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## **APPENDIX C**

### **ENGINEERING REPORT**

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**ENGINEERING REPORT  
DUNN MINE AND C&D FACILITY  
MSE BERM PERMIT MODIFICATION**

**Prepared For:**

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**CEE Project 182-442**

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Figure 1 – MSE Berm Modification Area

Figure 2 – Disturbed Area

Figure 3 – Site Vicinity Map

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Appendix C1 – Stability Analysis

Appendix C2 – Project Design Calculations

## **1.0 INTRODUCTION**

The purpose of this Engineering Report is to describe the proposed mechanically stabilized earthen (MSE) berm modification relative to the design, construction, operation, and closure at the Dunn Mine and C&D Facility and to demonstrate compliance with 6 NYCCR Part 360 and 363 Regulations.

### **1.1 LOCATION AND SITE DESCRIPTION**

Dunn Mine and C&D Facility is an existing sand and gravel mine and construction and demolition debris (C&D) disposal facility located in Rensselaer and North Greenbush, New York. The site is owned and operated by S.A. Dunn & Company, LLC. The facility activities include both C&D disposal and mining operations. The site is operating under a permit issued by the New York State Department of Environmental Conservation (NYSDEC) dated August 31, 2020. Figure 1 presents the site location in the region.

The topography in the vicinity of the project site, as shown on Figure 2, ranges from approximately 200 to 300 feet above mean sea level (MSL) and generally decreases to the west towards the Hudson River. Access to the project site is via Partition Street Extension through the City of Rensselaer. There is no other vehicular access to the project site.

The current C&D disposal footprint includes 9 phases of baseliner construction labelled as Phases 1 through 7A, which encompass approximately 37 acres. The total permitted waste footprint is 63.3 acres; with this modification the total permitted waste footprint will be reduced to 62.1 acres and is anticipated to include 7 additional phases of baseliner construction, including Phase 10C which is currently under construction and anticipated to be completed in spring 2022.

Prior to the disposal operations commencement in January 2015, the project site had historically had an existing sand and gravel mining operation, commonly known as the “Dunn Bank.” The mining operations on the project site resulted in the excavation of two (2) large pits known as the south and north pits. Current mining operations are taking place in the north pit and within the proposed and permitted future disposal areas.

Surrounding land uses are primarily residential around the southern and eastern boundaries of the project site. Vacant land is located around the remaining perimeter of the property, and a pre-kindergarten to 12<sup>th</sup> grade school owned by the Rensselaer City School District is located to the north and northeast of the project site, and a cemetery is located to the east.

Municipal water, as well as electric power and telecommunications, currently provide service to the project site along Partition Street.

## **1.2 PROJECT DESCRIPTION**

The facility is currently permitted to operate under a permit issued by the New York State Department of Environmental Conservation (NYSDEC) (Permit DEC #4-3899-00006/00006). The permit encompasses both the mining and disposal activities at the site. The initial permitting for the C&D disposal facility site was prepared by C.T. Male Associates, P.C. and included a Draft Environmental Impact Statement accepted by NYSDEC on January 9, 2012, a Final Environmental Impact Statement dated May 2012, and a solid waste permit application, including engineering drawings, dated October 2010, last revised December 2011. Subsequent to receiving approval for construction of the facility, the construction phasing plan as presented in the initial permit application was revised and a minor modification was prepared by Sanborn, Head and Associates in May 2014. NYSDEC approved the permit revisions, and construction of the site began in late summer 2014. The first phase of construction was approved by NYSDEC in early 2015, and waste placement activities began in January of 2015. Subsequently, CEE prepared a permit modification to revise the disposal facility construction sequencing that was approved in September 2015. In 2016, a permit modification to adjust the waste footprint was prepared by CEE and approved by NYSDEC in 2016. Most recently, a permit modification to revise the baseliner construction and waste filling sequence was prepared by CEE and approved by NYSDEC in May 2020.

This proposed modification includes modifying the northern and portion of eastern perimeter berm to include a mechanically stabilized earthen (MSE) berm. The current footprint limit, along with the approximate limits of the proposed berm modification, is shown on Sheet C100 of the permit drawings, located in Appendix H. The reduced C&D filling area is approximately 1.2 acres. The total land disturbance area has been reduced by 0.66 acres, based on difference between the limit of grading presented on the previously approved Footprint Modification permit drawings and the limit of grading on the MSE Berm Modification permit drawings.

## **1.3 EXISTING FACILITY ANALYSIS MODIFICATIONS**

### **1.3.1 Continued Mining Operations**

The proposed modification will not have any adverse effects on the current mining operations at the facility. The mining operations will continue within areas of future permitted cell construction; no changes are proposed to this permit condition.

To the extent possible, the mined soils will be utilized as part of the MSE berm construction. If sufficient quantity exists, clay from mining will be used for on-site baseliner construction. On-site sand and gravel will be utilized as part of the drainage layer (provided the material meets regulatory requirements) and for operating cover of the in-place C&D material.

### **1.3.2 Capacity, Construction and Operation**

The design capacity relative to the proposed modification will be reduced as part of the proposed modification by approximately 220,000 cubic yards (CY). Based upon an average annual volume of incoming waste at 580,000 CY, the proposed modification will reduce site life by approximately four months. The maximum in-place density of the C&D is expected to be approximately 0.6 tons per cubic yard, based on historical tonnage rates and airspace consumption. Currently, the site is permitted to accept 100 trucks per day; no changes are proposed to this permit condition. Therefore, the total tonnage per day accepted will be within the constraints of the quantity of trucks allowed. The proposed waste disposal area associated with this modification will encompass an approximate final capped area of 62.1 acres. The total area inclusive of detention ponds, perimeter road and perimeter swales, as well as the capped area will remain relatively unchanged at approximately 70 acres.

As shown on permit modification drawings, the disposal facility footprint is anticipated to be constructed in ten phases. Phases 1 through 7A have been constructed and Phase 10A is currently under construction. The overall fill progression plans, as shown on Sheets C308 and C309 of the Engineering Permit Drawings, includes six remaining cells to be constructed within Phases 7, 8, 9 and 10.. The proposed reduction in the limit of waste results in changes to the individual phase and cell limits resulting in the following areas for each area:

- Phase 10B – 1.9 acres
- Phase 10C – 1.4 acres
- Phase 9 – 3.7 acres
- Phase 8A – 6.8 acres
- Phase 8B – 4.6 acres
- Phase 7B – 5.0 acres

Individual cells in a given phase of the landfill development may be separated by an operational berm to provide hydraulic separation between cells to minimize leachate generation through control of stormwater. Intermediate "phase division berms" will be used to the degree practical to segregate rainfall from solid waste inside a given cell.

Further, while the fill progression depicted in the Drawings illustrates specific cell dimensions; in practice, cells will be sized based on incoming waste rate, access requirements, and practical construction considerations. Filling may begin in a partially completed cell as liner construction continues on remaining portions of the cell. This practice generally allows for the placement of the initial lift of Select Waste, which requires additional time to fill. Fill progression will typically begin at the low point of a particular operating cell and proceed in an upslope direction. Areas of each cell will be covered with operating cover, consisting of a minimum of twelve inches of soil, as they are brought to final or interim grade. When a sufficiently large area has achieved final grade, the final cover system will be constructed. Upon reaching final waste fill elevations, an active gas collection system will be installed for each area.

Filling operations will be implemented in accordance with a NYSDEC approved Facility Manual, consistent with the requirement of 6 CRR-NY Part 363-7.1 and Part 360.19. The Facility Manual is included in Appendix D of the Part 360 Permit Application.

### 1.3.3 Leachate Management

Collected leachate is and will continue to be pumped from five leachate sump locations, all of which are currently permitted and operational. The leachate pump system within the cells includes submersible pumps installed in riser pipes placed along the side slopes of the cells that convey leachate to the existing 275,000-gallon storage tank via a dual-contained force main. The stored leachate is then pumped into tankers at the existing load out station and trucked to an approved wastewater treatment facility for treatment and disposal. The load out station has been constructed

with containment for potential spills while loading, and a roofed enclosure to prevent stormwater from entering the station floor and containment structure. The proposed modification will not adversely affect nor significantly change the existing management of leachate. The facility will continue to utilize the same haulers as currently in place to remove leachate from the site.

#### 1.3.4 Phase Closure

The proposed modification will not adversely affect the phased closure. Closure construction will be phased and as the disposal area phases reach approved capacity, closure cap construction will be implemented, meeting 6 CRR-NY Part 363 requirements.

#### 1.3.5 Site Access and Vehicle Circulation

Access to the project site is via Partition Street Extension using the existing entrance road. The existing gate is secured during periods of non-use to prevent unauthorized access into the facility. Vehicle circulation internal to the project site is and will be via gravel access roads (i.e., service roads) constructed for each future phase of disposal area construction. The remaining existing service roads associated with the current mining operation will be incorporated to the maximum extent possible.

#### 1.3.6 Stormwater Management

Stormwater will be managed on site to maintain peak discharge of runoff off-site to less than the permitted conditions. Prior to the closing and capping of the facility, there will be no increase in stormwater leaving the site, as a portion of the runoff will continue to be directed into the mined areas, where it will infiltrate. Stormwater runoff that is not directed to the mining areas will be collected in a series of stormwater channels and conveyed to on-site stormwater ponds. Temporary and permanent detention ponds will be constructed, as necessary, to maintain zero increase in runoff discharge from the site.

There will be no increase in peak discharge of stormwater runoff as a result of the proposed modification above the previously permitted discharge levels. Stormwater will be directed towards on-site stormwater ponds and discharge will be controlled to avoid the potential for adversely altering downstream conditions. The proposed stormwater system is described in Section 4 and supporting calculations are provided in Appendix C2.

### 1.3.7 Electric Power, Water Supply and Sanitary Sewage Requirements

The proposed modification will not affect any existing electric power, water supply, or sanitary sewage requirements. The project site is currently serviced with electric power and municipal water. Three-phase electric power is currently supplied by National Grid to the project site via overhead transmission lines located along Partition Street Extension. City of Rensselaer water is available both along Partition Street Extension and along the existing entrance road where there is a 2-inch service connection. No municipal sewer service currently connects to the project site.

The existing office and maintenance garage building on the project site will continue to be used to service equipment and provide office space for all site operations. This existing building has three (3) toilets, several sinks, electric power, municipal water, and an on-site septic system.

The existing scale house is connected with electric power and municipal water. It has one rest room that is equipped with municipal water. A holding tank has been installed to collect wastewater. It is periodically pumped out with septic waste disposed of off-site.

Electric power is also connected to the leachate storage tank, leachate load-out facility, the flare, and to each of the existing leachate collection stations.

### 1.3.8 Service Area

The service area will not change as a result of this proposed modification and will remain operating as a private C&D facility that will accept only C&D waste material. Ongoing operations have accepted waste from the greater Capital District area including Albany, Saratoga, Schenectady and Rensselaer Counties, as well as areas outside of the Capital District.

### 1.3.9 Groundwater Management

The proposed modification will not affect the permitted conditions regarding groundwater management as this application includes a reduction in overall waste disposal footprint and no change to the maximum excavation grade.

#### 1.3.10 Erosion and Sedimentation Control Plan

The proposed modification will not affect the permitted conditions regarding erosion and sedimentation controls. Standard methods of erosion and sedimentation control will be practiced during each construction sequence. Long-term controls will be provided by establishing vegetative cover in as many areas as practical within the project area, including the proposed footprint area.

#### 1.3.11 Landscape Plan

The proposed modification will not significantly affect the permitted conditions regarding the landscape plan. After each phase of construction, disturbed areas will receive vegetative soil material and seed, consistent with the current existing conditions. At the end of C&D placement, the filled phase areas will have a final cover system installed in compliance with the closure plan.

#### 1.3.12 Materials Protection

The proposed modification will not affect the permitted conditions regarding materials protection. Facility operations personnel will continue to perform daily inspections of installed construction materials to verify that the materials meet specifications and comply with the requirements of 6 CRR-NY Part 363-6.5 through Part 363-6.18, from completion of construction to operation of the new phases.



## **2.0 BASELINER SYSTEM**

### **2.1 GENERAL**

In general, the proposed phases will require mass excavation prior to the construction of the baseliner system. The baseliner system for each proposed phase will consist of a single composite liner system in accordance with the requirements of 6 CCR-NY Part 363-6 Regulations.

The proposed subgrade locations and elevations were unaltered from the currently permitted design with the exception of the northern limit of the footprint that includes Phases 7 and 8. In this area, the floor of the landfill has been reduced to account for the subgrade changes and structural fill required to construct the revised MSE berm and maintain the minimum slope in accordance with the regulations. As required by Part 363, the baseliner elevations will:

- Maintain 10 foot buffer between bedrock and the proposed single liner system subgrade;
- Maintain 5 foot buffer between maximum groundwater elevation and the proposed single liner system subgrade;
- Provide minimum 2% slope within the lined area;
- Provide maximum 3 horizontal to 1 vertical (3H:1V) sideslopes;
- Provide a minimum slope of 1% on all piping.

### **2.2 BASELINER SYSTEM COMPONENTS**

The proposed baseliner system complies with Part 363-6, as described below:

1. A subgrade layer of compacted soil will provide support to the overlying baseliner, and attain design grades to assure adequate leachate drainage.
2. A compacted low permeability soil liner, minimum thickness of twenty-four (24) inches, with an in-place saturated hydraulic conductivity of  $1 \times 10^{-7}$  centimeters per second (cm/sec) or less.
3. A textured high-density polyethylene (HDPE) geomembrane liner, 60 mils thick.
4. A 16-ounce non-woven geotextile.
5. A leachate removal system, consisting of piping and leachate collection stone installed in a 24-inch drainage material layer.

Removal of leachate will be accomplished through the drainage layer and a network of perforated collection pipes enveloped in stone. The baseliner system has been designed so that leachate in the collection system will flow to and through the collection pipes to sumps located at the low points of the baseliner system along the west side of the facility, and then will be pumped from the sumps to the on-site leachate storage tank.

## **2.3 SUBGRADE LAYER**

The top twelve (12) inches of the subgrade in the future baseliner phases will be compacted to 92% Modified Proctor density, as specified in the Technical Specifications. Embankment construction and intermediate grading that is necessary to reach subgrade elevations will be compacted to 92% Modified Proctor Density. A minimum of nine moisture/density field tests will be performed per acre in order to provide certification of the required compaction. The Construction Quality Assurance/Construction Quality Control (CQA/CQC) includes subgrade construction procedures.

The disposal area will be excavated and filled as necessary to achieve the design grades shown on the permit modification drawings, maintaining a 2.8% minimum slope. There are no changes to the approved subgrade elevations and slope on the floor of the landfill, with the exception of the northern portion of Phases 7 and 8, where a portion of the 2.8% sloped floor has been reduced to allow for backfill of the modified berm which will be constructed at a 3H:1V slope. The maximum loading on the subgrade will not be increased as a result of this analysis; therefore, the post-settlement slope provided in the original design will not be impacted. A NYS licensed land surveyor will be required to provide certification of subgrade as-built conditions, to a tolerance of 1 +/- inch.

## **2.4 LOW PERMEABILITY SOIL LINER**

The low permeability soil liner component for the new phases will be installed over the prepared subgrade. The minimum compaction required will be 85% of the Modified Proctor Density, as well as sufficiently compacted to provide a maximum hydraulic conductivity of  $1 \times 10^{-7}$  cm/sec. The clay will be placed in lifts having a compacted thickness of 8 inches. QA/QC testing will be performed at the frequencies specified in the Technical Specifications. The QA/QC testing frequencies will meet the requirements of Part 363.

Excavated soils from the mining operations have historically been collected and tested to determine whether the material could be used as a low permeability material source. Test results have indicated that the clay is suitable for use as both the low permeability liner component and the final cover barrier layer when capping construction commences. Confirmatory testing will be conducted on material prior to construction of each phase in accordance with the Technical Specifications and CQA Manual. As required, low permeability soil consistent with regulatory requirements and the Technical Specifications, will be imported from an off-site source.

## **2.5 HDPE GEOMEMBRANE**

The HDPE liner component will be a minimum of 60-mil thickness, and have a textured finish on both sides. The HDPE geomembrane will be installed using approved methods, consistent with applicable regulations and the approved Technical Specifications. A CQA/CQC program shall be conducted as required by Part 363.

## **2.6 DRAINAGE MATERIAL LAYER**

The soil drainage layer component will be installed above the geomembrane. The drainage material layer will comply with the requirements of 6 CRR-NY 363-6.6(3)(ii). On slopes less than or equal to 10%, the material will have a minimum hydraulic conductivity of 1 cm/sec. On slopes greater than 10%, the material will have a minimum hydraulic conductivity of 0.1 cm/sec. The soil will be placed in one 24-inch lift using approved methods, consistent with applicable regulations and the Technical Specifications. A cushion layer of non-woven geotextile will be installed above the geomembrane to protect it from potential damage from the drainage layer materials.

Suitable material will be imported to the site for use in the baseliner system. Confirmatory testing will be conducted on the material prior to construction of each phase in accordance with the approved Technical Specifications and CQA/CQC Plan and as required by Part 363.

## 2.7 LEACHATE COLLECTION SYSTEM

### 2.7.1 General

In order to design the proposed system and confirm its feasibility, potential leachate generation quantities were estimated. Calculations are located in Appendix C2 and results are as follows:

- Leachate Generation:
  - The existing pumps and pipes have the capacity to handle the expected daily leachate generation rate.
- Maximum head on liner:
  - The maximum head on the liner will be less than 12 inches; the calculated daily leachate generation rate is similar to the previous design and the drainage stone permeability has increased, while the pipe spacing has remained the same. Therefore, the pipe layout will continue to maintain a head less than 12 inches on the liner and head calculations are not included.
- Pipe strength calculations:
  - Proposed pipes will consist of SDR-11 HDPE pipes and will withstand the dynamic and static loading stresses associated with construction, operation, closure, and post-closure. There is no increase in the maximum elevation of the landfill proposed with this permit application; therefore, the pipes as previously designed are adequate and pipe strength calculations are not included.

### 2.7.2 Leachate Generation

Leachate generation quantities were estimated using the Hydrologic Evaluation of Landfill Performance Model (HELP 3.07) developed by the U.S. Army Waterways Experiment Station for the USEPA. This program is used to model baseliner cross sections leachate generation rates, leachate collection and removal systems, and cover configurations by performing a water balance analysis of a landfill in any stage of completion.

The HELP program is used to model proposed groundwater protection systems, leachate collection and removal systems, and cover configurations by performing a water balance analysis of a landfill in any stage of completion. The HELP program models landfill performance using climatological data, soil characteristics, design details, and operational features, which together simulate

conditions expected at the site. Climatological data requirements included precipitation, temperature, and solar radiation. The soil characteristics include porosity, field capacity, wilting point, initial water content and hydraulic conductivity for each layer of the landfill profile. The required design data includes soil and waste layer thickness, liner and cover configurations, surface area, leachate collection or drainage layer slope, drainage length (flow path between collection pipes), and the condition of the landfill surface.

To perform the analyses for this proposed modification, climatological data for Albany, New York was used. The characteristics for the various layers of the facility were generally selected from a list of characteristics provided in the program that mostly reflect the types of materials that will be used at the Dunn facility. In various layers, in order to comply with project specifications, the hydraulic conductivity parameter within the model was modified.

The HELP model was utilized to predict leachate generation for the following waste depths: 10 feet, 50 feet, 100 feet, and 150 feet. A summary of the results is shown in the table below. As presented in the table below varying waste depths the average leachate generation was estimated between approximately 238 and 1,257 gallons per acre per day. The estimated generation rates shown below are similar to the rates which were presented in the previously approved permit modification. The HELP calculations are provided in Appendix C2.

#### **HELP MODEL RESULTS – PER ACRE ANALYSIS**

<b>Waste Depth (ft)</b>	<b>Predicted Generation Rate (gal/acre/day)</b>
10	1,257
50	755
100	381
150	238
Cap*	20

\*20 gal/acre/day is assumed for final cap leachate generation based on historical final cover rates.

Using this information and the proposed phasing plan for the facility landfill, the maximum leachate generation rate was determined. The maximum leachate generation rate will occur when in the last phase of development when Phase 7B has just been constructed and initial filling has begun. In this scenario, there will be one lift or 10 feet of waste in place in Phase 7B, while the other phases will have final cover, intermediate cover, or daily cover.

For this case, approximately 5.0 acres will be filled with an average waste depth of 10 feet, 15.4 acres will be filled with an average waste depth of 50 feet, 8.1 acres will be filled with an average waste depth of 100 feet, 18.5 acres will be filled with an average waste depth of 150 feet, and 15.2 acres will be capped. A drawing depicting this stage of filling is included in with the leachate generation supporting technical data in Appendix C2. The following table presents the total leachate generation for this scenario. Based on this analysis, the anticipated average daily pumping rate is 18 gallons per minute (gpm). The current pumps in each phase have the capacity to handle this rate.

#### **MAXIMUM LEACHATE GENERATION SCENARIO SUMMARY**

<b>Average Waste Depth (ft)</b>	<b>Area (acre)</b>	<b>Daily Collection Rate (gal/day)</b>
10	5.0	6,285
50	15.4	11,627
100	8.1	3,086
150	18.5	4,403
Final Cover	15.2	304
<b>TOTAL</b>	<b>62.2</b>	<b>25,705</b>

#### **2.7.3 Leachate Collection System**

The leachate collection system will be installed above the HDPE geomembrane. For future development areas, the system will be comprised of 24 inches of granular drainage material with a minimum permeability of 1 cm/sec on slopes less than or equal to 10 percent and a minimum permeability of 0.1 cm/sec on slopes greater than 10 percent. The soil drainage layer will be installed in one 24-inch lift. The collection system will also include a network of perforated collection pipes installed within the drainage protection layer. The pipe slope, spacing, and alignment have not been changed with this modification. Together, these components will convey leachate to the existing sump areas and minimize the leachate head on the liner.

The piping network is proposed to be installed within the base area, generally conveying the leachate in an east-to-west direction, at a minimum two percent (2%) slope, towards the existing sump locations. The leachate collection piping will be 8-inch diameter perforated SDR-11 HDPE pipes. The existing sumps are equipped with 18-inch diameter riser pipes installed on the side slope to facilitate the leachate pump, as shown on the permit modification drawings. These pipes were selected based on pipe strength and hydraulic calculations contained in Appendix C2.

The leachate collection piping will be installed within a minimum three-foot wide stone trench. The proposed stone for this application is crushed, screened, non-carbonate, NYSDOT #2 stone. Pipes will be placed at the locations and slopes as shown on the permit drawings. The capacity of each 8-inch diameter pipe, calculated based on a 1% post-settlement slope, is approximately 640 gallons per minute when flowing full, which greatly exceeds the anticipated leachate generation rate.

In order to maintain the integrity of the leachate collection pipes, proposed leachate cleanouts will be installed at the ends of perforated pipe sections. These cleanouts will be 8-inch diameter solid walled SDR-11 HDPE pipe, installed on the proposed baseliner sideslope and terminating outside of the limits of the cell. All cleanouts will be fitted with a cap to allow for easy access and maintenance. The leachate collection system will be cleaned on an annual basis to prevent clogging and to access the overall operation and performance of the system. Access for cleaning will be provided by strategically located cleanouts built into the system, as shown on the permit modification drawings.

The proposed configurations satisfy the requirements of the NYSDEC regulatory design and performance standards and are shown on Sheet C301 of the permit drawings.

#### *2.7.3.1 Evaluation of Impacts of 500-year, 24-hour Design Storm*

The 6 CRR-NY Part 363-4.3(e)(ii) requires that the impacts of a 500-year storm on the landfill's leachate collection and removal system that do not have intermediate or final cover placed. Based on the Cornell Extreme Precipitation for New York and New England data, Rensselaer, NY will receive 9.91 inches of rain during a 24-hour, 500-year storm event.

During the initial stages of construction when the cell is open, the entire storm volume would be stored within the cell based on the cell geometry. The average cell area is approximately 8 acres and the operational berms will have a minimum height of 4 feet; therefore, providing about 1.4 million cubic feet of storage capacity with no freeboard and not including the storage capacity of the 24-inch thick sand layer. A storm with a total precipitation rate of 9.91 inches will generate approximately 288,000 cubic feet of rainwater. There will be contributory areas from adjacent landfill sideslopes draining into an open cell that will increase the storage requirements, though the exact areas will vary throughout the life of the landfill. During initial stages, there will be C&D waste in the cell as well that will somewhat reduce storage capacity; however, the C&D waste will have capacity to hold water as well. The average open cell is expected to have sufficient capacity to fully contain rainwater from the 500-year storm. The time to fully dewater the cell will be 15 days based on average pumping rates of the pump currently installed in Phase 7.

When the cells are partially filled with waste grades above the operational and perimeter berm elevation, rainwater that falls on the external slopes will flow to perimeter channels alongside the road. Rainwater that falls on the landfill plateau area will be absorbed by the waste mass or may become overland flow that will also be directed to the stormwater management system. The storm will cause a temporary increase in leachate generation and collection rates; however, the timing and actual impacts to the collection system will vary depending on the total depth of waste in place at the time of the storm as well as the size of the plateau area. The piping system at the facility includes 8-inch diameter pipes spaced approximately 188 feet apart. Based on the site geometry and pipe layout, each pipe will potentially have a contributing drainage area of approximately 6 acres. At the leachate generation rate expected for the landfill of 1,367 gallons per acre per day, this equates to a total flow rate of 5.7 gallons per minute. The maximum pipe flow capacity is 640 gallons per minute; therefore, the pipes have capacity over 100 times the expected leachate generation rate. The 500-year storm will increase the rate, though likely it will be less than that factor; therefore, the collection piping system is expected to have adequate capacity to convey the peak generation. There will also be an increase in the leachate head on the liner, though this again will vary based on the waste depth and timing of the storm and will be a temporary situation.

Depending on the site configuration at the time of the storm, there is also the potential for leachate seeps to occur. The facility would continue to monitor the landfill side slopes as in current operations and would address seeps by placing additional soil above the seep area for containment.



#### 2.7.4 Leachate Pumping System

In accordance with 6 NYCRR 363-6.11(1), the leachate collection system must be designed to allow the peak flow from the 25-year, 24-hour design storm to be removed from the each disposal cell within seven days. All sumps have been constructed and the pumping systems were designed to meet this requirement.

There is no proposed modification to the leachate pumping system from the previously approved permit modification.

#### 2.7.5 Leachate Riser Buildings

There are five existing leachate riser buildings at the facility. Once leachate has been collected from the piping network and directed into the leachate sumps, it is removed via submersible pumps that convey the liquid in a 2-inch force main contained in an 18-inch sideriser pipe, which terminates in a leachate collection building. There are five existing leachate riser buildings at the facility. The pump controls, flow meters, monitoring devices, and flow control valves are located within the building. From the leachate riser building, leachate is conveyed through a dual-contained 3-inch in 6-inch diameter force main that connects to the main dual-contained 4-inch in 8-inch diameter force main located, on the outside of the western perimeter access road. The entire conveyance force main from Phases 1 through 7 has been built. No additional force main construction is anticipated.

### 2.8 LEACHATE STORAGE SYSTEM

#### 2.8.1 General

In accordance with 6 CRR-NY 363-6.20(a) and the transition requirements within 360.4(o)(2), no modifications are required to the existing on-site leachate storage tank. The existing tank, which was approved in a previous permit modification, is capable of storing at least 275,000 gallons of leachate, while maintaining a minimum of 2 feet of freeboard, has a secondary containment system in the event of a leachate spill, and provides adequate on-site storage for the expected leachate generation rate noted previously. The leachate within the tank is monitored as described below and leachate is removed as required.

### 2.8.2 Monitoring

The volume of leachate in the storage tank is monitored on a daily basis by recording the leachate as indicated by the level transmitter and converting this number to gallons using a conversion chart. The quantity of rainfall is also recorded on a daily basis from an on-site rain gauge. The leachate collection system is to be inspected on a regular basis as indicated by the inspection plan.

### **3.0 LANDFILL GAS MANAGEMENT SYSTEM**

#### **3.1 EXISTING GAS COLLECTION SYSTEM**

The existing facility gas management system consists of a blower to provide vacuum, gas extraction wellheads, vertical gas extraction wells, a gas conveyance header, and a gas treatment system. Gas extraction wellheads are installed at eight leachate cleanout locations, while a ninth leachate cleanout is connected to the vacuum system with a valve that is used for condensate management. Additionally, there are currently seven vertical gas extraction wells located in the existing waste mass.

Header and lateral pipes have been installed to convey landfill gas from the extraction points to the flare and blower system. The existing pipe network consists of a main header pipe sloped at a minimum five percent towards a condensate collection point. The header pipe on the west side of the landfill is an 8-inch diameter HDPE pipe, while the header on the east side is a 6-inch diameter HDPE pipe. The 6-inch pipe is connected to the existing 4-inch perimeter pipe as shown on the drawings to connect the leachate cleanout extraction points in this area to the flare system. A series of lateral pipes connects the leachate collection points and the vertical gas well extraction points to the header pipe.

Condensate management is accomplished by sloping the pipes to low points for collection of condensate. The conveyance system piping has been designed with a minimum 5 percent slope towards one low point on the west side of the landfill as shown on the Drawings. At this location, a condensate dripleg and trap is constructed to discharge to the nearby leachate cleanout pipe. The knockout pot located on the utility flare/blower skid system discharges to an adjacent leachate cleanout.

#### **3.2 PROPOSED GAS COLLECTION SYSTEM**

The proposed landfill gas management system will consist of 13 additional vertical gas extraction wells located as shown on the Engineering Drawings connected via a series of lateral pipes to the main header, which will be extended along the east and west sides of the landfill. The existing condensation collection point will continue to be used as the system is expanded and no additional condensate management structures are needed.

Vertical wells will consist of 8-inch diameter SDR17 HDPE perforated pipe (or 8-inch SCH80 PVC perforated/slotted pipe) installed in a 36-inch borehole backfilled with round or crushed stone; the upper 10-feet of the well consists of solid walled HDPE (or PVC) pipe with a bentonite plug to limit air intrusion. The vertical wells are detailed on the Drawings. A landfill gas wellhead will be installed to control the vacuum to the well.

Shallow gas extraction wells will be constructed in the future as needed to control odors. These wells will be constructed using an excavator to dig the well to an approximate depth of 20 feet. Similar to drilled wells, these extraction points will consist of perforated pipe within crushed stone; however, the upper 5 feet will be solid walled pipe. Backfill at these locations will be low permeability soil with a minimum thickness of 2 feet.

Similarly, horizontal collection pipes will be installed in the future as needed to control odors. These extraction points will consist of perforated pipe installed in a horizontal trench backfilled with round or crushed stone (or similar permeable media) or waste backfill and connected to the header pipe. Horizontal collection pipes will be sloped towards a low point where a 5-foot deep sump will be constructed to discharge condensate, which will then drain back into the landfill.

## **4.0 STORMWATER MANAGEMENT SYSTEM**

### **4.1 GENERAL**

The stormwater management system for the proposed MSE Berm modification will consist of a number of components designed to minimize erosion, prevent run-on of stormwater onto active disposal areas, control stormwater run-off, and segregate contact and non-contact run-off. These components include flow diversions, channels, swales, stormwater management structures, and stormwater ponds. Each component of the system has been analyzed to assure it can manage the regulatory design requirements.

The following specific control measures will be employed to attenuate run-off discharge peaks, reduce sediment transport, and enhance the infiltration of managed run-off in areas outside the footprint. Stormwater runoff generated from the proposed MSE Berm Modification will flow into the proposed North Pond, similar to the previously approved permit modification.

- Grading will be used to shape surfaces and on and off the disposal areas disturbed by construction. Grading will be performed in a manner, which does not produce sharp changes in grade and to minimize the uncontrolled channeling of run-off.
- Sideslope diversion berms, lined with an erosion mat, will be installed on the final cover to divert slope runoff to downchutes. These berms are designed to reduce the distance that overland runoff has to travel, thereby reducing erosion of the completed final cover areas.
- Gabion lined downchutes will be constructed as part of the final cover system to direct runoff from the sideslope diversion berms to the perimeter swales at the toe of the disposal area.
- Perimeter swales will be constructed along the inside of the access road to convey runoff to the North Pond.
- The existing South Pond and proposed North Pond have been designed to manage the 100-year, 24-hour storm event.
- Vegetation will be developed on the final cover system. A mixture of native grasses shall be used for this purpose.

## **4.2 EXISTING PERMITTED CONDITIONS**

The total area of land disturbance for the existing permitted conditions was approximately 70.5 acres. The closing and capping of the facility would have involved the installation of a low permeability cover system/cap designed to minimize the migration of water into the waste mass. Perimeter swales, two drainage ponds (North Pond and South Pond), and discharge pipes were permitted to control stormwater runoff. The drainage areas of the northern and southern ponds were about 45.1 and 27.7 acres, respectively. The drainage area of the south pond remains unchanged with the proposed modifications. There are no proposed changes to the south pond; therefore, the proposed modifications will not increase the discharge from this area. The drainage area to the North Pond has also decreased; however, the design grading has changed. A detailed analysis has been prepared to demonstrate that the peak discharge from the north pond will not increase post-development.

## **4.3 POST-DEVELOPMENT CONDITIONS**

The total area of land disturbance associated with the proposed modification for the post-development conditions is approximately 45.1 acres, which eventually flows into North Pond. The closing and capping of the facility will involve a similar installation of a low permeability cover system designed to minimize the migration of water into the waste mass. Various stormwater controls, discussed above, have been designed to manage stormwater runoff.

## **4.4 PERMIT MODIFICATION – PRE-DEVELOPMENT VS. POST-DEVELOPMENT**

The table below shows a comparison of permitted and post-development condition peak outflow rates for the proposed permit modification design. The existing South Pond will not be affected by the proposed modification and was not included in the analysis. The South Pond will be constructed as previously permitted. It should be noted that the outflow rates in the table below are based on infiltration capability through the basin, and are not discharges to the surface, i.e. in both permitted and proposed conditions, there is no proposed overland flow from the north pond. HydroCAD was used to compute flow rates for the 2-, 10-, 25-, and 100-year storm events.

PEAK OUTFLOW RATES COMPARISON						
Location	Condition	Node Number	2-Year (cfs)	10-Year (cfs)	25-Year (cfs)	100-Year (cfs)
North Pond	Permitted	10P	2.4	3.48	3.95	4.65
	Post-Development	26P	1.38	2.44	2.93	3.79

Supporting documentation for the stormwater management design calculations is provided in Appendix C2.

#### 4.5 DESIGN CRITERIA FOR STORMWATER MANAGEMENT CONTROLS

The design criteria for stormwater management controls was developed to comply with all applicable regulatory design standards, and are as follows:

- Maintain existing drainage patterns as much as possible and continue the conveyance of upland watershed runoff.
- Control increase in stormwater runoff resulting from the closure and capping of the disposal facility without adversely altering downstream conditions.
- Mitigate potential stormwater quality impacts and prevent soil erosion and sedimentation resulting from stormwater runoff.
- Provide an outlet for the surface water runoff, minimize ponding and divert runoff to stormwater management ponds.
- Minimize the surface water runoff and other stormwater run-on from infiltrating the area around the perimeter of the footprint.

#### 4.6 ANALYSIS OF STORMWATER MANAGEMENT CONTROLS

##### 4.6.1 Disposal Area Structures

Stormwater runoff from the disposal area will be conveyed to the North Pond via diversion berms, downchutes, perimeter swales, and drop structures. Each of these structures have been designed for a rainfall of 4.9 inches over a 24-hour duration (the equivalent of a 25-year storm event) and have the ability to handle the peak runoff for the particular structure from the design storm. Therefore, each structure (diversion berms, downchutes, channels, drop structures and swales) is

capable of handling the flow that would be generated by the largest drainage area contributing runoff to any such structure on the disposal area.

The maximum flow rate and the design capacity of each conveyance structure is summarized in the table below. Supporting design calculations are provided in Appendix B2.

<b>Stormwater Control Structure</b>	<b>Maximum Flow 25-Year Storm (cfs)</b>	<b>Maximum Capacity (cfs)</b>
Diversion Berms	16	45
Gabion Lined Downchutes	44	106
Perimeter Swales	60	183
Energy Dissipator 1	27	400
Energy Dissipator 2	57	507
Basin Inlet Channels	85	130
36" Dia. HDPE Pipes	82	136

#### 4.6.2 Diversion Berms

In general, sheet flow from the disposal area will be collected via the proposed sideslope diversion berms. The proposed locations of these sideslope diversion berms is shown on Sheet C303 and C304 of the permit modification drawings. The diversion berms are designed with a minimum slope of 4%. Diversion berms will be lined with an erosion control mat, which will minimize erosion caused by the maximum velocity developed in the channel.

The diversion berms are designed in accordance with the “Standards and Specifications for Lined Waterway or Outlet” in the *New York Standards and Specifications for Erosion and Sediment Control*. The HydroCAD report, and other relative calculations, for the proposed diversion berms can be found in Appendix C2.

#### 4.6.3 Gabion Lined Downchutes

In general, flow from the proposed diversion berms will be conveyed into gabion lined downchutes. The purpose of these downchutes is to collect runoff from the diversion berms and convey the flow towards the perimeter swales, located along the inside of the perimeter access road, or to basin inlet channels, located along the sideslopes of the proposed stormwater



management pond. The locations of the proposed gabion lined downchutes is shown on Sheet C303 and C304 of the permit modification drawings. The downchutes will be constructed on a 3H:1V slope and shall have gabion baskets linings installed for purposes of erosion control.

The gabion lined downchutes are designed in accordance with the “Standards and Specifications for Lined Waterway or Outlet” in the *New York Standards and Specifications for Erosion and Sediment Control*. The HydroCAD report, and other relative calculations, for the gabion-lined downchutes can be found in Appendix C2.

#### 4.6.4 Perimeter Swales

Drainage swales surround the perimeter of the disposal area with the purpose of collecting all runoff from the final cover area to convey the stormwater into the proposed North Pond and existing South Pond. Locations of the perimeter drainage swales can be found on Sheet C303 and C304 of the permit modification drawings. The proposed drainage swales will be lined with North American Green P550 Reinforcement Turf. This erosion mat was selected to handle the maximum velocity in a single swale section.

The swales are designed in accordance with the “Standards and Specifications for Lined Waterway or Outlet” in the *New York Standards and Specifications for Erosion and Sediment Control*. The HydroCAD report, and other relative calculations, for the perimeter drainage swales can be found in Appendix B2.

#### 4.6.5 Energy Dissipators

Prior to reaching the perimeter diversion swales, stormwater runoff within the proposed downchutes will flow into concrete drop structures or rip-rap lined energy dissipators. The purpose of the concrete drop structure is to collect and combine incoming flow from the perimeter drainage swales and gabion lined downchutes in order to control the flow and result in non-erosive flow conditions. The purpose of the rip-rap energy dissipator is to reduce the velocity of flow within the downchute before it enters the perimeter drainage swale or exiting the drop structures and entering the basin inlet channel. Locations of these structures can be found on Sheet C303 and C304 of the permit modification drawings.

The downchute energy dissipators are designed in accordance with the *Design of Roadside Channels with Flexible Linings* published by the U.S. Department of Transportation Federal Highway Administration. The HydroCAD report, and other relative calculations, for these structures can be found in Appendix C2.

#### 4.6.6 Basin Inlet Channels

Flow from perimeter swales will be conveyed into the North Pond via inlet channels. The purpose of the basin inlet channel is to convey flow into North Pond and resist the erosive flow velocities. Locations of the basin inlet channels can be found on Drawing Sheet C303 and C304 of the permit modification drawings.

The channels are designed in accordance with the “Standards and Specifications for Lined Waterway or Outlet” in the *New York Standards and Specifications for Erosion and Sediment Control*. The HydroCAD report, and other relative calculations, for the basin inlet channels can be found in Appendix B2.

#### 4.6.7 Outlet Protection

Rock outlet protection has been designed for the flows discharging from the basin inlet channels. The purpose of rock outlet protection is to prevent soil erosion. Locations of outlet protection areas are shown on Sheet C303 and C304 of the permit modification drawings.

The outlet protection structures are designed in accordance with the “Standards and Specifications for Rock Outlet Protection” in the *New York Standards and Specifications for Erosion and Sediment Control*. Calculations for the rock outlet protection structure for the basin inlet channels can be found in Appendix B2.

#### 4.6.8 Stormwater Management Pond

In the northwest corner of the facility, a stormwater infiltration pond, designated as the North Pond, will be constructed to control runoff from the disposal area. This pond has been designed and sized such that it will have no surface water discharge for a 100-year storm event. A rip-rap lined overflow weir will be constructed in the berm at an invert elevation of 167 feet MSL (mean sea level) to provide overflow capacity from the pond. The stormwater collected in this pond will

infiltrate the pond floor at a minimum rate of 0.5 inches per hour, based on the characteristics of the on-site sand.

An existing detention pond, located in the southwest corner of the facility, designated as the South Pond, was constructed during the baseliner construction of Phase 2 (summer 2015). The pond outlet structure consists of a 12-inch diameter pipe at an elevation of 157 feet. This discharge pipe leads to a four-foot square precast concrete chamber with a 12-inch diameter outlet pipe capped with a 7-inch diameter orifice cut into it. The chamber extends vertically to a height of 167 feet MSL where an 18-inch x 18-inch grate controls incoming flow. An emergency spillway has been built at elevation 168.9 feet MSL. The outlet structure will be modified as shown on the previously approved Footprint Modification drawings.

The proposed modification will not affect the permitted conditions of the South Pond.

#### 4.6.9 Evaluation of Impacts from 500-year Storm

In accordance with 6 CRR-NY 363-4.3(f), an evaluation of the impacts of a 500-year storm was evaluated. HydroCAD was used to compute flow rates for the 500-year storm event to evaluate the potential impacts on the stormwater management system. Based on the Cornell Extreme Precipitation for New York and New England data, Rensselaer, NY will receive 9.91 inches of rain during a 24-hour, 500-year storm event.

Based upon the HydroCAD analysis, the north pond will have the capacity to store and safely pass the volume from the 500-year, 24-hour storm through the rip-rap lined overflow. This overflow will direct stormwater towards the unnamed creek to the north and west of the site. There may be overtopping of the landfill diversion berms and gabion downchutes during this event. Based on landfill grades, this flow will be directed towards the perimeter channels. In general, during the peak of the storm, overflow from the perimeter channel on the west will flow to the unnamed creek to the north and west of the site. On the east side, there is additional containment from the 40 foot berm and road; any overtopping in that location would be directed to the north and could flow to the MSE Berm channel adjacent to the property line and then into the North Pond. The second section of the perimeter channel on the northeast side will have the capacity to handle flow from the 500-year, 24-hour design storm. Based on the hydrograph for each structure, the peak elevation will be exceeded at hour 12 but will have receded below the top elevation at hour 12.5; therefore, the overtopping of any structures is limited to that time frame. Minor erosion and rock

displacement may occur. The eroded vegetation and displaced rock will be restored to existing conditions.

## **5.0 GEOTECHNICAL ANALYSIS**

### **5.1 GENERAL**

This section provides a description of the slope stability evaluation performed for the proposed modification. The evaluation includes an analysis of the base and final cover liner systems and the construction and demolition (C&D) waste that will be disposed, and addresses stability of the disposal area during construction, operation, and closure.

This evaluation provides the minimum interface shear angle required for a stable configuration and establishes conformance test interface shear strength criteria for the base liner system and final cover system components. These criteria can be used to evaluate the shear strength of various interfaces to verify the materials supplied for cell and cap construction will provide factors of safety equal to or greater than those described herein.

Slope stability calculations are attached and evaluate the following conditions:

- Interim and overall slope stability of the liner system and waste mass under several intermediate and final waste slope conditions under static and seismic conditions.

Various slope stability models were prepared to evaluate these conditions. Each model requires input related to disposal area geometry and physical properties of the various construction materials and C&D waste. The following sections describe the properties developed for use in the stability models.

#### **BASE LINER SYSTEM COMPONENTS AND DISPOSAL AREA GEOMETRY**

The base liner system will consist of the following components from top to bottom:

1. 5 foot thick select fill sand layer;
2. 24 inches of drainage material;
3. 16-ounce non-woven geotextile;
4. 60-mil textured high density polyethylene (HDPE) geomembrane;
5. 24 inches of low permeability soil; and
6. Subgrade.

The proposed base grades for the modification create a bowl-shaped footprint consisting of 3 horizontal to 1 vertical (3H:1V) sideslopes that extend downslope from the perimeter and tie into a floor area which is sloped at approximately 3 percent. Final grades consist of maximum 3H:1V sideslopes that extend upslope from the landfill perimeter and transition to an approximate 5 percent slope to form the landfill top plateau. The addition of the MSE berm will modify portions of both the base and final grades. Specifically, the toe of the sideslope in Phases 7 through 10B will extend further into the floor areas, and the final cover elevations are modified slightly at the perimeter in certain locations to reflect the revised height of the MSE berm.

#### DESIGN PARAMETERS FOR SOIL COMPONENTS

Shear strength and unit weight properties for the drainage layer material and low permeability soil were obtained from lab results for the materials used during previous construction of the C&D Facility.

##### Drainage Layer Material (pea gravel)

Moist unit weight = 132.8 pounds per cubic foot (pcf)

Internal friction angle = 33.8°

Cohesion = 107 psf

##### Low Permeability Soil

Moist unit weight = 118.5 pcf

Internal friction angle = 17.4°

Cohesion = 67 psf

#### DESIGN PARAMETERS FOR C&D WASTE

Several references were reviewed to develop shear strength and unit weight properties for C&D waste. These references include:

- Waste Materials in Construction, Goumans, van der Sloot and Aalbers, November 1991;
- Construction Demolition Waste, Sustainable Waste Management and Recycling, Limbachiya and Roberts, 2004; and
- Construction Materials and Structures, First International Conference on Construction Materials and Structures, Ekelu, Dundu and Gao, November 2014.

These references showed internal friction angles for C&D waste ranging from 35 to 42 degrees, and cohesion ranging from 0 to 3,500 pounds per square foot (psf). Based on a review of the reference material, a composite shear strength envelope was used to model the parameters of the C&D waste, as follows:

C&D Waste at Normal Stress < 2,000 psf

Unit weight = 60 pcf

Internal Friction Angle = 35°

Cohesion = 0

C&D Waste at Normal Stress ≥ 2,000 psf

Unit weight = 60 pcf

Internal Friction Angle = 33°

Cohesion = 0

GEOSYNTHETIC INTERFACE SHEAR STRENGTH PROPERTIES

The minimum acceptable interface shear strength properties for the base liner systems were determined as part of this evaluation. Specifically, the interface shear strength required to provide a factor of safety (FS) of at least 1.50 was determined for interim slope stability during waste filling operations and overall stability following closure per 6 CRR-NY 363-4.3(c)(3)(iii).

INTERIM AND OVERALL SLOPE STABILITY ANALYSIS

An analysis was performed to evaluate stability of the liner system and the waste mass under several different intermediate and final waste slope grading configurations, referred to as interim (i.e., at intermediate grading configurations) and overall (i.e., at final grading configurations) slope stability. The interim analysis evaluates slope stability during waste filling operations, when there could be long interior intermediate waste slopes coupled with relatively small temporary interior toe berms at the base of these slopes. The overall analysis evaluates slope stability of the longest and steepest portions of the 3H:1V final grades and/or base grades, such that the maximum forces that cause instability are captured in these cross-sections. The locations of these cross-sections are provided in Appendix C. The required interface shear strengths above and below the textured HDPE geomembrane which would provide a minimum interim slope stability FS of at least 1.5 were determined using the SLIDE slope stability computer program. The analysis uses a search

routine that generates hypothetical failure surfaces through a modeled cross-section of the disposal area and subsequently calculates the ratio of the mobilized shear strength to the driving shear stress to determine the FS along the failure surface.

Additionally, this slope stability analysis includes a piezometric surface located 1 foot above the base of the geosynthetic interface layer (i.e., within the sand drainage layer) in order to represent the maximum allowable hydraulic design pressure that is allowed to accumulate above the liner system.

Sliding block analyses were used to determine the minimum interface friction angle required of the liner system to provide a FS of at least 1.5. Cross-Sections A through C were evaluated based on the configuration of the base and final waste grades and Cross-sections D through F were evaluated based on the configuration of the base and intermediate waste grades. The locations of these cross-sections are shown in Appendix C.

- Results of the sliding block analysis show that an adequate factor of safety can be achieved during interim and overall waste conditions with the following minimum large displacement interface friction values:
- Drainage Layer Sand above Geotextile;  $\phi = 12^\circ$
- Geotextile above Textured HDPE Geomembrane:  $\phi = 12^\circ$
- Low Permeability Soil below Textured HDPE Geomembrane:  $\phi = 12^\circ$

The condition being modeled contains a broad range of normal loads being applied to the liner system. As such, conformance tests that are performed to evaluate interface shear strength of the base liner system for interim slope stability should be conducted over a range of normal loads. Based on the minimum interface friction angles noted above, the required peak shear strengths that correspond to the above interface friction angles can be determined over a range of normal loads using Equation 1.

Thus, for interim slope stability confirmation, the interface shear strength conformance tests should be conducted over a range of normal loads, and results should be evaluated against the information in Table 5-1.



**Table 5-1**  
**Shear Strength Conformance Requirements for Liner System Components**

<b>Interface</b>	<b>Normal Stress <math>\sigma</math> (psf)</b>	<b>Peak Shear Stress<sup>(1)</sup> <math>\tau</math> (psf)</b>	<b>Peak Friction Angle<sup>(1)</sup> (degrees)</b>
Drainage Layer Sand above Textured HDPE	2,500	531	12°
Geomembrane and	5,000	1,063	
Geotextile above Geomembrane <sup>(2)</sup>	10,000	2,493	
Low Permeability Soil below Textured HDPE	2,500	623	12°
Geomembrane <sup>(2)</sup>	5,000	1,063	
	10,000	2,126	

#### Seismic Slope Stability Analysis

6 CRR-NY 366-4.3(d) requires a seismic analysis be performed if the landfill is located in a seismic impact zone. Per this section of the Regulations, a seismic impact zone is defined as an area with a 10 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. Using the "Unified Hazard Tool" United States Geological Society, retrieved June 6, 2019 from <https://earthquake.usgs.gov/hazards/interactive/>, the Maximum Horizontal Acceleration (MHA) at the facility is approximately 0.105g. Therefore, a pseudo-static seismic evaluation was prepared in accordance with the United States EPA Seismic Design Guidance for Municipal Solid Waste Landfill Facilities, April 1995. The same cross-sections investigated in the static slope stability analysis were examined in the pseudo-static seismic slope stability analysis in accordance with the procedure outlined in the USEPA Guidance. The minimum large-displacement interface shear strength of 14 degrees (°) was used to model the liner system interface along the entire liner system as determined from the static analyses. The minimum acceptable pseudo-static FS is 1.0 using residual interface shear strength properties. The analyses demonstrated that the pseudo-static FS for each section analyzed is greater than or approximately 1.0. As such, it is not necessary to perform further seismic deformation analysis, and the interim and final grading configuration at Dunn is considered stable relative to the peak ground acceleration of 0.105g.

## **6.0 CLOSURE AND POST-CLOSURE MAINTENANCE AND OPERATION**

### **6.1 CLOSURE DESIGN**

The conceptual closure design, developed in accordance with 6 NYCRR Part 360-2.15, is shown in plan and detail on the permit drawings. An approvable final closure plan will be submitted to the department within 60 days before the last receipt of waste, within 60 days before the last day of operating permit, or in accordance with permit requirements, whichever is earliest. The plan will comply with the requirements of 6 CRR-NY Part 363-6.6(d).

Filling of the waste disposal area will progress with side slopes of 33% (3H:1V) to an elevation of approximately 300 feet. The remaining fill will be placed at a slope of 5% across the plateau, which will encompass approximately 33 acres at full closure of the facility. As fill progression develops, areas that have achieved final design elevations will be closed and capped, as required, following the final grading plan provided by Sheet C302 of the permit drawings.

The final cover section will consist of the following components, from bottom to top, which are described subsequently. For side slope areas with slopes over 25%, the cap will consist of, from bottom to top, an intermediate cover layer, a geocomposite gas venting layer, a 40 mil LLDPE geomembrane liner, a geocomposite drainage layer, a 12-inch barrier soil layer and 6 inches of vegetative soil material. In areas where the slope is less than 25%, the cap will consist of, from bottom to top, an intermediate cover layer, a geocomposite gas venting layer, a geosynthetic clay liner (GCL), a 40 mil LLDPE geomembrane liner, a geocomposite drainage layer, a 12-inch barrier soil layer, and 6 inches of vegetative cover soil.

### **6.2 POST-CLOSURE WATER QUALITY MONITORING**

The proposed MSE Berm modification will not affect the currently permitted post-closure water quality monitoring.

### **6.3 CLOSURE OF LEACHATE COLLECTION SYSTEM AND STORAGE**

Closure of the system will be completed within 180 days after leachate collection has ceased and in accordance with the requirements set forth in Part 363-9.6.

Closure of the leachate management system will consist of removal of solid waste from the tanks, connecting lines, and all associated secondary containment systems. Solid waste removed will be properly handled and disposed of according to Federal and State requirements, and all connecting lines will be disconnected and securely capped or plugged.

Access ways to the aboveground tanks will be securely fastened in place to prevent unauthorized access. Tanks will be stenciled with the date of permanent closure.

#### **6.4 LANDFILL GAS MANAGEMENT SYSTEM**

As shown in the Engineering Drawings (Sheet C305), an active gas collection system has been constructed in the existing landfill area; this system will be expanded as additional cells are constructed and reach final grade.

#### **6.5 FINANCIAL ASSURANCE**

Currently, the site maintains financial assurance for the closure and post-closure of the currently constructed and operational phases of the landfill, which encompasses the 37 acres associated with Phases 1 through 7A.

As subsequent phases of new cells are constructed and the environmental monitoring plan is expanded to include more wells, the financial assurance closure cost estimate will be revised to reflect actual conditions.

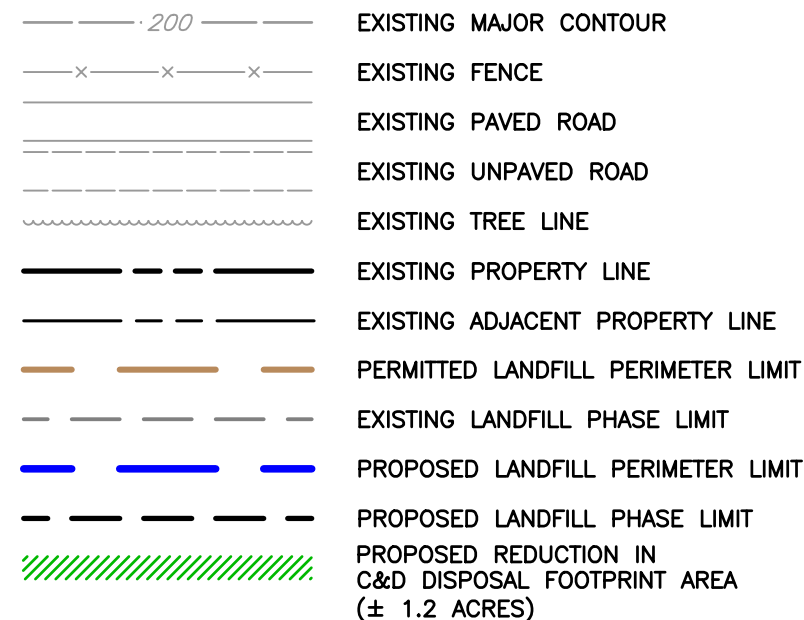
Financial assurance requirements will be maintained throughout the life of the project, until final closure.

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## FIGURES

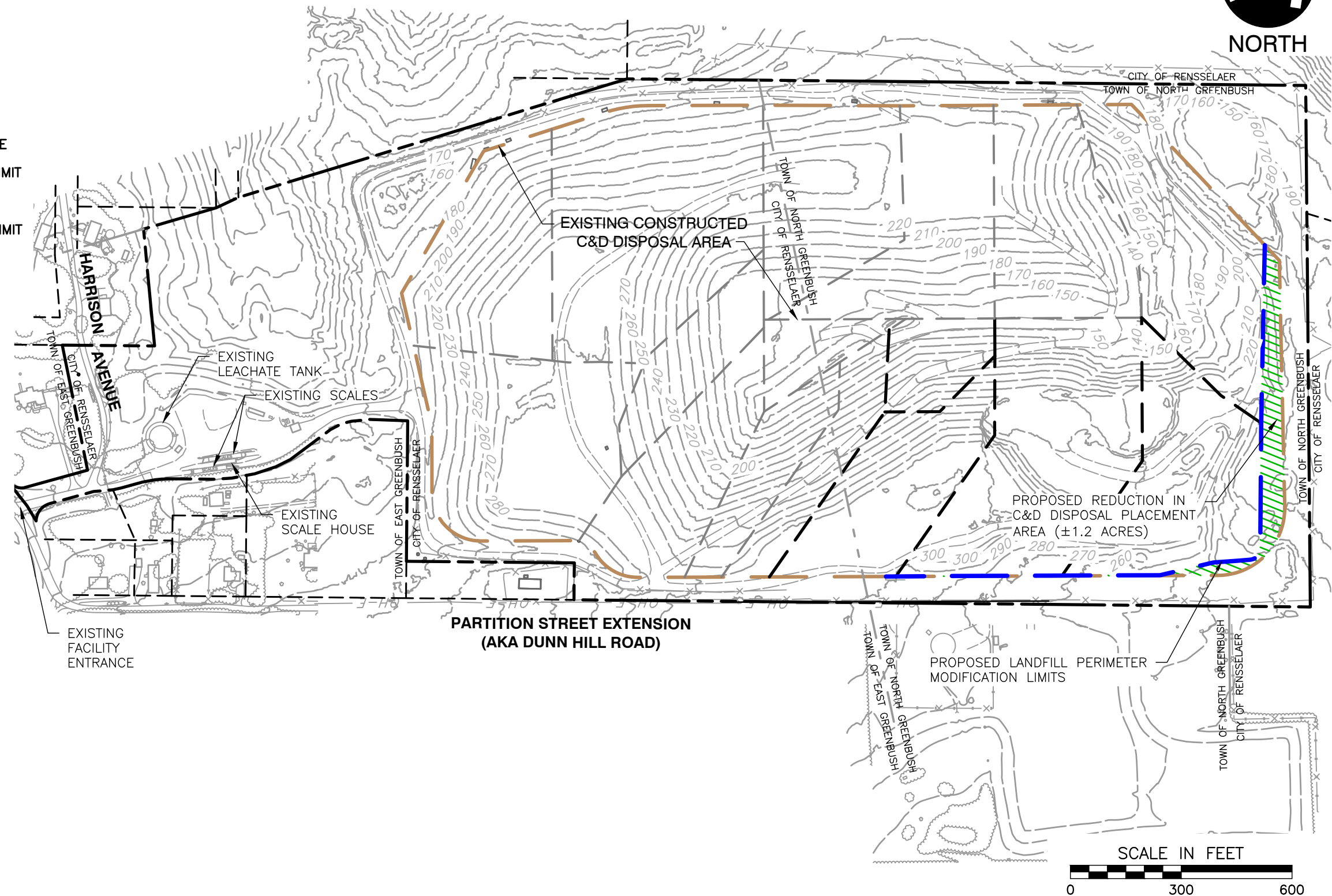
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## LEGEND



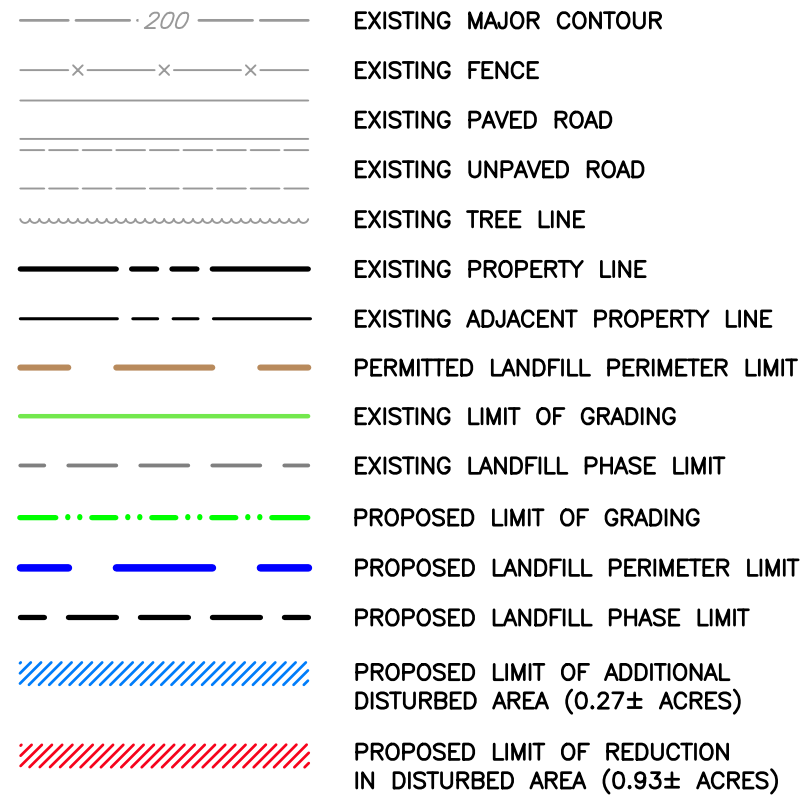
REFERENCE NOTES:

1. EXISTING TOPOGRAPHY WAS COMPILED BY SOUTHERN RESOURCES MAPPING CORPORATION USING PHOTOGRAMMETRIC METHODS FROM PHOTOGRAPHY DATED MARCH 16, 2021, AND SUPPLEMENTED WITH TOPOGRAPHY FROM OCTOBER 10, 2020 AND NOVEMBER 4, 2020. SURVEY CONTROL DATA WAS PROVIDED BY WASTE CONNECTIONS, INC.
2. PROPERTY LINE INFORMATION WAS TAKEN FROM A PLAN TITLED "ATLA/ASCM LAND TITLE BOUNDARY SURVEY, LANDS OF KELLY AND DUNN TO BE CONVEYED TO S.A. DUNN & COMPANY, LLC," PREPARED AND SEALED BY WILLIAM N. CURRAN, AND UPDATED AS NECESSARY BASED ON AVAILABLE TITLE INFORMATION.



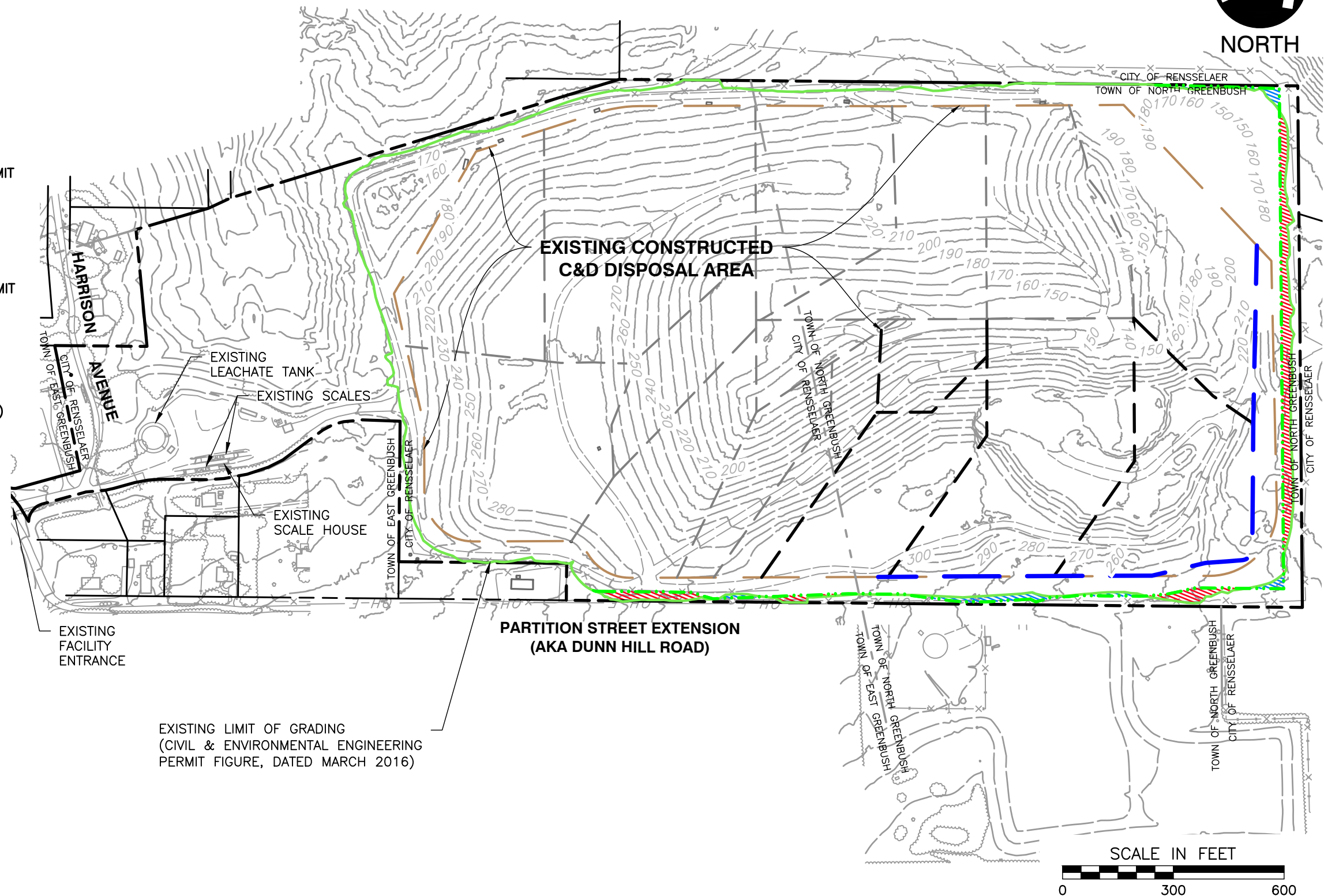
<p align="center"><b>Civil &amp; Environmental Engineering, Landscape Architecture and Land Surveying, PLLC</b></p> <p align="center">31 Bellows Road Raynham, MA 02767</p>		<p align="center"><b>MSE BERM MODIFICATION DUNN MINE AND C&amp;D FACILITY S.A. DUNN &amp; COMPANY, LLC RENSSELAER, NEW YORK</b></p>	
		<p align="center"><b>MSE BERM MODIFICATION AREA</b></p>	
<p><b>DRAWN BY:</b> TLG</p>	<p><b>CHECKED BY:</b> EMW</p>	<p><b>APPROVED BY:</b> AJK</p>	<p><b>FIGURE NO.:</b></p>
<p><b>DATE:</b> JANUARY 2022</p>	<p><b>DWG SCALE:</b> 1"=300'</p>	<p><b>PROJECT NO:</b> 182-442</p>	<p align="center"><b>1</b></p>





1. NET DISTURBED AREA IS REDUCED BY APPROXIMATELY 0.66 ACRES.

1. EXISTING TOPOGRAPHY WAS COMPILED BY SOUTHERN RESOURCES MAPPING CORPORATION USING PHOTOGRAMMETRIC METHODS FROM PHOTOGRAPHY DATED MARCH 16, 2021, AND SUPPLEMENTED WITH TOPOGRAPHY FROM OCTOBER 10, 2020 AND NOVEMBER 4, 2020. SURVEY CONTROL DATA WAS PROVIDED BY WASTE CONNECTIONS, INC.
2. PROPERTY LINE INFORMATION WAS TAKEN FROM A PLAN TITLED "ATLA/ASCM LAND TITLE BOUNDARY SURVEY, LANDS OF KELLY AND DUNN TO BE CONVEYED TO S.A. DUNN & COMPANY, LLC," PREPARED AND SEALED BY WILLIAM N. CURRAN, AND UPDATED AS NECESSARY BASED ON AVAILABLE TITLE INFORMATION.
3. THE EXISTING LIMIT OF GRADING WAS TAKEN FROM A FIGURE TITLED "ADDITIONAL DISTURBED AREA" PREPARED BY CIVIL & ENVIRONMENTAL ENGINEERING, LANDSCAPE ARCHITECTURE AND LAND SURVEYING PLLC, DATED MARCH, 2016.



<p align="center"><b>Civil &amp; Environmental Engineering, Landscape Architecture and Land Surveying, PLLC</b></p> <p align="center">31 Bellows Road Raynham, MA 02767</p>		<p align="center"><b>MSE BERM MODIFICATION DUNN MINE AND C&amp;D FACILITY S.A. DUNN &amp; COMPANY, LLC RENSSELAER, NEW YORK</b></p>	
		<p align="center"><b>DISTURBED AREA</b></p>	
<p><b>DRAWN BY:</b> TLG</p>	<p><b>CHECKED BY:</b> EMW</p>	<p><b>APPROVED BY:</b> AJK</p>	<p><b>FIGURE NO.:</b></p>
<p><b>DATE:</b> JANUARY 2022</p>	<p><b>DWG SCALE:</b> 1"=300'</p>	<p><b>PROJECT NO:</b> 182-442</p>	<p align="center"><b>2</b></p>

P:\2018\182-442\182-442-CADD\DWG\SW16-Engineering Report Figures\182442-SW16-ER Report-Figure 3-Site Vicinity Figure.dwg{11X17} LS:(1/11/2022 - ewilson) - LP: 1/12/2022 9:12 AM

LEGEND:

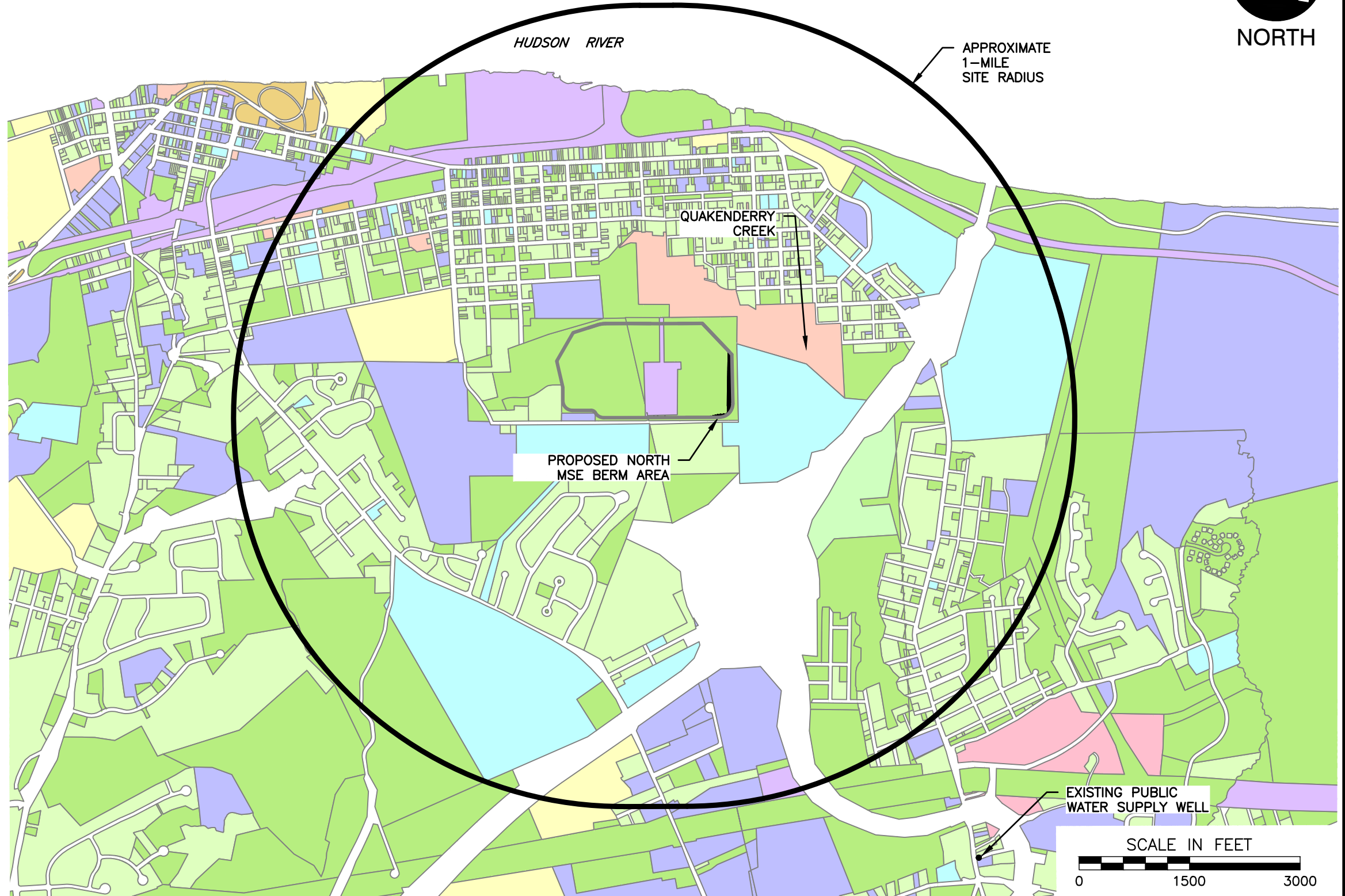
- EXISTING 10-FOOT CONTOUR  
PROPERTY BOUNDARY  
1-MILE SITE RADIUS  
EXISTING PAVED ROAD  
EXISTING WATER BODIES  
AGRICULTURAL ZONE  
COMMERCIAL ZONE  
COMMUNITY SERVICES ZONE  
INDUSTRIAL ZONE  
PUBLIC SERVICES ZONE  
RECREATION & ENTERTAINMENT ZONE  
RESIDENTIAL ZONE  
VACANT LAND ZONE  
WILD, FORESTED, CONSERVATION LAND ZONE  
PUBLIC/PRIVATE WATER SUPPLY WELL (SEE NOTE 3)

NOTES:

1. VECTOR DATA FROM OPEN STREETMAP INCLUDING ROADS, AND WATER FEATURES FOR WORLDWIDE LOCATIONS. ACCESS RASTER DATA FROM BINGMAP; GLOBAL, 90 METER SRTM TERRAIN DATA; AND US DEPARTMENT OF AGRICULTURE DATA AT 10 METER RESOLUTION.

2. ZONING/LANDUSE, AND WELL INFORMATION ARE BASED ON INFORMATION OBTAINED FROM NEW YORK GIS DATABASES.

3. THERE ARE NO PUBLIC/PRIVATE WELLS WITHIN 1-MILE OF THE PROPOSED AREA OF MODIFICATION.



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Landscape Architecture and Land Surveying, PLLC

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Raynham, MA 02767

MSE BERM MODIFICATION  
DUNN MINE AND C&D FACILITY  
S.A. DUNN & COMPANY, LLC  
RENSSELAER, NEW YORK

SITE VICINITY PLAN

DRAWN BY:	TLG	CHECKED BY:	EMW	APPROVED BY:	AJK	FIGURE NO.:
DATE:	AUGUST 2020	DWG SCALE:	1"=1500	PROJECT NO:	182-442	

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## **APPENDIX C.1**

### **STABILITY ANALYSIS**

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**S. A. DUNN C&D FACILITY  
PERMIT RENEWAL/MODIFICATION APPLICATION  
SLOPE STABILITY EVALUATION**

**INTRODUCTION**

This narrative provides a description of the slope stability evaluation performed to evaluate the stability of the landfill incorporating the mechanically stabilized earth (MSE) wall along the northern perimeter, as well as recent revisions to the operational sequencing and final grading configurations for the S. A. Dunn C&D Facility (Dunn). The landfill is located in Rensselaer County, New York. The evaluation includes an analysis of the stability of the liner system and the construction and demolition (C&D) waste fill slopes that will exist during operation of the site.

Slope stability models were evaluated using SLIDE, a slope stability modeling software to evaluate various conditions. Each model requires inputs related to landfill geometry and physical properties of the various construction materials and waste. The following sections describe the properties developed for use in the stability models.

**BASE LINER SYSTEM AND LANDFILL GEOMETRY**

The base liner system will consist of the following components from top to bottom:

1. 5 foot thick select fill sand layer;
2. 24 inches of pea gravel drainage layer material;
3. 16-ounce non-woven geotextile;
4. 60-mil textured high density polyethylene (HDPE) geomembrane;
5. 24 inches of low permeability soil; and
6. Subgrade.

**DESIGN PARAMETERS FOR SOIL COMPONENTS**

Shear strength and unit weight properties for the drainage layer, low permeability soil, and subgrade soil materials were obtained from lab results for sand materials from 2015 construction at the landfill.

**Table 1  
Soil Components Material Property Summary**

<b>Material</b>	<b>Moist unit weight (pcf)</b>	<b>Internal friction angle (deg)</b>	<b>Cohesion (psf)</b>
Select Fill Sand	132.8	33.8	107
Pea Gravel Layer	132.8	33.8	107
Low Permeability Soil	118.5	17.4	67
Subgrade Soil	127	30.6	31

## DESIGN PARAMETERS FOR C&D WASTE

Several references were reviewed to develop shear strength and unit weight properties for C&D waste. These references showed internal friction angles for C&D waste ranging from 35 to 42 degrees, and cohesion ranging from 0 to 3,500 pounds per square foot (psf). Based on a review of the reference material, a composite shear strength envelope was used to model the parameters of the C&D waste, as follows:

### C&D Waste at Normal Stresses < 2,000 psf

Unit weight = 60 pcf

Internal Friction Angle = 35°

Cohesion = 0

### C&D Waste at Normal Stresses ≥ 2,000 psf

Unit weight = 60 pcf

Internal Friction Angle = 33°

Cohesion = 0

## GEOSYNTHETIC INTERFACE SHEAR STRENGTH PROPERTIES

The minimum acceptable interface shear strength properties for the base liner system to provide a FS of at least 1.50 was determined for interim slope stability during waste filling operations, per 6 CRR-NY 366-4.3(c)(3)(iii) of the New York Solid Waste Management Facilities Regulations. As such, this evaluation provides conformance test interface shear strength criteria for the base liner system components to be installed in the remaining phases of the landfill. These criteria can be used to evaluate the shear strength of various interfaces during pre-construction activities to verify the materials supplied for cell construction will provide a factor of safety equal to or greater than (FS ≥) 1.50.

### Static Slope Stability Analysis

An analysis was performed to evaluate stability of the liner system and the C&D waste mass of the critical waste fill slope-grading configuration under interim and final conditions. Per inspection of the subgrade development and waste filling plans, CEC anticipates the critical interim waste fill slope will occur during Phase 9 waste filling. Three (3) cross sections were used to evaluate slope stability under final conditions. The location of this cross-sections is shown in the attached Static Slope Stability Calculation Brief.

The slope stability analyses include a piezometric surface located 1 foot above the base of the geosynthetic interface layer (i.e., within the sand drainage layer) in order to represent the maximum allowable hydraulic design pressure that is allowed to accumulate above the liner system.

Failures within the liner system were evaluated to determine the minimum interface friction angle required of the liner system to provide a FS ≥ 1.50. Results of the analysis show that an adequate factor of safety can be achieved during interim waste conditions with a minimum peak and large displacement interface friction value of 12 degrees (°) for all geosynthetic interfaces.

The conditions being modeled contains a broad range of normal loads being applied to the liner system. Thus, for interim and overall slope stability confirmation, the interface shear strength conformance tests should be conducted over a range of normal loads, and results should be evaluated against the information in Table 2 below.

Additionally, a veneer stability analysis performed as part of the 2018 footprint modification evaluated stability of the liner system during and immediately after construction. This evaluation included shear strength requirements under lower normal loads. As such, these previously provided shear strengths requirements for lower normal loads (i.e., 500 psf) have been included in the Table 2 below.

**Table 2**  
**Shear Strength Conformance Requirements for Liner System**

<b>Interface</b>	<b>Normal Stress <math>\sigma</math> (psf)</b>	<b>Peak/Large-Displacement Shear Strength<sup>(1)</sup> <math>\tau</math> (psf)</b>
Geosynthetic Interfaces <u>above</u> the Geomembrane	500 (from previous evaluation)	233 (from previous evaluation)
	2,500	531
	5,000	1,063
	10,000	2,126
Geosynthetic Interfaces <u>below</u> the Geomembrane	500 (from previous evaluation)	212 (from previous evaluation)
	2,500	531
	5,000	1,063
	10,000	2,126

Notes:

1. Peak/Large displacement shear strengths are provided in both friction angle and shear stress at the specified normal stress. Shear stress is calculated using the equation:  $\tau = c + (\sigma \tan \phi)$  where  $c$  equals cohesion or adhesion. Exceeding either the required friction angle with cohesion/adhesion equal to zero or the large displacement shear stress at the required normal load is acceptable.
2. For lab testing, low permeability soils shall be compacted to 85 maximum dry density and  $\pm 2$  percent of the optimum moisture content as determined by modified Proctor compaction test. Drainage layer sand shall be compacted to a dry density at a specified permeability. Geosynthetic materials shall be oriented in the machine direction. Shear rate shall be 0.04 inches per minute (in/min).

Additionally, the slope stability cross sections were analyzed to encompass failures in the waste mass. Using the waste and soil shear strength properties described above, a failure analysis of the interim and final grading configurations was performed to estimate the FS with respect to failures within the waste mass. Note, for this analysis, the geosynthetic liner system was excluded in order to limit the failure searches to the waste mass. A minimum FS of 1.50 is required by §363-4.3(c)(3)(iii) of the New York State solid waste regulations for the interim waste slopes. As shown

in the results in the attached calculation briefs, all of the calculated FS's were greater than 1.50, and therefore acceptable.

### Seismic Slope Stability Analysis

6 CRR-NY 366-4.3(d)(i) requires a seismic analysis be performed if the landfill is located in a seismic impact zone. Per this section of the Regulations, a seismic impact zone is defined as an area with a 10 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 such, an evaluation was performed to determine if the S. A. Dunn Landfill is located in a seismic impact zone. This evaluation was performed using the American Society of Civil Engineers (ASCE) 7 Hazard Tool to evaluate the maximum horizontal acceleration for the probability and return interval described above. Using this tool, the maximum horizontal acceleration for the location of the S. A. Dunn C&D Landfill is 0.105g (hazard tool output is included with attached calculation brief). As such, a seismic slope stability analysis is warranted. Per the Regulations, a minimum long-term  $FS \geq 1.0$  is required using the seismic coefficient.

To be conservative, the seismic coefficient was based on an amplified maximum horizontal acceleration to account for acceleration amplification during a seismic event from the free field PGA through the waste mass. This resulted in an amplified PGA of 0.15g and a seismic coefficient of 0.075g.

As shown in the attached Seismic slope stability calculation brief, the calculated FS's are all greater than 1.0. As such, the interim and final grading configuration at Dunn is considered stable relative to the peak ground acceleration of 0.15g.

### Summary

In summary, the calculated FS's indicate that the overall and interim grading conditions for Dunn will be stable under static and seismic conditions. Laboratory interface shear strength conformance testing should be performed on the specific products to be used in each construction increment to verify the materials used in construction of the landfill liner system are consistent with the shear strength properties listed in Table 2.

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**CALCULATION BRIEF  
STATIC SLOPE STABILITY**

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**CALCULATION BRIEF**

**S.A. DUNN C&D LANDFILL  
PERMIT RENEWAL/MODIFICATION APPLICATION  
STATIC SLOPE STABILITY**

**OBJECTIVE:** Evaluate the final and interim (i.e., during waste placement) slope configurations with respect to static slope stability at the S.A. Dunn Landfill (Dunn). This evaluation will incorporate the recent operational sequencing modification, addition of the northern mechanically stabilized earth (MSE) wall, and minor revisions to the permitted final grading configuration. The analyses will encompass failure surfaces within the construction and demolition (C&D) waste mass and base liner system. Seismic conditions are evaluated in a separate calculation brief.

**METHODOLOGY:** Use the slope stability computer software SLIDE (Ref. No. 1) to evaluate slope stability by means of evaluating failure surfaces along the liner system and within the waste mass under static conditions.

- REFERENCES:**
1. Rocscience, Inc. (2021) "SLIDE," Version 9.020.
  2. S.A. Dunn Sand & Gravel, Proposed Mined Land Use Plan Modification/C&D Landfill Project, prepared by C.T. Male Associates, P.C., amended by Sanborn, Head Engineering, P.C., last revised December 14, 2011.
  3. ASTM D5321 Direct Shear (500, 5,000, 10,000 psf) results for Sand and Clay by RSA GEOLAB, LLC, July 2015
  4. Waste Materials in Construction (pp. 225-231), edited by Th.G. Aalbers, J.J.J.M. Goumans, and H.A. van der Sloot, 1991.
  5. Construction Demolition Waste (pp. 150-154), edited by Mukesh C. Limbachiya and J. J. Roberts, 2004.
  6. Proceedings of the First International Conference on Construction Materials and Structures (p. 591), edited by Steven O. Ekelu, Morgan Dundu, and Xiaojian Gao, 2014.
  7. Kavazanjian, E. Jr., et al, "Evaluation of MSW Properties for Seismic Analysis," Geoenvironment 2000, ASCE Geotechnical Special Publication No. 46, 1995.
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8. "S.A. Dunn C&D Landfill; Operational Sequencing Modification; Static Slope Stability Evaluation", Civil Environmental Consultants, Inc., prepared July 2021.

### STATIC ANALYSIS:

The static slope stability analysis to evaluate the addition of the northern MSE wall, as well as a modified operational sequencing and final grading for Dunn, was performed using three (3) cross sections that pass through the final grades and one (1) cross-section that passes through the critical interim waste slope configuration.

The overall stability cross sections that pass through the final grading configuration were selected to represent the longest and steepest sections of the final grades. Base and final grading configurations for the overall slope stability cross-section locations are shown on the attached Figures 1 and 3, respectively.

The critical interim waste slope was determined based on a review of the proposed filling and subgrade development plan drawings. The proposed phasing for Dunn includes relatively long 3 horizontal to 1 vertical (3H:1V) interim waste fill slopes that terminate along relatively flat portions (approximately 2% slope) of the permitted base grades. Due to the relatively flat and consistent configuration of the base grades, the critical interim waste slope occurs at the longest steepest 3H:1V interim waste fill slope. The longest steepest 3H:1V interim waste fill slope occurs during the filling of Phase 9 and Phase 10C. As such, the critical interim waste slope was evaluated to represent landfill conditions as of development of Phase 9. The interim grading configuration for the Phase 9 filling with the stability cross section is shown on the attached Figures 2.

The stability of the landfill largely depends on the shear strength properties of the soils, waste, and geosynthetic interfaces of the components used in its construction and operation. This analysis incorporates several design parameters for C&D waste and the Critical Liner System Interface used in Reference Number (Ref. No.) 2, and laboratory results for stockpiled sources of drainage sand, low permeability soil, and subgrade soils that were used during previous landfill cell construction events (Ref. No. 3). The laboratory results are included in an attachment to this calculation brief. Material properties used in this slope stability evaluation are further summarized below:

Shear Strength and Unit Weight Properties of Select Fill Sand Layer, Pea Gravel Drainage Layer (Ref. No. 3):

Moist Unit Weight = 132.8 pcf

Angle of Internal Friction ( $\phi$ ) = 33.8 degrees

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Cohesion (c) = 107 pounds per square foot (psf)

Shear Strength and Unit Weight Properties of Low Permeability Soil (Ref. No. 3):

Moist Unit Weight = 118.5 pcf

$\phi$  = 17.4 degrees

c = 67 psf

Shear Strength and Unit Weight Properties of Subgrade Soil (Ref. No. 3):

Moist Unit Weight = 127 pcf

$\phi$  = 30.6 degrees

c = 31 psf

Shear Strength for and Unit Weight Properties of Waste:

C&D Unit Weight = 60 pounds per cubic foot (pcf) [Ref. No. 2]

Shear strength properties for the C&D waste material were determined using data from Ref. Nos. 4, 5, and 6, as well as typical municipal solid waste (MSW) properties (Ref. No. 7) for comparison. The table and graph below present the peak shear strength properties determined for the various waste materials.

**Table 1 – Waste Shear Strength Properties**

	<b>C&amp;D Waste - Source 1 (Ref. No. 4)</b>	<b>C&amp;D Waste - Source 2 (Ref. No. 5)</b>	<b>C&amp;D Waste - Source 3 (Ref. No. 6)</b>	<b>Typical MSW</b>
<b>Friction Angle (degrees)</b>	42	35	42	0 for $0 < \sigma < 500$ psf, 33 for $\sigma > 500$ psf
<b>Cohesion (psf)</b>	0	0	3,500	500 psf for $0 < \sigma < 500$ psf, 0 for $\sigma > 500$ psf





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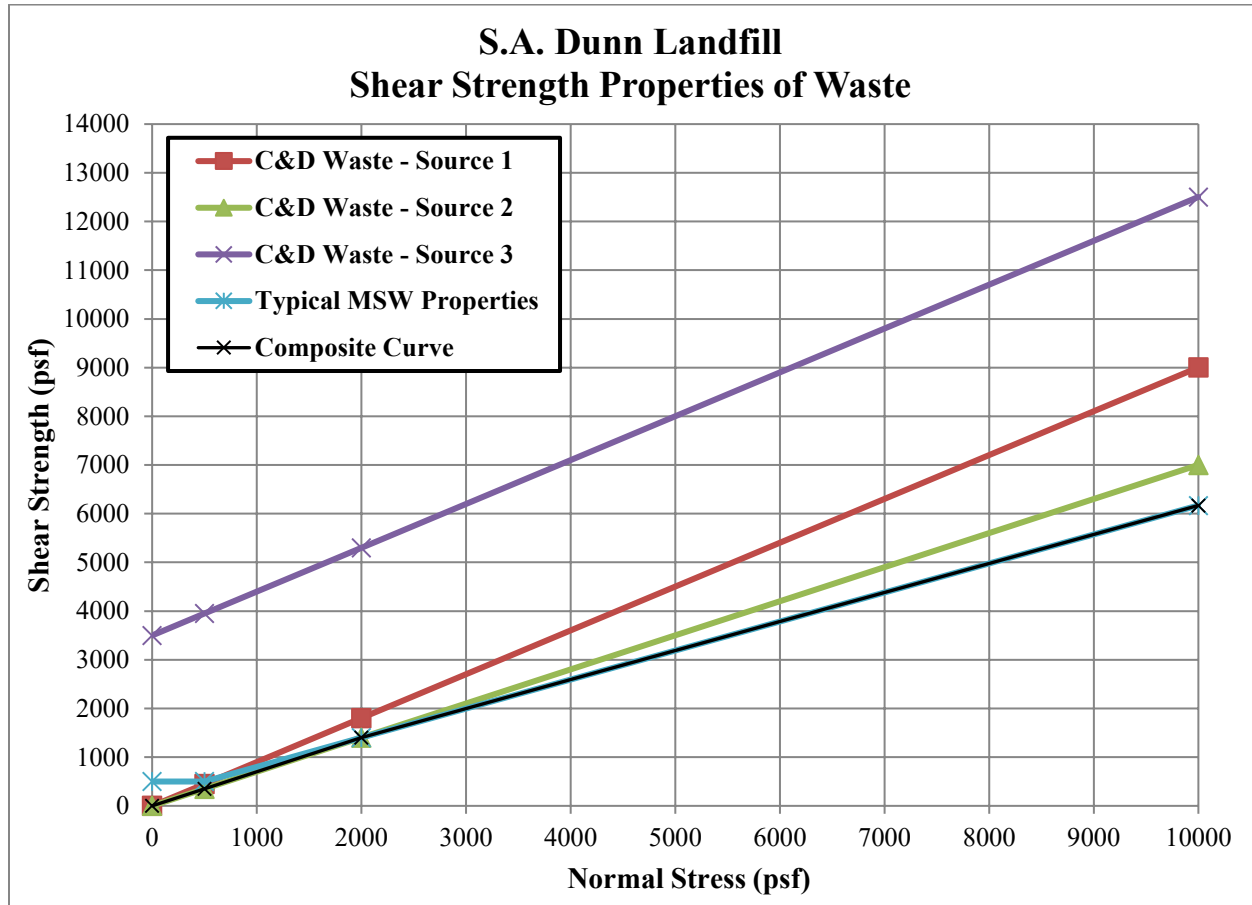
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As shown in the above graph, a composite curve was developed to represent the lowest anticipated peak shear strength properties of the C&D waste at different normal stresses. These normal and shear stress parameters were then input into the SLIDE program to model the C&D waste mass.

Additionally, this slope stability analysis includes a piezometric surface located 1 foot above the base of the geosynthetic interface layer (i.e., within the sand drainage layer) in order to represent the maximum allowable design pressure that is allowed to accumulate above the liner system.



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Shear Strength for and Unit Weight Properties for the Critical Liner System Interface

The Dunn base liner system will consist of the following (listed from top to bottom):

- 5-foot thick select fill sand layer;
- 24-inch pea gravel drainage layer;
- 16 ounce non-woven geotextile;
- 60-mil textured high density polyethylene (HDPE) geomembrane; and
- 24-inch subbase comprised of low permeability soil.

The critical liner system interface was modeled with the following unit weight properties:

Critical Liner System Interface Unit Weight = 60 pcf [Typical]

The critical liner system interface was modeled using a cohesion value of 0 psf, and the friction angle was varied to determine the minimum allowable friction angle required to obtain a Factor of Safety (FS) of 1.5. The critical liner system interface value is defined as the value that produces a minimum FS of 1.5, required by §363-4.3(c)(3)(iii) of the New York State solid waste regulations. Results of the analyses are presented through the remainder of this calculation brief.

**STATIC SLOPE STABILITY ANALYSIS RESULTS**

*“Failure along the Liner System” Analysis*

The cross sections were analyzed to determine the minimum friction angle required for the critical liner system interface to obtain an adequate Factor of Safety. The Cross Section D was controlling, therefore the liner interface strengths for the other cross sections were also analyzed using a similar interface friction angle. The results of the analyses are as follows:

**Table 2 – “Failure along the Liner System” Analysis Results**

<b>Cross Section</b>	<b>Failure Location</b>	<b>Interface Friction Angle</b>	<b>Factor of Safety (≥1.5 required)</b>
A	Liner System	12°	2.23
B	Liner System	12°	2.11
C	Liner System	12°	3.45
D (Interim)	Liner System	12°	1.59 (Controlling)



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Outputs from the SLIDE program for the static slope stability analyses are included at the end of this calculation brief.

Based on the values listed above, the required shear strengths that correspond to the above friction angles can be determined over a range of normal loads from the following equation:

$$\tau = \sigma \tan \phi$$

With:  $\tau$  = shear strength [pounds per square feet (psf)]  
 $\sigma$  = normal strength (psf)  
 $\phi$  = interface friction [degrees (deg)]

Thus, for interim stability confirmation, the interface shear strength conformance tests should be conducted over a range of normal loads, and results should be evaluated against the values in Table 3 below. Please note, interface shear strength conformance values are for both the peak large-displacement shear strength test results.

**Table 3 – Shear Strength Conformance Requirements**

Interface	Normal Stress $\sigma$ (psf)	Peak/Large- Displacement Shear Strength <sup>(1)</sup> $\tau$ (psf)
All Geosynthetic Interfaces	2,500	531
	5,000	1,063
	10,000	2,126

Notes:

1. Peak/Large displacement shear strengths are provided in both friction angle and shear stress at the specified normal stress. Shear stress is calculated using the equation:  $\tau = c + (\sigma \tan \phi)$  where  $c$  equals cohesion or adhesion. Exceeding either the required friction angle with cohesion/adhesion equal to zero or the large displacement shear stress at the required normal load is acceptable.

*“Failure in the Waste” Analysis*

The slope stability cross-sections were also analyzed to encompass critical locations and combinations of waste mass, base grades, and interim grades. Using the waste and soil shear strength properties described above, a failure analysis of the final and interim grading configurations was performed to estimate the FS with respect to failures within the waste mass. Note, for this analysis, the geosynthetic liner system was excluded in order to limit the failure searches to the waste mass. A minimum FS of 1.5 is required by §363-4.3(c)(3)(iii) of the New York State solid waste regulations for the interim waste slopes. Results of the analyses are presented in Table 4 below.



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**Table 4 – “Failure in the Waste” Analysis Results**

<b>Cross-Section</b>	<b>Failure Location</b>	<b>Factor of Safety (≥1.5 required)</b>
A	C&D Waste Mass	2.11
B	C&D Waste Mass	2.11
C	C&D Waste Mass	2.47
D (Interim)	C&D Waste Mass	1.57

**CONCLUSIONS:** Laboratory interface shear strength conformance testing should be performed on the specific products to be used in each construction increment to verify the materials used in construction of the landfill liner system are consistent with the shear strength properties listed in Table 3 above. Additionally, the final and interim grading configurations are stable with respect to failure in the waste mass under static conditions.

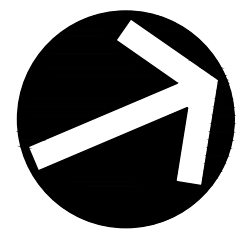
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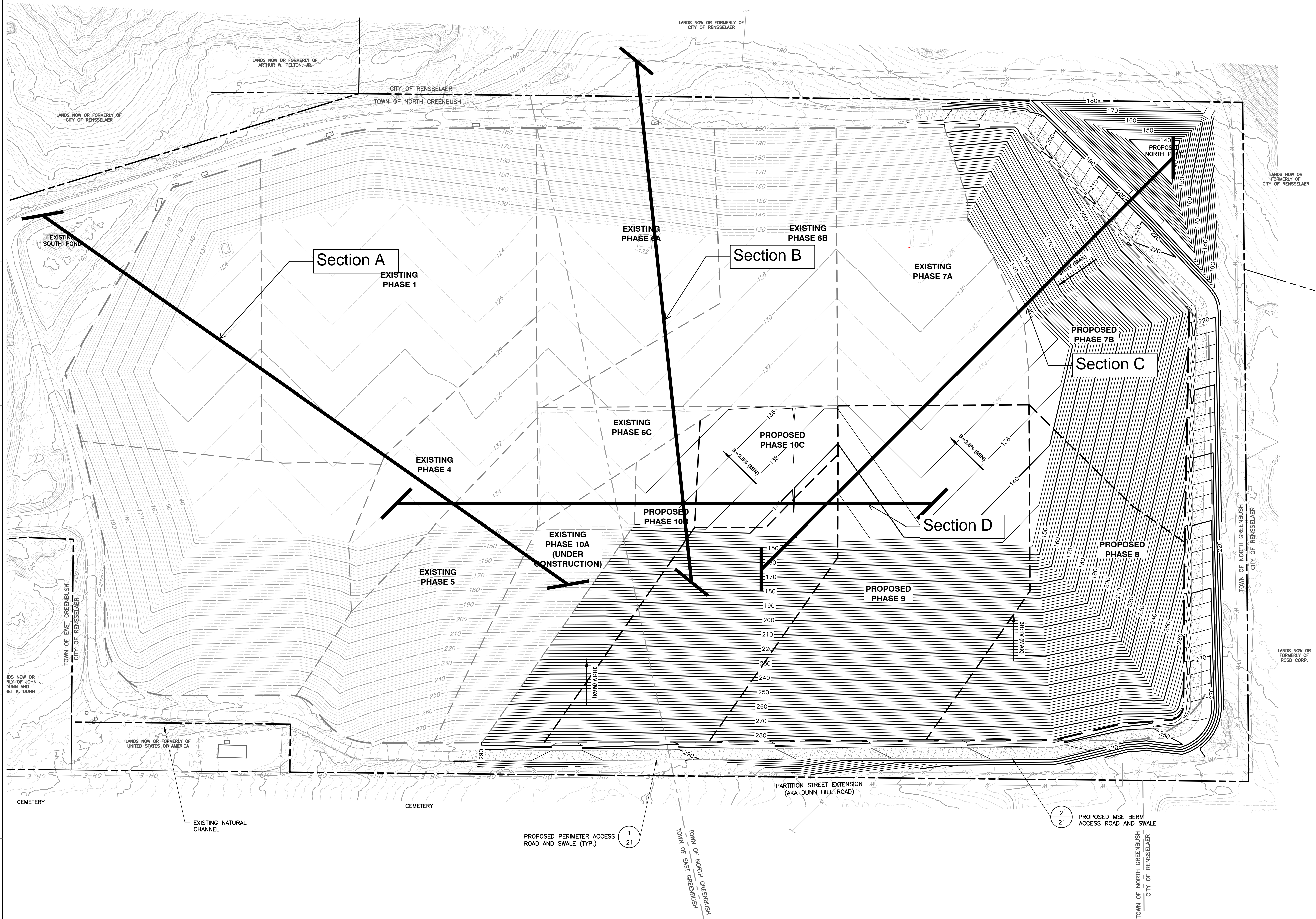
## FIGURES

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# Figure 1 - Permitted Base Grade Configuration



LEGEND	
	EXISTING PROPERTY LINE
	EXISTING ADJACENT PROPERTY LINE
	EXISTING TOWN LINE
	EXISTING LIMIT OF FRANCHISE AGREEMENT
	EXISTING WATER MAIN EASEMENT
	PERMITTED LANDFILL PERIMETER LIMIT
	EXISTING LANDFILL PHASE LIMIT
	PROPOSED LANDFILL PERIMETER LIMIT
	EXISTING MAJOR CONTOUR
	EXISTING MINOR CONTOUR
	EXISTING FENCE
	EXISTING PAVED ROAD
	EXISTING UNPAVED ROAD
	EXISTING TREE LINE
	EXISTING STRUCTURE
	EXISTING WATER MAIN
	EXISTING FORCE MAIN
	PROPOSED MAJOR CONTOUR
	PROPOSED MINOR CONTOUR
	PROPOSED ACCESS ROAD

## REFERENCE

## REVISION RECORD

NO.	DATE	DESCRIPTION

Civil & Environmental Engineering,  
Landscape Architecture and Land Surveying, PLLC

31 Bellows Road  
Raynham, MA 02767

NORTH BERM MODIFICATION  
DUNN MINE AND C&D FACILITY  
S.A.DUNN & COMPANY, LLC  
RENSSELAER, NEW YORK

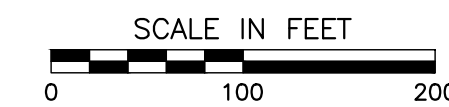
## PROPOSED SUBGRADE ELEVATION PLAN

DATE:	MAY 2021	DRAWN BY:	KFH
DWG SCALE:	1"=100'	CHECKED BY:	AJK
PROJECT NO:	182-442	APPROVED BY:	AJK

DRAWING NO:  
**C300**

SHEET 3 OF 23

## LINER SYSTEM SECTION

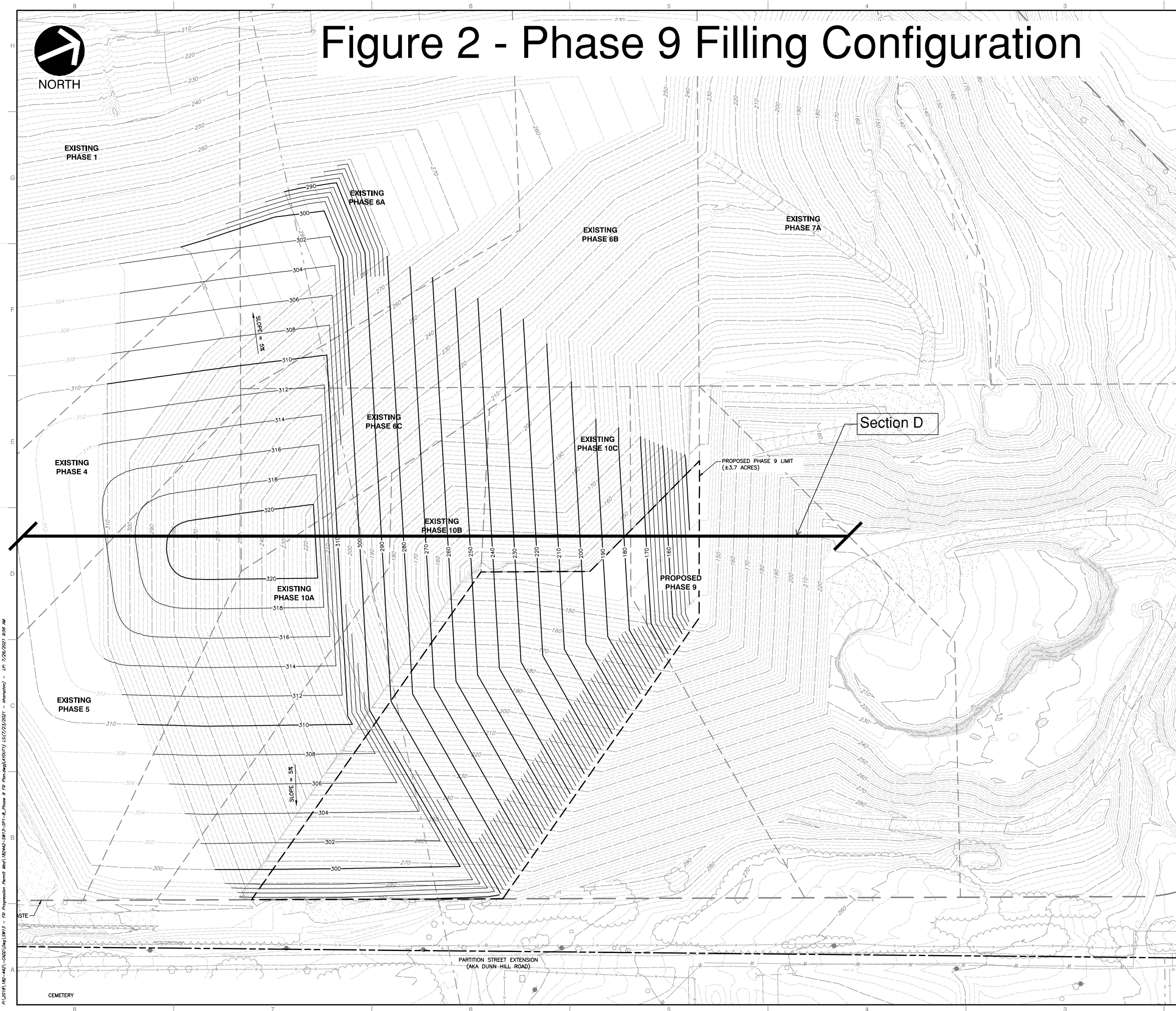


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SECTION 7209, SUB-DIVISION 2, OF THE  
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PROFESSIONAL'S SEAL SHALL BE  
CONSIDERED TO BE VALID TRUE COPIES.






















## Figure 2 - Phase 9 Filling Configuration



## LEGEND

	EXISTING PROPERTY LINE
	EXISTING ADJACENT PROPERTY LINE
	EXISTING TOWN LINE
	EXISTING LIMIT OF FRANCHISE AGREEMENT
	PERMITTED LANDFILL PERIMETER LIMIT
	EXISTING LANDFILL PHASE LIMIT
	PROPOSED LANDFILL PERIMETER LIMIT
	PROPOSED LANDFILL PHASE LIMIT
	EXISTING MAJOR CONTOUR
	EXISTING MINOR CONTOUR
	EXISTING FENCE
	EXISTING PAVED ROAD
	EXISTING UNPAVED ROAD
	EXISTING TREE LINE
	EXISTING STRUCTURE
	PROPOSED MAJOR CONTOUR
	PROPOSED MINOR CONTOUR

[illegible]

**Civil & Environmental Engineering,  
Landscape Architecture and Land Surveying, PLLC**

**S.A. DUNN & COMPANY, LLC  
DUNN MINE AND C&D FACILITY  
RENSSELAER, NEW YORK**

## PHASE 9 FILLING PLAN

DATE:	JULY 2021	DRAWN BY:	KFH
DWG SCALE:	1"=60'	CHECKED BY:	CSD
PROJECT NO:	182-442		

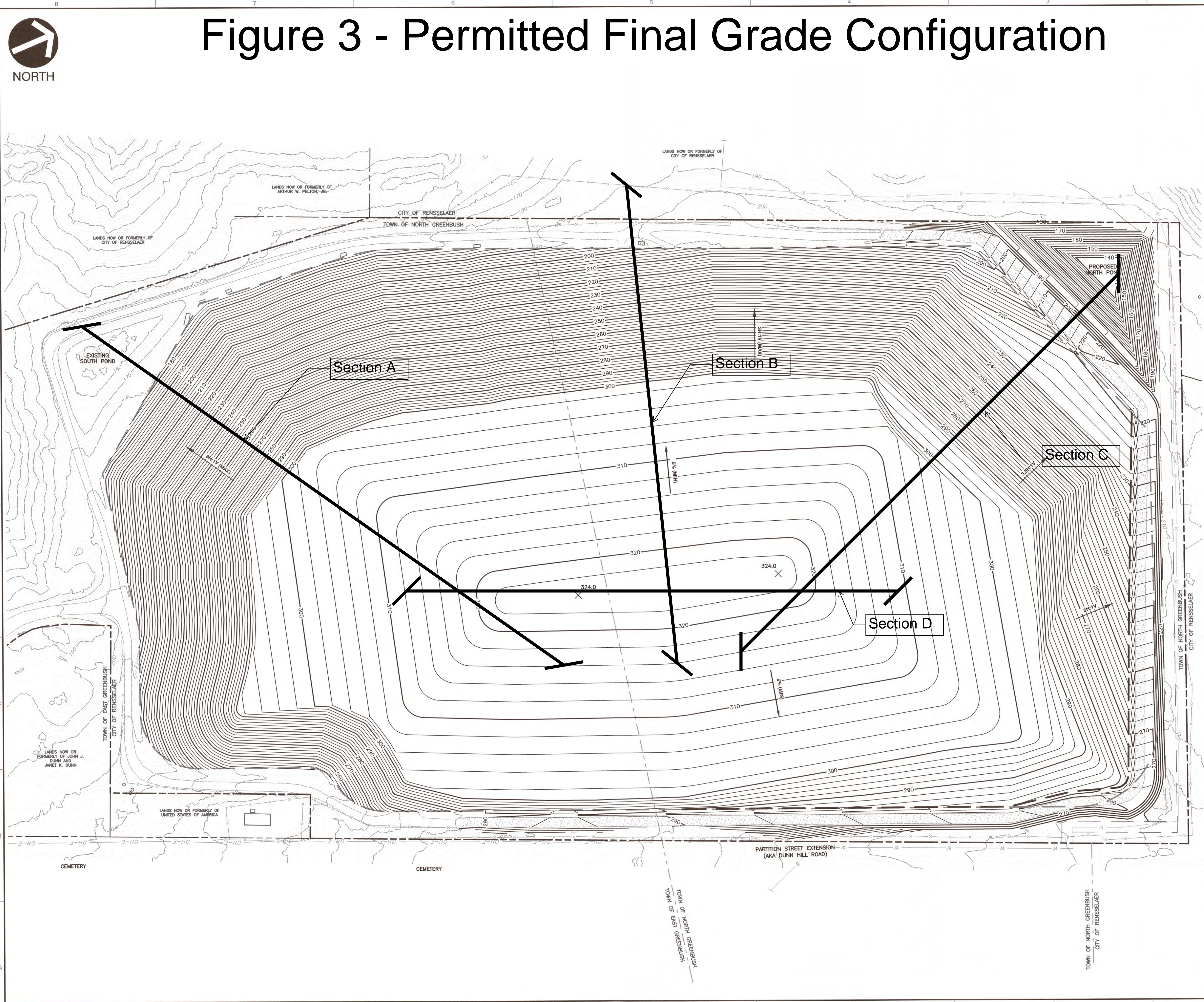
FIGURE NO.

8

05

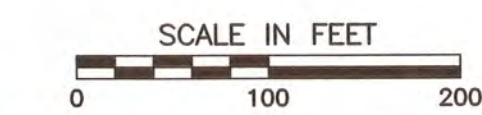


P:\2018\182-442-C302\DWG\SW11 - Road Cover Modification\182442-SW11 - C302-Proposed Final Cover Elevation Plan.dwg(3/20/2021 - 1P, 5/18/2021 4:23 PM)



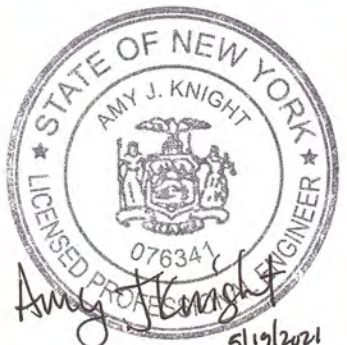
**LEGEND**

---	EXISTING PROPERTY LINE
---	EXISTING ADJACENT PROPERTY LINE
---	EXISTING TOWN LINE
---	EXISTING LIMIT OF FRANCHISE AGREEMENT
---	PERMITTED LANDFILL PERIMETER LIMIT
---	EXISTING LANDFILL PHASE LIMIT
---	PROPOSED LANDFILL PERIMETER LIMIT
---	PROPOSED LANDFILL PHASE LIMIT
---	EXISTING MAJOR CONTOUR
---	EXISTING MINOR CONTOUR
---	EXISTING FENCE
---	EXISTING PAVED ROAD
---	EXISTING UNPAVED ROAD
---	EXISTING TREE LINE
---	EXISTING STRUCTURE
---	EXISTING WATER MAIN
---	EXISTING FORCE MAIN
---	PROPOSED MAJOR CONTOUR
---	PROPOSED MINOR CONTOUR
---	PROPOSED ACCESS ROAD



UNAUTHORIZED ALTERATION OR ADDITION TO THIS PLAN BEARING A LICENSED PROFESSIONAL'S SEAL IS A VIOLATION OF SECTION 7209, SUB-DIVISION 2, OF THE NEW YORK STATE EDUCATION LAW.

ONLY COPIES FROM THE ORIGINAL OF THIS PLAN MARKED WITH AN ORIGINAL PROFESSIONAL'S SEAL SHALL BE CONSIDERED TO BE VALID TRUE COPIES.



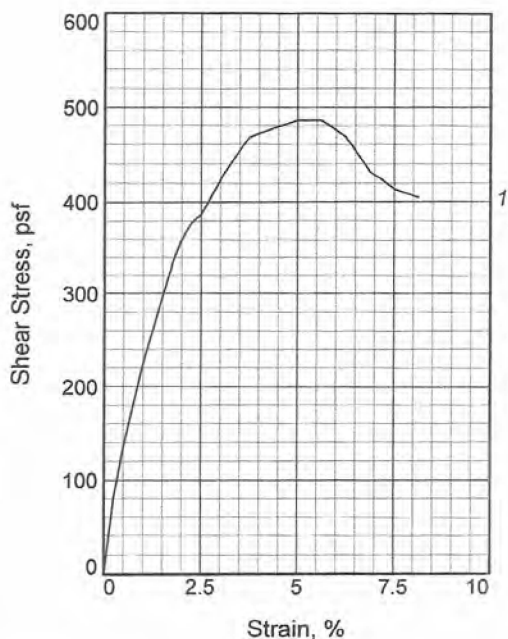
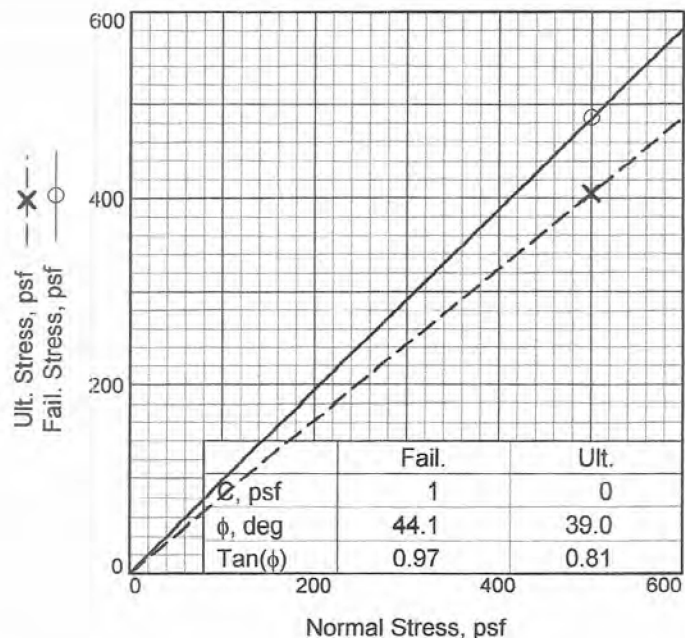
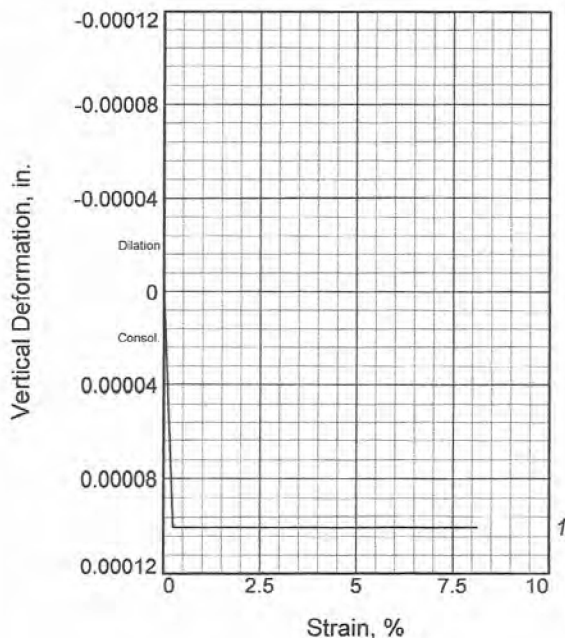
REVISION RECORD		PROPOSED FINAL COVER ELEVATION PLAN		NORTH BERM MODIFICATION DUNN MINE AND C&D FACILITY S.A. DUNN & COMPANY, LLC RENSSELAER, NEW YORK		Civil & Environmental Engineering, Landscape Architecture and Land Surveying, PLLC 31 Bellows Road Rayham, MA 02767	
NO.	DATE	DESCRIPTION	DATE	DRAWN BY:	CHECKED BY:	PROJECT NO.	APPROVED BY:
1				KFH	AJK	182-442	AJK
2							
3							
4							
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23							



---

**DIRECT SHEAR STRENGTH RESULTS  
SITE SOILS  
S.A. DUNN C&D LANDFILL**

---



Sample No.		1
Initial	Water Content, %	4.8
	Dry Density, pcf	113.1
	Saturation, %	25.5
	Void Ratio	0.5127
	Side Length, in.	4.00
	Height, in.	1.00
At Test	Water Content, %	17.0
	Dry Density, pcf	113.2
	Saturation, %	91.2
	Void Ratio	0.5115
	Side Length, in.	4.00
	Height, in.	1.00
Normal Stress, psf		500
Fail. Stress, psf		486
Strain, %		5.0
Ult. Stress, psf		405
Strain, %		8.1
Strain rate, in./min.		0.008

**Sample Type:** ASTM D3080

**Description:** Dark Brown poorly graded sand

**LL=** NV

**PI=** NP

**Assumed Specific Gravity=** 2.74

**Remarks:** Recompacted sample. -3/8" material tested. Tested with water in box.

**Figure** \_\_\_\_\_

**Client:** Civil Environmental Consultants, Inc.

**Project:** Dunn Landfill 151-336 SA

**Sample Number:** SIFT-3

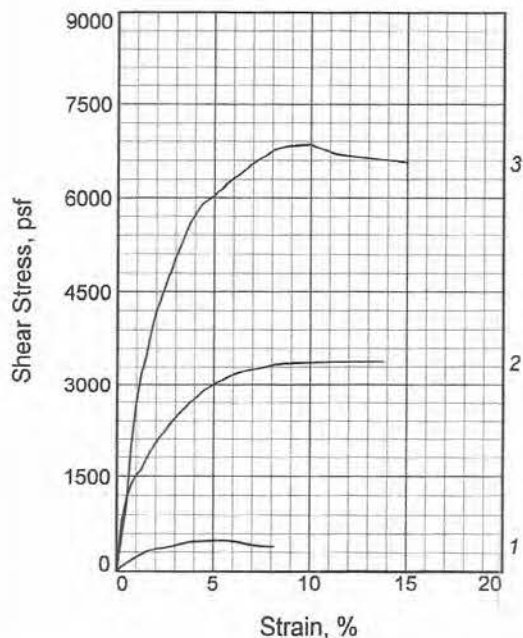
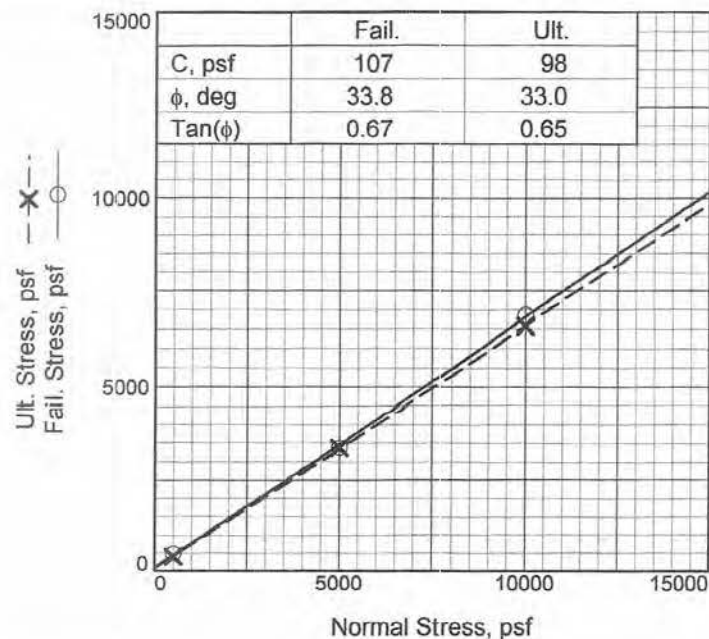
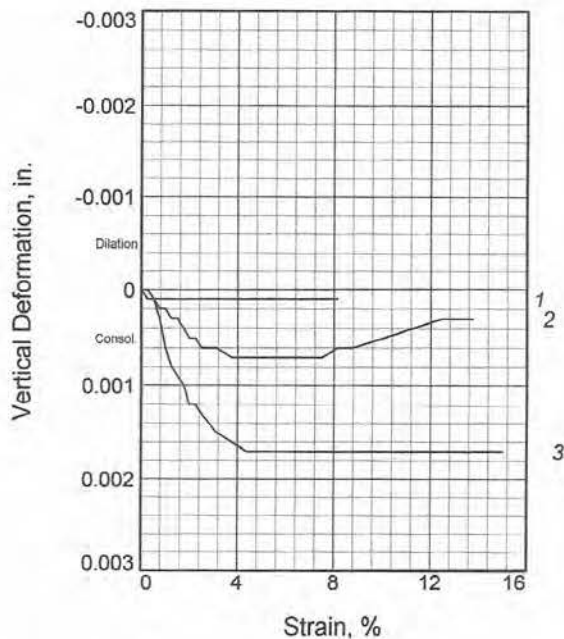
**Proj. No.:** 848

**Date Sampled:** 8-4-15

**DIRECT SHEAR TEST REPORT**  
 RSA Geolab  
 Union, New Jersey

**Tested By:** KK

**Checked By:** KP



Sample No.		1	2	3
Initial	Water Content, %	4.8	5.0	4.5
	Dry Density, pcf	113.1	112.9	113.4
	Saturation, %	25.5	26.4	24.1
	Void Ratio	0.5127	0.5155	0.5084
	Side Length, in.	4.00	4.00	4.00
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.0	15.8	16.9
	Dry Density, pcf	113.2	113.9	114.6
	Saturation, %	91.2	86.5	94.1
	Void Ratio	0.5115	0.5015	0.4922
	Side Length, in.	4.00	4.00	4.00
	Height, in.	1.00	0.99	0.99
Normal Stress, psf		500	5000	10000
Fail. Stress, psf		486	3375	6849
Strain, %		5.0	12.5	10.0
Ult. Stress, psf		405	3375	6570
Strain, %		8.1	13.8	15.0
Strain rate, in./min.		0.008	0.008	0.008

**Sample Type:** ASTM D3080

**Description:** Dark Brown poorly graded sand

**LL=** NV

**PI=** NP

**Assumed Specific Gravity=** 2.74

**Remarks:** Recompacted sample. -3/8" material tested. Tested with water in box.

**Figure** \_\_\_\_\_

**Client:** Civil Environmental Consultants, Inc.

**Project:** Dunn Landfill 151-336 SA

**Sample Number:** SIFT-3

**Proj. No.:** 848

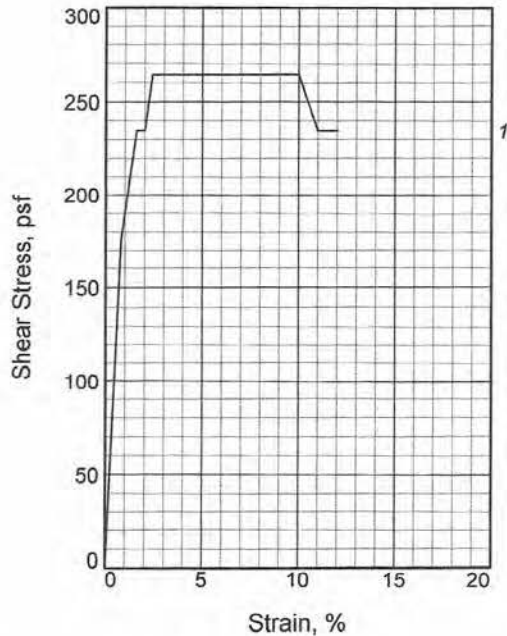
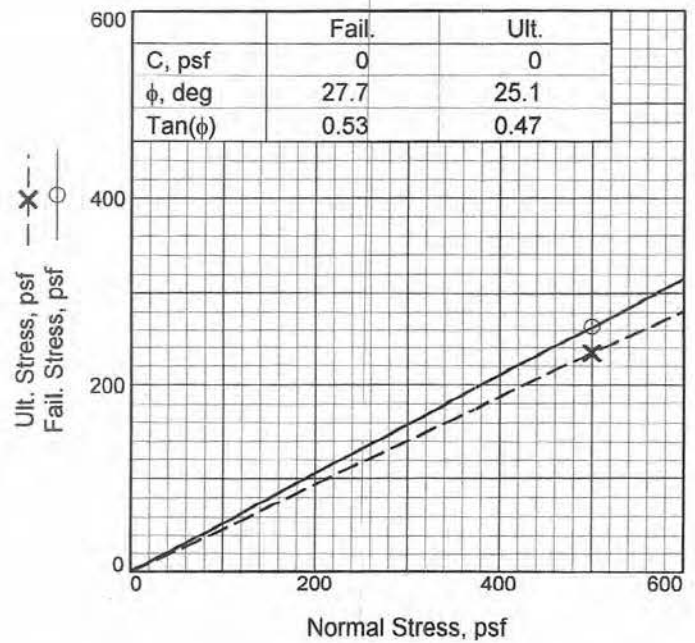
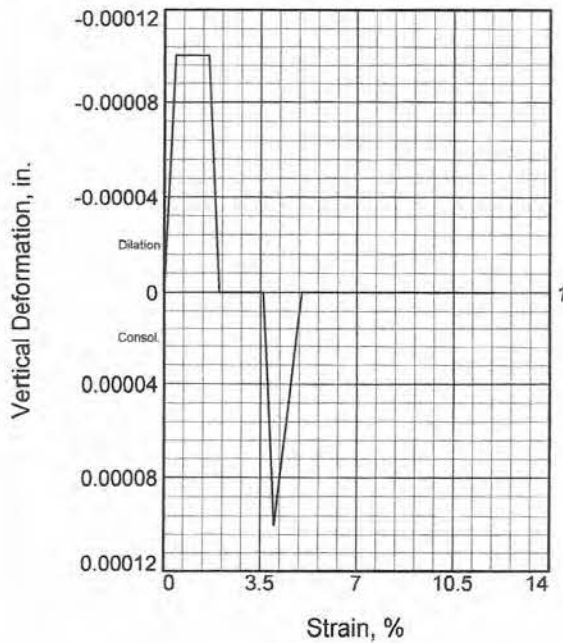
**Date Sampled:** 8-4-15

**DIRECT SHEAR TEST REPORT**  
**RSA Geolab**  
**Union, New Jersey**

**Tested By:** KK

**Checked By:** KP





Sample No.		1
Initial	Water Content, %	22.7
	Dry Density, pcf	97.7
	Saturation, %	83.5
	Void Ratio	0.7383
	Diameter, in.	2.50
	Height, in.	1.00
At Test	Water Content, %	29.0
	Dry Density, pcf	97.7
	Saturation, %	106.9
	Void Ratio	0.7379
	Diameter, in.	2.50
	Height, in.	1.00
Normal Stress, psf		500
Fail. Stress, psf		264
Strain, %		2.4
Ult. Stress, psf		235
Strain, %		12.0
Strain rate, in./min.		0.003

**Sample Type:** ASTM D3080

**Description:** Gray lean clay

LL= 41      PL= 22      PI= 19

**Assumed Specific Gravity=** 2.72

**Remarks:** Recomacted sample. Tested with water in box.

**Figure** \_\_\_\_\_

**Client:** Civil Environmental Consultants, Inc.

**Project:** Dunn Landfill 151-336 SA

**Location:** Sample #2 Grey Clay Swent Slope Cell I

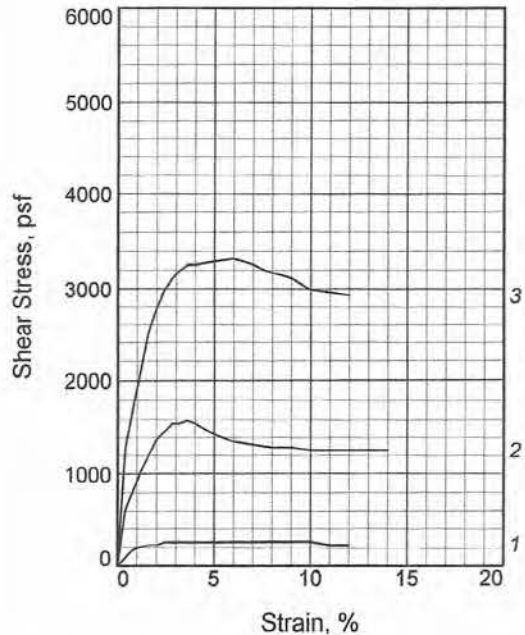
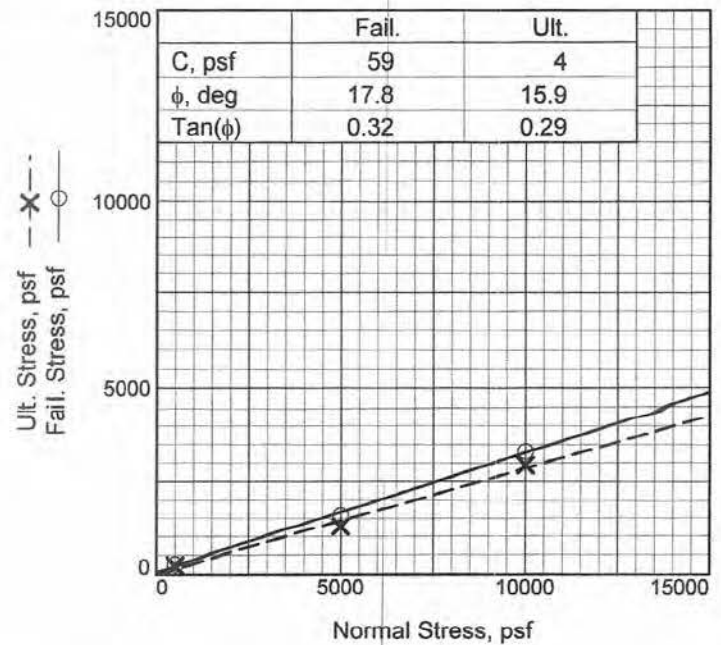
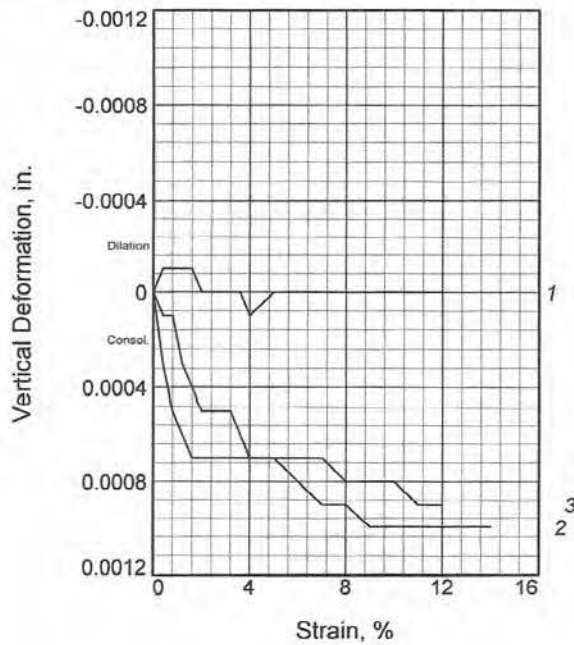
**Proj. No.:** 848

**Date Sampled:** 5-7-15

**DIRECT SHEAR TEST REPORT**  
RSA Geolab  
Union, New Jersey

**Tested By:** KH

**Checked By:** JW



Sample No.		1	2	3
Initial	Water Content, %	22.7	22.9	23.1
	Dry Density, pcf	97.7	97.5	97.3
	Saturation, %	83.5	84.0	84.4
	Void Ratio	0.7383	0.7417	0.7445
	Diameter, in.	2.50	2.50	2.50
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	29.0	24.5	24.4
	Dry Density, pcf	97.7	98.5	98.2
	Saturation, %	106.9	91.9	91.0
	Void Ratio	0.7379	0.7243	0.7290
	Diameter, in.	2.50	2.50	2.50
	Height, in.	1.00	0.99	0.99
Normal Stress, psf		500	5000	10000
Fail. Stress, psf		264	1584	3315
Strain, %		2.4	3.6	6.0
Ult. Stress, psf		235	1261	2934
Strain, %		12.0	14.0	12.0
Strain rate, in./min.		0.003	0.003	0.003

**Sample Type:** ASTM D3080

**Description:** Gray lean clay

**LL= 41      PL= 22      PI= 19**

**Assumed Specific Gravity= 2.72**

**Remarks:** Recomacted sample. Tested with water in box.

**Figure** \_\_\_\_\_

**Client:** Civil Environmental Consultants, Inc.

**Project:** Dunn Landfill 151-336 SA

**Location:** Sample #2 Grey Clay Swent Slope Cell 1

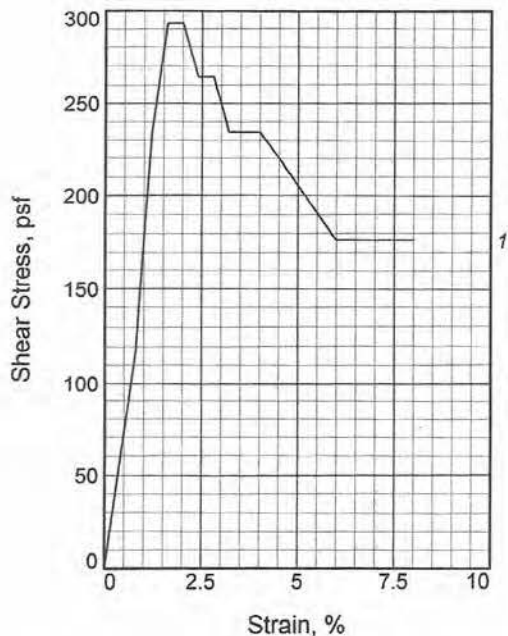
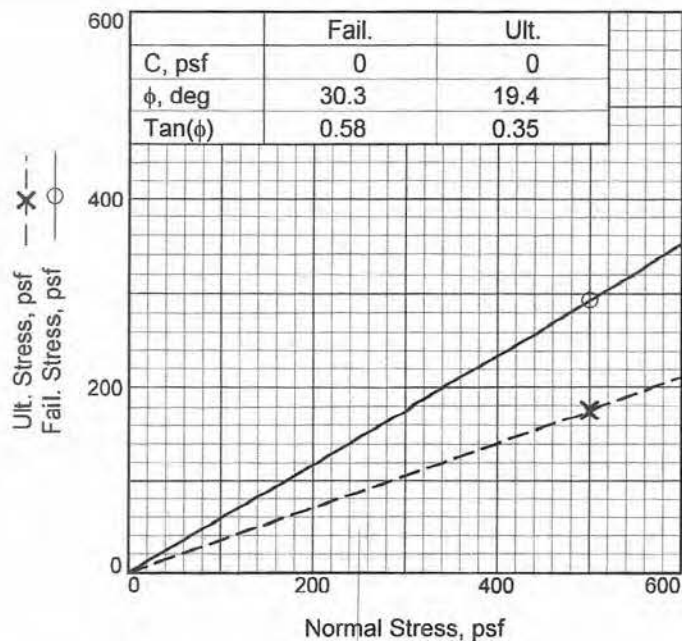
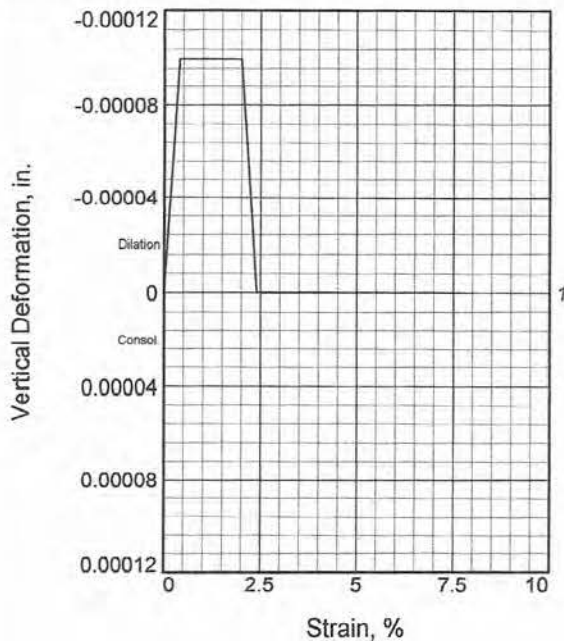
**Proj. No.:** 848

**Date Sampled:** 5-7-15

**DIRECT SHEAR TEST REPORT**  
**RSA Geolab**  
**Union, New Jersey**

**Tested By:** KH

**Checked By:** JW



Sample No.		1
Initial	Water Content, %	23.8
	Dry Density, pcf	90.8
	Saturation, %	74.4
	Void Ratio	0.8707
	Diameter, in.	2.50
	Height, in.	1.00
At Test	Water Content, %	27.9
	Dry Density, pcf	91.0
	Saturation, %	87.4
	Void Ratio	0.8670
	Diameter, in.	2.50
	Height, in.	1.00
Normal Stress, psf		500
Fail. Stress, psf		293
Strain, %		1.6
Ult. Stress, psf		176
Strain, %		6.0
Strain rate, in./min.		0.003

**Sample Type:** ASTM D3080

**Description:** Light Olive Brown lean clay

**LL= 47      PL= 25      PI= 22**

**Assumed Specific Gravity= 2.72**

**Remarks:** Recomacted sample. Tested with water in box.

**Figure** \_\_\_\_\_

**Client:** Civil Environmental Consultants, Inc.

**Project:** Dunn Landfill 151-336 SA

**Location:** Sample #1 Brown Clay SW Slope Cell 1

**Proj. No.:** 848

**Date Sampled:** 5-6-15

**DIRECT SHEAR TEST REPORT**  
**RSA Geolab**  
**Union, New Jersey**

**Tested By:** KH

**Checked By:** JW