drainage piping will be placed within the cover soil (sideslopes only) to prevent buildup of the storm water which percolates into the final cover. These pipes will be placed incrementally across the sideslopes as shown on Sheet 33 of the engineering drawings. They will intercept water flowing through the cover soil and direct it to storm water channels.

3.8.3.2 Final Cover Grading

The final grading plan for the facility designated a peak height of the final cover system at an approximate elevation of 843.5 feet, with outside slopes of 3H:1V. Plateaus are graded on a 5% slope (20H:1V) In practice, it is considered important to maintain a high degree of control over the placement of waste at the outside slope of the landfill. SMI's in-house survey team utilizes GPS surveying equipment that will provide cut-fill grades to working face personnel based on the design waste grades.

The final cover stability was evaluated using an infinite slope analyses. The final cover consists of (from bottom to top):

- Intermediate cover and prepared subgrade
- Landfill gas venting sand layer or geocomposite
- Geomembrane
- Geocomposite
- Barrier protection layer
- Topsoil layer

The slope for the final cover is to be 3H:1V. The minimum residual interface friction angle for any of the above interfaces is 27 degrees. This value will be confirmed by testing on the actual products before cover construction.

The sand layer or geocomposite will have drainage pipes spaced adequately to collect the subsurface flow. The spacing is dependant on the actual permeability of the top soil and the sand layer or transmissivity of the geocomposite, a conservative spacing of 150 feet is used for the calculations. This spacing may be adjusted at the time of construction based on the actual permeability of the material chosen for construction, see Appendix H of this report for calculations.

The static factor of safety for residual interface strengths is 1.5 assuming the geocomposite layer is functioning properly to control seepage. A minimum value of 1.5 is typically acceptable. As the site is not located within a seismic impact zone the seismic load conditions were not evaluated.

As the final cover consist of engineered materials, the shear strength of the drainage layer, and the drainage layer-textured geomembrane interface control the factor of safety against slope failure. The analyses were conducted using a 1-ft. head build up in the sand drainage layer. Note that the 1-ft. conservatively estimates the head build up in the cover materials based on the permeability of the soil and synthetic materials and the anticipated water infiltration.

The infinite slope analysis was computed to confirm the conditions along other sections of the slope. Analyses were conducted for fully drained conditions as well as the sand drainage layer being fully saturated (depth of 1-ft.) as noted above.

3.9 LANDFILL GAS MANAGEMENT SYSTEM DESIGN

Landfill gas management at the Facility reduces methane emmisions, prevents uplift of capped areas, and allows for energy recovery through the implementation of LFG to Energy Systems. Existing landfill gas pipes and collection wells exist at the site and will be utilized during landfill development to properly control landfill gas at the site. The installation of new landfill gas collection systems and modifications to the existing permitted system are outlined in the Landfill Gas Collection and Control System Plan (GCCS) which has been prepared by GHD and is included under separate cover as part of the permit application.

3.10 STORMWATER MANAGEMENT SYSTEM DESIGN

Stormwater management at the Facility reduces leachate generation, minimizes the potential for erosion, and protects adjacent natural waterways. Stormwater management, therefore, is an integral part of landfill operations and contributes to the mitigation of environmental impacts. Permanent stormwater control structures at the landfill were designed using a USDA Soil Conservation Services (SCS), Technical Release No. 55, (TR 55), Type II distribution. The control structure was designed based on a 100-year storm, in accordance with Seneca Meadows, Inc operating requirements. Appendix D of this report provides the Drainage Report and Preliminary Stormwater Pollution Prevention Plan (SWPPP) (collectively referred to as the Drainage Report) for the SMI Valley Infill project permit application.

4.0 LANDFILL CONSTRUCTION AND OPERATIONS

4.1 GENERAL

This section provides an overview of the construction of the landfill facilities. Landfill construction is addressed in the Construction Quality Assurance/Quality Control Plan (CQA/CQC Plan), which addresses the management structure and quality assurance program and procedures in place to document and certify landfill systems construction. A Facility Manual has been developed for the SMI Valley Infill project permit application and is provided in Appendix E of this report. The Facility Manual addresses the landfill operations management structure, waste placement, controls, and environmental monitoring.

Landfill construction includes landfill liner systems, leachate collection/conveyance/storage facilities, landfill gas management systems, final cover and stormwater management features. The day-to-day operations and maintenance activities at the Landfill generally focus on waste placement and environmental controls. Waste placement operations include acceptance and tracking of incoming waste, inspection and compaction of waste at the working face and the placement of daily and intermediate cover materials. For this landfill, the construction of landfill gas collection and leachate management systems (recirculation lines) will be performed concurrent with landfill operations (e.g., waste placement). Other construction items noted above typically occur prior to or following completion of waste placement within an operational area.

4.2 LANDFILL CONSTRUCION

Landfill construction involves the construction of the liner system, waste placement, and construction of cover systems, landfill gas, leachate and stormwater management controls. The design of these components is discussed in Section 3.

Development of the landfill footprint components will consist of earthwork/grading and baseliner construction, followed by waste placement to interim final grade (final grade for each stage). Prior to achieving interim final grade in a stage, construction of the subsequent cell will be completed so that the landfill activities may progress from one stage to the next without interruption.

A typical liner area will be constructed in the following sequence:

- Excavation/fill of the area to design subgrade elevation;
- Construction of porewater drainage layer (as appropriate);
- Placement and compaction of a low permeability soil liner (secondary);
- Installation of secondary geomembrane liner;
- Placement of geocomposite secondary drain system;
- Placement and compaction of secondary leachate drainage layers (as appropriate);
- Installation of primary geocomposite (GCL and geomembrane) liner;
- Placement of geotextile cushion; and,
- Placement of leachate collection blanket drain system.

Landfill construction will entail the excavation of the interior portion of each cell to design subgrade elevations coupled with the construction of perimeter embankments. In addition to providing containment, these perimeter berms provide for the following critical components of the landfill:

- Visual screening and noise abatement during initial filling operations;
- Provides the foundation for a perimeter access road to facilitate operations and maintenance of the facilities; and
- The pitch of the berms has been designed to direct surface water runoff at the foot of the landfill to stormwater management systems.

The overall landfill will be completed with a final cover system designed in accordance with 6NYCRR Part 360 requirements. Final cover will be placed at a maximum slope of three feet horizontal for every one foot of vertical rise. This 3H:1V slope will continue to a point near the peak elevation (the plateau). The plateau will be graded at a minimum five percent slope from the edge of the 3H:1V slope up to the peak elevation.

The plan calls for the continued operation on the Western Expansion concurrent with construction of the proposed landfill. Based on the actual completion date, filling in the new cells is anticipated to commence as filling within the Western Expansion nears completion.

4.3 LANDFILL PHASING PLAN

The proposed development areas will be constructed in phases. These phases are illustrated on Sheets 12-20 of the accompanying Engineering Drawings. These operational drawings show the various facilities that need to be in place at a given point in the operation, each area of new liner construction, each area of active landfill operations, areas where preparatory activities will begin for the next phase of liner construction (e.g., clearing and grubbing), the stormwater and soil erosion controls for each phase. Construction layouts for gas and leachate lines are provided in the GCCS. The phases discussed below are presented in overall anticipated sequence of construction/operations.

4.3.1 Predevelopment

Prior to the start of landfill liner construction and waste placement several activities are necessary to ready the first area (Phase 1). Currently, the area designated as the first cell of landfill development, at the southwest corner of the Tantalo Site is occupied by several operational features including the access road, wheel wash and Citizens Drop-off Area. As part of the initial site preparations, these features will be reallocated as noted on the Engineering Drawings. Rock excavation will be completed at the south end of Phase 1 to accommodate the installation of the leachate collection features of the cell. Placement of fill within the bedrock removal area to provide a 10-foot separation between bedrock and base of constructed liner is anticipated this Phase.

In addition to the infrastructure relocation, a limited portion of the existing Tantalo Waste Disposal Area would be intercepted by the grading required for the Phase 1 subgrade. Where the excavation limits intersect, the existing cap will be removed, the underlying waste excavated and relocated and then the cap would be reconstructed using the original cap design and construction requirements.

The predevelopment work is illustrated on Sheet 12 of the Engineering Drawings.

4.3.2 Cell Construction

4.3.2.1 Phase 1

During Phase 1, the initial landfill liner (Cell 1) will be completed and will receive waste. As part of the earthwork component, a porewater collection system will be installed below the double composite liner system. The porewater collection will convey groundwater to the stormwater management system. Leachate collection and conveyance systems, including a leachate force main, sideriser station, and leachate collection sump will be installed. The leachate force main will tie into the existing Southeast landfill force main.

The subgrade for Phase 2 will be prepared by placing additional fill in the northeast corner of the Tantalo Site.

The Phase 1 work is illustrated on Sheet 13 of the Engineering Drawings.

4.3.2.2 Phase 2

During Phase 2, the landfill liner will be completed and will receive waste. Construction of the dual composite liner system will include approximately 7.9 acres of liner located on the eastern half of this phase. Liner tie-ins will be completed along the western edge of the SELF and Stage 7 and 8 of the NEX.

The subgrade for Phase 3 will be prepared by placing additional fill at the north end of the Tantalo Site. During this Phase the manhole at the north end of the Tantalo leachate collection system will be abandoned. The T-15 force main outside the western limits of the Tantalo cap (west side of the Tantalo landfill) will also be abandoned.

During Phase 2, waste will be placed in the liner area and will be placed over the SMI Valley Infill Phase 1, SELF and NEX Stage 7/8 liner areas.

The Phase 2 work is illustrated on Sheet 14 of the Engineering Drawings.

4.3.2.3 Phase 3

During Phase 3, the landfill liner for will be completed and will receive waste. Construction of the dual composite liner system will include approximately 6.0 acres of liner located on the western half of this cell. Liner tie-ins will be completed along the eastern edge of Stage 3, 5 and 6 of the WEX and the western edge of Phase 2.

The subgrade for Phase 4 will be prepared by placing additional fill on the west side of the Tantalo Site. During this the Phase the existing Tantalo waste that was not relocated at the time of closure and has been covered by an asphalt layer will be excavated and relocated.

During Phase 3, waste will be placed in the liner area and will be placed over the SMI Valley Infill Phase 2, WEX Stage 5 and 6, NEX Stage 7/8, and SMI Landfill liner areas. The Phase 3 work is illustrated on Sheet 15 of the Engineering Drawings.

4.3.2.4 Phase 4

During Phase 4, the landfill liner for will be completed and will receive waste. Construction of the dual composite liner system will include approximately 6.3 acres of liner located on the western half of this cell. Liner tie-ins will be completed along the eastern edge of Stage 3 of the WEX and the western edge of Phases 1 and 2.

The subgrade for Phase 5 will be prepared by placing additional fill on the west side of the Tantalo Site.

During Phase 4, waste will be placed in the liner area and will be placed over the SMI Valley Infill Phases 2 and 3, WEX Stage 3, 5 and 6, NEX Stage 7/8, and SMI Landfill liner areas.

During this Phase the maintenance facility and the fuel island will be moved into the open area around Pond S-4. Preparation for removal of a bedrock knob (drilling and blasting) located within the Phase 6 footprint landfill footprint may begin this Phase. The Phase 4 work is illustrated on Sheet 16 of the Engineering Drawings.

4.3.2.5 Phase 5

During Phase 5, the landfill liner for will be completed and will receive waste. Construction of the dual composite liner system will include approximately 6.5 acres of liner located on the western half of this cell. Liner tie-ins will be completed along the eastern edge of Stages 3 and 4 of the WEX, and the western edge of Phase 1.

The subgrade for Phase 6 will be prepared by excavating soil and rock at the south end of the development. Removal of a bedrock knob (drilling and blasting) located within the Phase 6 footprint landfill footprint will be completed this Phase. Placement of fill within the bedrock removal area to provide a 10-foot separation between bedrock and base of constructed liner is anticipated this Phase.

During Phase 5, waste will be placed in the liner area and will be placed over the SMI Valley Infill Phases 2 and 3, WEX Stage 3, 5 and 6, NEX Stage 7/8, and SMI Landfill liner areas. The Phase 5 work is illustrated on Sheet 17 of the Engineering Drawings.

4.3.2.6 Phase 6

During Phase 6, the landfill liner for will be completed and will receive waste. Construction of the dual composite liner system will include approximately 6.0 acres of liner located on the western half of this cell. Liner tie-ins will be completed along the eastern edge of Stage 4 of the WEX, and the western edge of Phase 1.

During Phase 6, waste will be placed in the liner area and will also be placed over all of SMI Valley Infill Cells and WEX Stages 3 and 4. The Phase 6 work is illustrated on Sheet 18 of the Engineering Drawings.

4.3.3 Final Closure Activities

Following completion of Phase 6, the remaining portions of the landfill will be capped, as depicted in Sheet 19. The final closure of the landfill has an estimated duration of two construction seasons and will involve construction of the remainder of the final cover system.

5.0 LIMITATIONS

The work product included in the attached was undertaken in full conformity with generally accepted professional consulting principles and practices and to the fullest extent as allowed by law we expressly disclaim all warranties, express or implied, including warranties of merchantability or fitness for a particular purpose. The work product was completed in full conformity with the contract with our client and this document is solely for the use and reliance of our client (unless previously agreed upon that a third party could rely on the work product) and any reliance on this work product by an unapproved outside party is at such party's risk.

The work product herein (including opinions, conclusions, suggestions, etc.) was prepared based on the situations and circumstances as found at the time, location, scope and goal of our performance and thus should be relied upon and used by our client recognizing these considerations and limitations. Cornerstone Environmental Group, LLC shall not be liable for the consequences of any change in environmental standards, practices, or regulations following the completion of our work and there is no warrant to the veracity of information provided by third parties, or the partial utilization of this work product.