STATE OF MAINE
DEPARTMENT OF ADMINISTRATIVE & FINANCIAL SERVICES
BUREAU OF GENERAL SERVICES
BURTON M. CROSS BUILDING
4TH FLOOR, 77 STATE HOUSE STATION
AUGUSTA, MAINE
04333-0077

PAUL R. LEPAGE
GOVERNOR

RICHARD W. ROSEN
COMMISSIONER

EDWARD A. DAHL
DIRECTOR

March 4, 2016

Ms. Kathy Tarbuck P.E.
Division of Technical Services
Bureau of Hazardous Materials & Solid Waste
Maine Department of Environmental Protection
17 State House Station
Augusta, ME 04333

Subject: Response to Department Staff's Review Comments on the Juniper Ridge Landfill Expansion Application, MEDEP #S-020700-WD-BI-N

Dear Kathy:

The Maine Bureau of General Services (BGS) and NEWSME Landfill Operations, LLC (NEWSME) have prepared responses to review comments and technical recommendations on the Juniper Ridge Landfill Expansion (JRL) Application, MEDEP #S-020700-WD-BI-N, as contained in and attached to a January 22, 2016 letter from the Department authored by Michael T. Parker, Project Manager, Division of Materials Management. The letter included four attachments:

1) Fame Correspondence: consisting of an email dated January 6, 2016 from the Finance Authority of Maine (FAME), Subject Casella Waste Systems, Inc. authored by Christopher Roney.

2) DOT Comments: consisting of a memorandum dated December 28, 2015 from the Maine Department of Transportation authored by Stephen Landry, P.E. State Traffic Engineer MaineDOT.

3) Comments of R. Behr: consisting of a memorandum dated January 15, 2016 from the Department's Division of Technical Services Bureau of Remediation and Waste Management authored by Richard S. Behr Environmental Hydrogeology Specialist Certified Geologist GE # 342.

4) Comments of S. Farrar, et al.: consisting of a memorandum dated January 20, 2016 from the Department's Division of Technical Services Bureau of Remediation and Waste Management, and Division of Watershed Management authored by Stephen E. Farrar, P.E. Environmental Service Specialist; Victoria Eleftheriou, P.E. Environmental Engineering Service Manager; and Ken Libbey, Jr., P.E. Environmental Engineer.

PHONE: (207) 624-7314 www.Maine.gov FAX: (207) 287-4039

As you are aware, Staff's comments and recommendations were discussed during a January 29 meeting between the Department Staff and the applicant, and our responses provided herein are based on that discussion. We've provided our responses in the form of three exhibits, which are attached to this letter. Exhibit A responds to the Department's, FAME's, and MDOT's comments on the Chapter 400 evidence contained in the Application. Exhibit B responds to the comments and recommendations contained in R. Behr January 15, 2016 memorandum. Exhibit C responds to the comments and recommendations contained in S. Farrar, et al. January 20, 2016 memorandum.

We appreciate the opportunity to respond to these comments.

Sincerely,

Edward A Dahl

Edward A. Dahl
Director, Bureau of General Services

Brian Oliver
Vice-President, NEWSME Landfill Operations, LLC

cc: Service List

EXHIBIT A

<u>BGS AND NEWSME'S RESPONSE TO DEP'S JANUARY 22, 2016 TECHNICAL REVIEW LETTER</u>

Below BGS and NEWSME set forth each of Staff's comments in the January 22 letter and follow each comment with our response.

<u>Chapter 400.4.B, Financial Ability</u>: The projected total cost for the design and construction of the proposed expansion is $24.6 million. Construction of Cell 11, slated for 2018, is estimated at $6.24 million. A letter from Bank of America, N.A. was submitted with the application that demonstrates a secured credit facility of $190 million, of which $38 million is currently available to cover the costs of design and construction of the expansion. Staff comments that this secured credit facility is available for Casella Waste Systems, Inc. and all its wholly-owned subsidiaries, including NEWSME Landfill Operations, LLC. The cost of ongoing operations, estimated to be $7.0 million per year, will be financed by revenues generated from the operation of the landfill, such as tipping fees. Finally, the cost for closure and post-closure care of the facility is estimated to be $21.1 million. NEWSME Operations, LLC maintains a surety bond, currently in the amount of $21,072,243, for the closure and post-closure care of the landfill. Staff comments that the period for the primary surety bond (#853746) expired on September 12, 2015. A current Continuation Certificate needs to be provided and updated annually.

In addition to the supporting documentation submitted with the application, staff accessed and reviewed the 2014 Corporate Annual Report for Casella Waste Systems, Inc. to verify financial commitments and environmental liabilities associated with other Casella subsidiaries. Finally, staff verified the status of bonds issued through the Finance Authority of Maine (FAME). FAME staff confirmed that the Casella makes payments to bondholders directly or through a trustee, that FAME has no direct exposure in the case of default on the bonds and that Casella is considered to be in good standing with no payment defaults. A copy of the correspondence with FAME is attached.

> <u>Response</u>: NEWSME provided Staff with updated surety bond riders for closure and post-closure care of JRL at the January 29 meeting with Staff, and copies are again provided here as Attachment 1 for convenience. No additional response to the above comment is necessary, as this comment, along with the related evidence in the application, demonstrate that NEWSME and BGS satisfy the financial ability and financial assurance standards of the statute and rules.

<u>Chapter 400.4.D, Traffic</u>: Staff have reviewed all the statements and supporting information contained in Volume I, Section 3.4 and Volume I, Appendix E of the application. In addition, the Maine Department of Transportation (MDOT) conducted a similar review of the same submittals. Both MEDEP and MDOT comment that the slight increase (3 trips in the peak hour) will not result in the need to modify roadways or

intersections in the vicinity of the landfill, that there are no high crash locations in the area that will be impacted by the proposed development and that a traffic study is not warranted. A copy of MDOT's comments is attached.

Response: No response necessary.

Chapter 400.4.E, Fitting the Facility Harmoniously into the Natural Environment: Staff have reviewed all the statements and supporting information contained in Volume I, Sections 3.5 and 3.6, and Volume I, Appendix F of the application. Staff comment that three Significant Wildlife Habitats are located within the boundaries of the property on which the expansion area is located, but are likely not to be impacted by the proposed development. Further, correspondence from the U. S. Fish and Wildlife Service (USFWS) notes that critical habitat for Atlantic Salmon (*Salmo salar*), a federally and Maine-listed endangered species, lies within the watershed of the project. Staff comment that a final determination by the USFWS or the Army Corps of Engineers on potential impacts to critical habitat of Atlantic Salmon associated with the proposed expansion has not been issued.

Response: After the issuance of Staff's January 22 letter, DEP's Kathy Tarbuck and Lynn Caron conveyed comments from Maine IF&W and DMR. In an email dated October 16, 2015, IF&W's John Perry, the agency's Environmental Review Coordinator, stated that "[m]inimal additional impacts to wildlife are anticipated," and "[b]ased upon the proposal as presented, fisheries staff do not anticipate any adverse impacts on fisheries resources associated with this landfill expansion."

In an email dated February 1, 2016, DMR's Oliver Cox, of the Division of Sea Run Fisheries and Habitat, commented that "[n]one of the stream[s] in the project area are Atlantic salmon stream[s]."

Chapter 400.4.F, No Unreasonable Adverse Effect on Existing Uses and Scenic Character: Staff have reviewed all the statements and supporting information contained in Volume I, Section 3.6, and Volume I, Appendices F, G and H of the application. Staff comment that on bottom of page 7-6 of the Sound Level Assessment Report[1], there is reference to Figure 7-1 through 7-6. Staff could only locate Figures 7-1 and 7-2. This is likely a typographical error, however, if not, please submit the additional figures. Further, it is stated at the bottom of page 9-1 of the same Report that "Operational restrictions will be necessary in certain regions of the western expansion area during the one hour of nighttime operations in order to comply with the noise limits." For the purposes of compliance, the applicant should clarify which of the mobile equipment listed in Table 7-1 of the Report will not be operating in the western expansion area during the one hour of nighttime operations (6:00 a.m. to 7:00 a.m.).

[1] *Sound Level Assessment Report Juniper Ridge Landfill Expansion – Old Town, Maine*. Epsilon Associates, Inc., July 7, 2015

Volume I, Appendix F of the application contains correspondence from the Maine Historic Preservation Commission stating that there will be no historic properties affected by the expansion and that a Phase I archeological investigation will not be required.

Staff comment that the Visual Assessment Report[2] was prepared using both the definition of "public viewing area" contained in 06-096 CMR 400.1.LI and "scenic resource" contained in 06-096 CMR 315.5.H of the Departments rules. Further, the visual assessment study area was expanded out to a distance of 6 miles, well beyond the 2,000 feet specified in 06-096 CMR 400.4.F(3)(b) of the Solid Waste Rules and the City of Old Town's ordinance.

> <u>Response</u>: The commenter is correct: there are no figures beyond Figure 7-2 in the Sound Level Assessment Report. That was a typo.
>
> In response to the question regarding which of the mobile equipment listed in Table 7-1 will not be operating during the one hour of nighttime operations (6:00 a.m. to 7:00 a.m.), we note that the constraint placed on equipment operation on the western side of the expansion during the 6:00 a.m. to 7:00 a.m. hours is that the landfill equipment cannot exceed a combined sound level of 77 dBA from the equipment, during these hours. Therefore, any <u>single piece</u> of equipment included on Table 7-1 could be operated, within 60 feet of the western solid waste boundary during this short time period, because they all have sound levels less than 77 dBA.
>
> No additional response is necessary.

<u>Chapter 400.4.G, No Unreasonable Adverse Effect on Air Quality</u>: See the January 20, 2016 memorandum from MEDEP Technical staff on the landfill gas management plans and operations.

> <u>Response</u>: We address DEP Technical Staff's comments on the landfill gas management plans and operations in our response to the January 20, 2016 technical recommendations and review comments on the Juniper Ridge Landfill engineering comments on the Expansion #S-020700-WD-BI-N dated March 4, 2016.

<u>Chapter 400.4.H, No Unreasonable Adverse Effect on Surface Water Quality</u>: Staff have not identified any facet of the siting or operation of the proposed expansion that would cause the facility to discharge any water pollutants that would affect the state classification of a surface water body. Further, staffs analysis shows that there are no "waterbodies most at risk from new development" within the watershed of the proposed expansion. Staff note, as stated by the applicant, the existing Stormwater Pollution Prevention Plan will need to be updated to include and address changes brought about by the proposed expansion.

---

[2] *Visual Assessment Report Juniper Ridge Landfill – Old Town, Maine*. SMRT Architects and Engineers, July, 2015

Response: We agree; the existing Stormwater Pollution Prevention Plan will be updated, as needed, to address any changes brought about by the expansion. No further response necessary.

Chapter 400.4.I, No Unreasonable Adverse Effect on Other Natural Resources: The NRPA application submitted as part this overall project is still under review pending responses from outside reviewers, including Maine Inland Fisheries and Wildlife, the USFWS and the Army Corps of Engineers.

Response: Subsequent to Staff's January 22 letter, Staff forwarded comments on the NRPA application. As noted above, neither Maine IF&W nor DMR expressed concerns about the expansion project. Moreover, as the DEP's James Beyer stated at the February 10 pre-hearing conference, while the Army Corps process is a parallel federal wetland permitting process, the NRPA process is an independent state permitting process and the DEP is not "waiting" on Corps (or presumably USFWS) comments.

Chapter 400.4.N, Solid Waste Management Hierarchy: Staff have reviewed all the statements and supporting information contained in Volume I, Section 3.14 of the application. In addition, staff reviewed data contained in the 2013 and 2014 Annual Reports for the Juniper Ridge Landfill, the 2014 Annual Report for the Hawk Ridge Landfill and summaries of 2014 data for the generation, disposal and utilization of residuals in Maine. These last data were compiled by the Department from annual reports for calendar year 2014. In general, the information contained in the application regarding the application of the solid waste hierarchy adequately identified and addressed those wastes that are sufficiently within the control of the applicant to manage or facilitate. Staffs analysis of the summary of wastes accepted at JRL determined that seven categories of wastes accounted for 88.7% of the wastes accepted at the facility. These are mixed CDD (199,000 tons), CDD processing residue – fines (126,000 tons), FEPR (57,000 tons), MSW ash (54,000 tons), CDD processing residue - bulky waste (48,000 tons), Municipal WWTP/POTW sludge (38,000 tons) and MSW (37,000 tons). Of these seven categories, FEPR and MSW ash currently have no other viable management option. CDD processing residue – fines and CDD processing residue - bulky waste are arguably largely generated from the processing of out-state wastes. However, these wastes are considered in-state wastes, as they are generated at processing facilities located in Maine and the fines are used as daily cover to the extent possible in accordance with the statutes and rules governing these wastes. The Department analyzed the use of fines as daily cover at JRL as part of its review of the Public Benefit Determination and noted no irregularities in this practice. Mixed CDD, the largest category of waste accepted at JRL, is generated at many sources in Maine, some of which are under the direct control of the applicant. Staff comments that the applicant should provide additional detail on current and future efforts to decrease the amount of mixed CDD sent to JRL. In reviewing the 2014 Annual Report, staff noted efforts by the applicant to divert MSW from the landfill to other facilities higher on the hierarchy, including ecomaine and MMWAC. Staff note that agreements between these facilities were executed late in 2014 and would not be reflected in the 2014 Annual Report. The

applicant should continue to divert MSW to these facilities and provide data on the quantities of MSW diverted to these facilities in 2015. Finally, staff comment that 38,000 tons of Municipal WWTP/POTW sludge was accepted at JRL in 2014, of which approximately 28,000 was generated by Portland, South Portland and Rockland. By comparison, the 2014 Annual Report for the Casella-owned Hawk Ridge Compost facility accepted 27,000 tons of Maine-generated biosolids and 24,000 tons of out-of-state biosolids. Staff are aware that there is limited capacity for land applying and composting biosolids. However, the applicant stated that biosolids from Maine sources in excess of the limitations must be disposed in a secure landfill. Staff propose that a large portion of the Maine-generated biosolids could be managed at the Hawk Ridge facility if out-of-states sources were managed through options other than JRL.

Response:

CDD: Staff comments that some of the mixed CDD accepted at JRL "is generated at many sources in Maine, some of which are under the direct control of the applicant." NEWSME is not a generator of CDD; it is not involved in the construction and demolition of structures. It is the generators of CDD in Maine – contractors and homeowners – that directly control the management and destination of the waste streams they create. Those generators may choose to deliver their CDD to a transfer station that is owned and/or operated by a sister company of NEWSME. Prior to disposal, however, materials such as clean wood and metal are removed, sorted and recycled at these transfer stations.

Maine CDD generators are also provided the option to source-separate clean wood and deliver it to JRL's clean wood pad for on-site processing and for beneficial use on site or for sale to offsite users as a biomass fuel.

Casella has an agreement with ReEnergy Lewiston to deliver to that CDD processing facility all of the CDD that is collected by Casella within the boundaries of Poland, Minot, Auburn, Lewiston, Sabattus, Green, Turner, Livermore and Wales. In 2015, 3,979 tons of CDD were delivered to ReEnergy Lewiston pursuant to that agreement.

We also note that CDD movement to waste facilities within the state is based on commercially reasonable factors, such as proximity, cost of transportation and tip fees.

MSW: In the JRL Expansion, the Applicants propose to accept only MSW bypass. Existing JRL, however, as a result of the 2012 closure and sale of Maine Energy is licensed to accept up to 81,800 tons of Maine MSW annually until March 31, 2018.

The following table summarizes information on Casella's efforts to divert Maine MSW from JRL during 2015. We've also included the same data from 2014 for comparison.

EXHIBIT A

MSW DIVERSION FROM JUNIPER RIDGE LANDFILL	2014	2015
Maine MSW Recyclables Delivered to Casella Zero-Sort in Lewiston, ME and Boston, MA		
Number of Maine municipalities participating in Casella Zero-Sort program:	52	62
Number of Maine businesses participating in Casella Zero-Sort program:	3,200	3,482
Tons of Maine MSW recyclables processed in Casella Zero-Sort program	25,026	28,688
Casella cardboard recycling: Fiber from Maine municipalities, Maine businesses, or transfer stations (tons).		
Brokered	37,385	53,244
Collected / Baled	12,840	29,071
Maine MSW delivered by Casella to Maine incinerators (tons):		
a. PERC:	89,902	89,054
b. ecomaine:		
i. Lewiston Zero-Sort processing residue:	97	329
ii. Single-stream recyclables:	42,506	11,430
iii. MSW:		41,130
c. MMWAC:		
i. Lewiston Zero-Sort processing residue:	-	1,742
ii. MSW:	147	32,212
Maine MSW delivered by Casella to Maine landfills other than Juniper Ridge (tons):		
a. Bath Landfill:	388	6,097
b. Brunswick Landfill:	10,144	528
c. Fort Fairfield Landfill:	7,249	10,500
d. Norridgewock Landfill:	2,495	2,720
The total Maine MSW diverted from disposal at JRL through efforts described above (tons):	228,179	306,745
THE TOTAL MAINE MSW DISPOSED OF AT JRL (TONS)	38,516[1]	62,662[2]

Notes:

1. This includes 1,638 tons of MSW Bypass from PERC
2. This includes 5,141 tons of MSW Bypass from PERC

Municipal Wastewater Treatment Plant Sludge: There are several reasons that are not within the Applicants' control why municipal wastewater treatment plant sludge, only about six percent of the waste disposed at JRL in 2014 and 2015, is diverted to disposal at JRL by Casella Organics:

- The biosolids do not meet the regulated standards for recycling. Biosolids from Biddeford, Houlton, Bangor, Greater Augusta Utility District, and Portland Water District's Westbrook facility have all been landfilled because they cannot always meet regulatory screening standards required for composting.
- Biosolids quality is not preferred for composting. Low solids content biosolids, such as those from Rockland, are more cost-effectively managed by landfilling. Low solids sludges require more bulking agent and therefore lead to higher costs to the municipality.

- Some Maine municipalities, such as Portland and South Portland, put significant value on the cost savings component of their biosolids management programs. These generators did not require recycling in their bid processes and having multiple biosolids management options, including landfilling at JRL, resulted in cost savings being realized by the municipalities.

Casella Organics, whose corporate name is New England Waste Services of ME, Inc., always tries to keep its Hawk Ridge Compost Facility full to the maximum extent practicable. Biosolids from sources such as Portland and South Portland, although often landfilled, are important seasonally to Hawk Ridge to keep the facility full when other biosolids generators' volumes are reduced.

Casella Organics is a market leader in managing biosolids in the Northeast and access to a variety of biosolids and biosolids processing and disposal options supports the ongoing operation of the Hawk Ridge Compost Facility at or near its permitted processing capacity. Casella Organics' efficient operation of the Hawk Ridge Compost Facility assures that this recycling option will remain a viable option in the solid waste management hierarchy for biosolids.

It should be noted that the operation of the Hawk Ridge composting facility in 2014, as described in the staff memo, represents significant and dramatic compliance with the solid waste management hierarchy: if Hawk Ridge had not composted approximately 27,000 tons of Maine-generated biosolids, that waste would most likely have been disposed at JRL, which would have brought the 2014 total to approximately 55,000 tons disposed rather than 28,000 tons.

Finally, we don't understand the last sentence of Staff's comment on this topic ("Staff propose that a large portion of the Maine-generated biosolids could be managed at the Hawk Ridge facility if out-of-states [sic] sources were managed through options other than JRL"). No out-of-state wastes are managed at JRL. To the extent this proposal is to manage the acceptance of out-of-state waste at Hawk Ridge, a separate privately-owned commercial enterprise, through the JRL expansion license, it raises serious Constitutional issues protected by the Commerce Clause.

<u>Chapter 400.12, Civil and Criminal Disclosure Statement</u>: Staff comment that civil criminal disclosure must be expanded to include Casella Waste Systems, Inc., the parent company of both New England Waste Services of Maine, Inc. and New England Waste Services of Maine Landfill Operations, LLC. A cursory review of the organization of Casella Waste Systems, Inc. and its subsidiaries, as shown in Volume I, Appendix Q of the application, shows a direct link to the management and control of the various entities. Also, some of the documentation and agreements contained in the application, such as the letter from Bank of America, specifically name Casella Waste Systems, Inc. The expanded disclosure must address all the pertinent information on Casella's other subsidiaries, including those operating in other states and countries, as required in 06-096 CMR 400.12

EXHIBIT A

Response: We discussed this comment with DEP Staff on January 29. In responding to the requirement in the regulations for a civil and criminal disclosure statement, the Applicants have provided what the regulations require under Chapter 400.12.A for disclosure statements and have provided information for all entities and individuals called for by that rule.

We respectfully disagree with Staff's suggested interpretation of Chapter 400.12 above, which goes well beyond what is required by that regulation. Moreover, in the recent JRL amendment license for the acceptance of a finite amount of MSW as a result of the closure of Maine Energy, the Applicants provided disclosure statements for the same entities and individuals as in this proceeding (i.e., those required by Chapter 400.12.A(1)(b)), and the disclosure statement was accepted and approved by the Department. Nothing has changed in Chapter 400.12 since that DEP approval.

At Staff's request, and for clarity purposes, we are providing an organizational chart of the Casella companies authorized to do business in Maine. A copy of this organizational chart is appended as Attachment 2. We note, however, that only the yellow highlighted companies are actively operating in Maine. Several companies exist in name only. For example, Maine Energy Recovery Company sold its site in Biddeford to the City and the facility there has been demolished. Additionally, the KTI Biofuels facility in Lewiston was sold more than a year ago to ReEnergy, which now owns and operates that facility. We also have explained to Staff, and make clear here, that Casella Organics and Pine Tree Landfill are trade names (i.e., d/b/a's) for New England Waste Services of ME, Inc., and thus these entities are covered by the disclosure statement submitted in the application. Thus, the disclosure statement in the application addresses the disclosure requirements for BGS and NEWSME Landfill Operations, LLC, the applicants, as well as New England Waste Services of ME, Inc. and all other persons required by Chapter 400.12.A (1)(b).

List of Attachments

Attachment 1 Updated Surety Bond Rider
Attachment 2 Casella Companies Authorized to do business in Maine

ATTACHMENT 1

UPDATED SURETY BOND RIDER

Facility Name: Juniper Ridge Landfill –Closure
Maine DEP Site No.: S-020700-WD-N-A

INCREASE RIDER TO SURETY BOND

PURPOSE: **INCREASE RIDER**

To be attached to Bond Number **853746** by **Evergreen National Indemnity Company**, as Surety in the amount of **Eleven Million, Ninety-Four Thousand, Nine Hundred Forty-Three and 00/100 Dollars ($11,094,943.00),** on behalf of **NEWSME Landfill Operation, LLC,** the Principal, in favor of the **State of Maine Department of Environmental Protection.**

In consideration of the premium charged for the attached bond, it is mutually understood and agreed by the Principal and the Surety that the bond shall be modified to read as follows: The above said bond amount shall be **Thirteen Million, Two Hundred Forty-Four Thousand, Two Hundred Forty-Eight and 00/100 Dollars ($13,244,248.00),** effective the 12th day of August, 2015.

All other items, limitations and conditions of said bond except as herein expressly modified shall remain unchanged.

Signed, sealed and dated this 4th day of August, 2015.

Principal: NEWSME Landfill Operation, LLC

By: ________________, Pres & Sec
John W. Casella

Surety: Evergreen National Indemnity Company

By: ________________
Patricia A. Temple, Attorney-In-Fact

EVERGREEN NATIONAL INDEMNITY COMPANY

MAYFIELD HEIGHTS, OH

POWER OF ATTORNEY

POWER NO. **853746**

KNOW ALL MEN BY THESE PRESENTS: That the Evergreen National Indemnity Company, a corporation in the State of Ohio does hereby nominate, constitute and appoint:

Patricia A. Temple

its true and lawful Attorney(s)-In-Fact to make, execute, attest, seal and deliver for and on its behalf, as Surety, and as its act and deed, where required, any and all bonds, undertakings, recognizances and written obligations in the nature thereof, PROVIDED, however, that the obligation of the Company under this Power of Attorney shall not exceed **Thirteen Million, Two Hundred Forty-Four Thousand, Two Hundred Forty-Eight and 00/100 Dollars ($13,244,248.00).**

This Power of Attorney is granted and is signed by facsimile pursuant to the following Resolution adopted by its Board of Directors on the 23rd day of July, 2004:

"RESOLVED, That any two officers of the Company have the authority to make, execute and deliver a Power of Attorney constituting as Attorney(s)-in-fact such persons, firms, or corporations as may be selected from time to time.
FURTHER RESOLVED, that the signatures of such officers and the Seal of the Company may be affixed to any such Power of Attorney or any certificate relating thereto by facsimile; and any such Power of Attorney or certificate bearing such facsimile signatures or facsimile seal shall be valid and binding upon the Company; and any such powers so executed and certified by facsimile signatures and facsimile seal shall be valid and binding upon the Company in the future with respect to any bond or undertaking to which it is attached."

IN WITNESS WHEREOF, the Evergreen National Indemnity Company has caused its corporate seal to be affixed hereunto, and these presents to be signed by its duly authorized officers this 1st day of December, 2014.

EVERGREEN NATIONAL INDEMNITY COMPANY

EVERGREEN NATIONAL INDEMNITY COMPANY CORPORATE SEAL 1939 OHIO

By: Matthew T. Tucker
Matthew T. Tucker, President

By: David A. Canzone
David A. Canzone, CFO

Notary Public)
State of Ohio) SS:

On this 1st day of December, 2014, before the subscriber, a Notary for the State of Ohio, duly commissioned and qualified, personally came Matthew T. Tucker and David A. Canzone of the Evergreen National Indemnity Company, to me personally known to be the individuals and officers described herein, and who executed the preceding instrument and acknowledged the execution of the same and being by me duly sworn, deposed and said that they are the officers of said Company aforesaid, and that the seal affixed to the preceding instrument is the Corporate Seal of said Company, and the said Corporate Seal and signatures as officers were duly affixed and subscribed to the said instrument by the authority and direction of said Corporation, and that the resolution of said Company, referred to in the preceding instrument, is now in force.

IN TESTIMONY WHEREOF, I have hereunto set my hand and affixed my official seal at Cleveland, Ohio, the day and year above written.

NOTARIAL SEAL STATE OF OHIO
PENNY M HAMM
NOTARY PUBLIC
STATE OF OHIO
Comm. Expires
April 04, 2017

Penny M. Hamm, Notary Public
My Commission Expires April 4, 2017

State of Ohio) SS:

I, the undersigned, Secretary of the Evergreen National Indemnity Company, a stock corporation of the State of Ohio, DO HEREBY CERTIFY that the foregoing Power of Attorney remains in full force and has not been revoked; and furthermore that the Resolution of the Board of Directors, set forth herein above, is now in force this 4th day of August, 2015.

EVERGREEN NATIONAL INDEMNITY COMPANY CORPORATE SEAL 1939 OHIO

Wan C. Collier, Secretary

Facility Name: Juniper Ridge Landfill – Post-Closure
Maine DEP Site No.: S-020700-WD-N-A

INCREASE RIDER TO SURETY BOND

PURPOSE: **INCREASE RIDER**

To be attached to Bond Number **853747** by **Evergreen National Indemnity Company**, as Surety in the amount of **Nine Million, Nine Hundred Seventy-Seven Thousand, Three Hundred and 00/100 Dollars ($9,977,300.00),** on behalf of **NEWSME Landfill Operation, LLC,** the Principal, in favor of the **State of Maine Department of Environmental Protection.**

In consideration of the premium charged for the attached bond, it is mutually understood and agreed by the Principal and the Surety that the bond shall be modified to read as follows: The above said bond amount shall be **Ten Million, Two Hundred Eighty Thousand, Three Hundred Ninety and 00/100 Dollars ($10,280,390.00),** effective the 12th day of August, 2015.

All other items, limitations and conditions of said bond except as herein expressly modified shall remain unchanged.

Signed, sealed and dated this 4th day of August, 2015.

Principal: NEWSME Landfill Operation, LLC

By: ____________ , Pres & Sec
John W. Casella

Surety: Evergreen National Indemnity Company

By: ____________
Patricia A. Temple, Attorney-In-Fact

EVERGREEN NATIONAL INDEMNITY COMPANY

MAYFIELD HEIGHTS, OH
POWER OF ATTORNEY

POWER NO. **853747**

KNOW ALL MEN BY THESE PRESENTS: That the Evergreen National Indemnity Company, a corporation in the State of Ohio does hereby nominate, constitute and appoint:

Patricia A. Temple

its true and lawful Attorney(s)-In-Fact to make, execute, attest, seal and deliver for and on its behalf, as Surety, and as its act and deed, where required, any and all bonds, undertakings, recognizances and written obligations in the nature thereof, PROVIDED, however, that the obligation of the Company under this Power of Attorney shall not exceed **Ten Million, Two Hundred Eighty Thousand, Three Hundred Ninety and 00/100 Dollars ($10,280,390.00).**

This Power of Attorney is granted and is signed by facsimile pursuant to the following Resolution adopted by its Board of Directors on the 23rd day of July, 2004:

"RESOLVED, That any two officers of the Company have the authority to make, execute and deliver a Power of Attorney constituting as Attorney(s)-in-fact such persons, firms, or corporations as may be selected from time to time.
FURTHER RESOLVED, that the signatures of such officers and the Seal of the Company may be affixed to any such Power of Attorney or any certificate relating thereto by facsimile; and any such Power of Attorney or certificate bearing such facsimile signatures or facsimile seal shall be valid and binding upon the Company; and any such powers so executed and certified by facsimile signatures and facsimile seal shall be valid and binding upon the Company in the future with respect to any bond or undertaking to which it is attached."

IN WITNESS WHEREOF, the Evergreen National Indemnity Company has caused its corporate seal to be affixed hereunto, and these presents to be signed by its duly authorized officers this 1st day of December, 2014.

EVERGREEN NATIONAL INDEMNITY COMPANY

EVERGREEN NATIONAL INDEMNITY COMPANY CORPORATE SEAL 1939 OHIO

By: Matthew T. Tucker
Matthew T. Tucker , President

By: David A. Canzone
David A. Canzone, CFO

Notary Public)
State of Ohio) SS:

On this 1st day of December, 2014, before the subscriber, a Notary for the State of Ohio, duly commissioned and qualified, personally came Matthew T. Tucker and David A. Canzone of the Evergreen National Indemnity Company, to me personally known to be the individuals and officers described herein, and who executed the preceding instrument and acknowledged the execution of the same and being by me duly sworn, deposed and said that they are the officers of said Company aforesaid, and that the seal affixed to the preceding instrument is the Corporate Seal of said Company, and the said Corporate Seal and signatures as officers were duly affixed and subscribed to the said instrument by the authority and direction of said Corporation, and that the resolution of said Company, referred to in the preceding instrument, is now in force.

IN TESTIMONY WHEREOF, I have hereunto set my hand and affixed my official seal at Cleveland, Ohio, the day and year above written.

NOTARIAL SEAL STATE OF OHIO
PENNY M HAMM
NOTARY PUBLIC
STATE OF OHIO
Comm. Expires
April 04, 2017

Penny M. Hamm
Penny M. Hamm, Notary Public
My Commission Expires April 4, 2017

State of Ohio) SS:

I, the undersigned, Secretary of the Evergreen National Indemnity Company, a stock corporation of the State of Ohio, DO HEREBY CERTIFY that the foregoing Power of Attorney remains in full force and has not been revoked; and furthermore that the Resolution of the Board of Directors, set forth herein above, is now in force this 4th day of August, 2015.

EVERGREEN NATIONAL INDEMNITY COMPANY CORPORATE SEAL 1939 OHIO

Wan Collier
Wan C. Collier, Secretary

ATTACHMENT 2

CASELLA COMPANIES AUTHORIZED TO DO BUSINESS IN MAINE

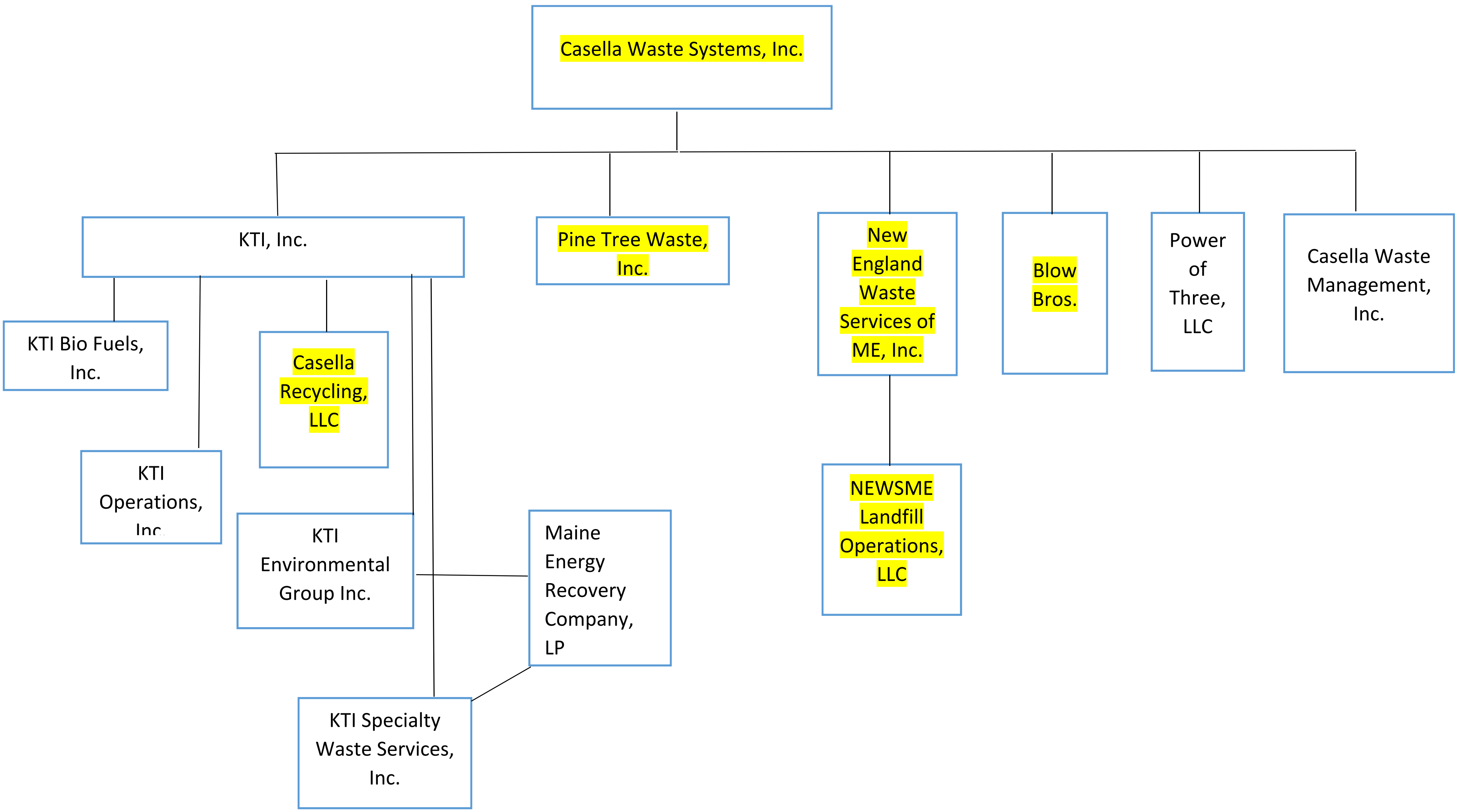

Casella Waste Systems, Inc.
KTI, Inc.
Pine Tree Waste, Inc.
New England Waste Services of ME, Inc.
Blow Bros.
Power of Three, LLC
Casella Waste Management, Inc.
KTI Bio Fuels, Inc.
Casella Recycling, LLC
KTI Operations, Inc.
KTI Environmental Group Inc.
Maine Energy Recovery Company, LP
KTI Specialty Waste Services, Inc.
NEWSME Landfill Operations, LLC

EXHIBIT B

BGS AND NEWSME'S RESPONSE TO DEP'S JANUARY 15, 2016 TECHNICAL REVIEW MEMORANDUM

Below BGS and NEWSME set forth each of Staff's comments in the January 15, 2016 memorandum and follow each comment with our response.

<u>Pg 1, Par. 2</u>:[1] **Overall JRL's expansion application is well organized and documented. Based upon my review of the information presented in the expansion application, nearly all of the requirements of the Solid Waste Regulations have been satisfactorily addressed. I do, however, have a wide variety of comments and recommendations that will need to be addressed. The detailed memorandum that follows outlines my comments and recommendations."**

> <u>Response</u>: In our discussions with DEP at a meeting held on January 29, 2016, we discussed this comment with Mr. Behr. It is our understanding that Mr. Behr believes the Expansion application adequately addresses each of the Rule's hydrogeologic criteria, but he wants SME to provide the additional clarifying information as requested in his memorandum.

<u>VOLUME I – Maine Solid Waste Management Rules</u>

<u>Pg 3-28, 3.12 Adequate Provision for Utilities and No Unreasonable Adverse Effect on Existing or Proposed Utilities</u>. I understand there are two existing water supplies (Scale House Well and Facility Well) on site but these wells are not shown on many of the relevant site plans. Both wells are located within the expansion footprint and will have to be abandoned and replaced if the facility expands. Therefore, the application should include details about abandonment of these wells and information about where the replacement wells may be located. In the meantime, JRL's Environmental Monitoring Program should be revised to include plans to sample both wells annually to characterize water quality. The well locations should also be shown on all the relevant site plans.

> <u>Response</u>: The location of the two referenced wells (i.e., scale house well and office facility well), and a well that serves the landfill maintenance building on the eastern side of the site have been added to the Site Surrounding Map included in Volume I, Appendix M. The updated map is included in Attachment SME-1. The scale house and office facility wells will be abandoned prior to the construction of Cells 12 and 13, respectively. A new water supply well will be drilled in the vicinity of the relocated Scales and Administrative Building (see Site Surrounding Map). The existing wells will be abandoned by pulling the casings and grouting each well's borehole in general accordance with the techniques identified in specification 1520 of the project

[1] This is the only page reference that relates to the pagination of Mr. Behr's memo. The other pages referenced in this response refer to those pages from the expansion application upon which Mr. Behr had specific comments in his January 15, 2016 memorandum.

specifications included in Volume III, Appendix A of the Application. The environmental monitoring program will be modified to include sampling of these water supply wells.

VOLUME II – Site Assessment Report

Pg 2-6, 2.6.1 Surficial Soils. The description of the surficial geology notes that the Maine Geological Survey's mapping suggests some of the elongated hills are glacial drumlins. The available LIDAR imagery may provide further evidence of the existence of glacial drumlins in the vicinity of the landfill. I have attached a LIDAR image that appears to depict linear features that may be interpreted to be drumlins (DEP – Figure 2). It is also possible to see the boundary between the Presumpscot formation and the till deposits as well as some of the bedrock outcrops located along the western edge of the proposed expansion. I urge JRL to include this information in the section describing the regional geologic setting.

Response: We have reviewed the LiDAR imagery of the Expansion site and surrounding region. The imagery supports the interpretation that the hill on which the JRL is positioned is a drumlin. There are numerous other glacial streamforms or drumlin-like features apparent in the imagery with their long-axes oriented towards the south-southeast (i.e., direction of ice sheet movement). Some of the streamforms appear to be associated with shallow bedrock based on the imagery. From the imagery, surficial soils over much of the area surrounding the landfill can be interpreted as glacial till based on the topography and presence of these streamform features. The imagery confirms shallow bedrock outside the west side of the Expansion. The bedrock appears to be shallow beneath the hills west of the Expansion, as well. There is a northeast-southwest textural pattern in some areas of shallow bedrock. This pattern is consistent with the principal bedrock fracture set identified beneath the Expansion site and infers the regional nature of this fracture set. The principal fracture set is associated with foliation of the clay minerals of the phyllite. The imagery also confirms the sandy glacial outwash deposit mapped east of the site along Route 16. The esker associated with this outwash deposit can also be identified on the LiDAR imagery east of Route 16.

Pg 2-10, 2.6.2 Bedrock. The report states JRL obtained fracture orientation data from three of the four outcrops identified in the vicinity of the facility. Apparently fractures visible on OC-4 could not be measured. If measurements could not be obtained from OC-4, the text appearing on the following page should not indicate measurements were collected from all four outcrops.

Response: As shown on Table 2-1, relic bedding data was measured at OC-4. As stated in the text, there were fractures associated with this bedding and these fractures were used to measure for strike and estimate the dip of the bedding. The difficulty of measuring fracture dip at OC-4 was associated with the flat nature of the outcrop, resulting in fracture faces that were too small to accommodate a Brunton compass.

However, a ruler was used to visually extend the dip angle above the outcrop and this inferred dip angle was estimated using a Brunton compass.

<u>Pg 2-16, 2.9 Local Groundwater Resources</u>. This section includes data gathered by the Maine Geological Survey (MGS) about drilled wells in the neighborhood of the landfill. The MGS information is useful but it should be augmented with information JRL gathered when they sampled numerous residential wells along the West Old Town and Old Stagecoach Roads. JRL completed this sampling in 2004.

<u>Response</u>: We have revised and attached Figure 2-8 (Attachment SME-1) to show wells sampled by NEWSME in 2004 in response to DEP's request at that time to examine groundwater quality near the landfill. The sampling provided a background "snapshot" of the groundwater quality at the residences sampled. The wells were identified in cooperation with DEP as those closest to the JRL. The wells are located along Route 43 and Old Stagecoach Road and on Route 16. Water samples were taken from taps within each residence and analyzed for a suite of parameters, including field parameters (pH, conductivity and temperature), as well as laboratory parameters (arsenic, calcium, iron, lead, magnesium, manganese, potassium, sodium, total organic carbon, bicarbonate, nitrate, chloride, sulfate and volatile organic compounds). This data was supplied to the property owners and the DEP on July 27, 2004.

<u>Pg 3-17, 3.2.6 Groundwater Tracer Test in Glacial Till</u>. To provide additional data about groundwater velocities in the till, JRL conducted a tracer test using sodium bromide. I have reviewed the details of the test contained in Appendix G. The analytical solutions produced an estimated velocity of 11 ft/year. Interestingly, the estimated velocity based on the arrival of the peak bromide concentration (i.e., graphic solution) yields a slightly higher velocity of 17 ft/year. I too analyzed the data graphically (DEP Figure 3) and calculated a velocity of 15.5 ft/year.

It seems to me the graphically derived solution may be more representative of the in-situ velocity. Particularly since the well containing the highest bromide concentrations is likely not directly downgradient of the injection well. Perhaps more importantly, this test was not conducted within the proposed expansion area. I recognize the till in and around the proposed expansion may be relatively uniform, but ideally I would expect tests like this would be performed within the footprint or directly downgradient. JRL should, to the extent possible, explain why the results of a tracer test conducted several hundred feet from the expansion are representative of site conditions beneath the proposed expansion.

<u>Response</u>: This test was positioned within the footprint of a larger expansion area that was originally proposed and considered when DEP approved the Applicants' Preliminary Information Report in 2006. The glacial till, as illustrated by the grain size curves in Appendix O, is relatively consistent throughout the drumlin. Because of the relative uniformity of the till, the tracer test has general applicability in terms of the spreading behavior of the tracer. The velocity of groundwater will vary somewhat throughout the

till, depending on the till's local texture, compactness, and groundwater gradient. However, the value of the tracer velocity lies in its confirmation of the relatively low groundwater velocity calculated for the till elsewhere throughout the drumlin based on slug testing. It generally confirms the geometric mean hydraulic conductivity estimated for the till deposit. Therefore, although any tracer test location will differ slightly from others in the till, the nearby test provides useful corroborative data for estimating groundwater velocities within the till deposit of the drumlin.

SME used an analytical solution to the three-dimensional solute transport equation to estimate the groundwater velocity from the bromide data collected at the downgradient observation wells. The single velocity that provided a best-fit for all the bromide curves, simultaneously at all the downgradient wells, was selected as representative of the average groundwater velocity during the 477-day test. The other velocities estimated by Mr. Behr confirm the equation's estimate as being reasonable.

<u>Pg 3-18, 3.2.7 Groundwater Tracer Test in Bedrock</u>. The details of this test are provided in Appendix H. I provided detailed comments about this tracer test in an October 15, 2008 review memorandum.[2] Although I do not have record of a written response from JRL, review of the report included in Appendix H appears to address several of the concerns outlined in my memorandum.

My primary concern with the results of the tracer test was the failure to detect bromide at significant levels (i.e., > 1% of the injection fluid concentration) in any of the six downgradient observation wells. I agree with JRL that the detection of bromide in each of the six observation wells verifies the existence of an interconnected fracture network. However, my interpretation of the analytical results, based on discussions with my colleagues in the Department, lead me to conclude the majority of the tracer passed beneath the observation wells. Calculations supporting this interpretation (DEP Attachment A) are discussed later in this memorandum. I understand that JRL has revised its earlier interpretation and now believes the density of the introduced tracer induced a significant downward vertical flow of the introduced tracer. Regardless of the fate of the majority of the introduced tracer, I agree the tracer test data has produced a reasonable range of estimated groundwater flow velocities. However, uncertainty regarding the trajectory of the tracer demonstrates why multilevel wells are necessary to increase the likelihood of intercepting leachate constituents that may pass through the liner system.

Additional comments related to this test are found following the Appendix H heading.

<u>Response</u>: The bedrock tracer test was conceived as a means to corroborate earlier conclusions that the bedrock fractures were, in general, well interconnected. Previous

[2] Technical Review Memorandum from Richard Behr to Cyndi Darling. October 15, 2008, Bedrock Tracer Test at Proposed Juniper Ridge Landfill Expansion Site, Old Town, Maine – NEWSME Landfill Operations, LLC. September 2008, Prepared by Sevee & Maher, Inc.

data collected from bedrock outcrop mapping, bedrock core samples and Maine Geological Survey mapping showed that the bedrock was commonly fractured. The data showed fracture spacing of less than a foot in most areas, fracture lengths typically greater than the fracture spacing, and fractures oriented in virtually all azimuths and dips (although there were two prominent fracture sets). These conditions were identified for the existing landfill back in the early 1990s, the proposed Expansion area, and the areas surrounding the existing landfill and proposed Expansion. These repetitive findings, along with our experience with similar bedrock at other sites, led us to conclude the bedrock fractures were well interconnected. A simple tracer test (as well as a pumping test) was a useful and efficient way of testing this conclusion. The test was planned to provide a qualitative indication of whether the fracture system in a localized area of "typical" site bedrock would result in a spreading of the tracer from the injection point and whether it could be found at several observation points surrounding the downgradient side of the injection point. If one well only or two non-adjacent observations wells recorded the tracer, there may have been some question as to the interconnectedness conclusion. If a tracer pattern was detected in the observation wells over a broad downgradient area, then the fracture interconnectedness was confirmed. This was our goal, to test the bedrock in a qualitative manner. If the data from the test could be analyzed by common methods to estimate a groundwater velocity that would potentially corroborate velocities calculated from slug test data, that would be beneficial, but was not the primary goal. The originally conceived purpose of the test was to see if tracer spreading occurred, confirming the previous conclusions about the fracture interconnectedness. Appendix U explains our rationale for considering the bedrock fracture system as being generally well interconnected.

As indicated in our analysis of the bedrock tracer, we concur with Mr. Behr that the centroid of the tracer mass migrated downward under the influence of both the local groundwater gradient and density of the tracer. This interpretation was noted in Appendix H. However, this behavior does not invalidate the results that were collected: even though the majority of the bromide mass moved downward, the tracer was able to spread out enough to be detectable horizontally away from the tracer injection point. This is a result of the well-integrated fracture system of the bedrock. This fracture system is ubiquitous at the Expansion site and surrounding area based on outcrop mapping, down-hole geophysical fracture surveys, and bedrock cores. The spreading of the bromide plume over an angle of at least 90 degrees is a function of the fracture integration.

The value of this test, in our opinion, is associated with the observed spreading of the tracer, which began at a two-inch diameter well. By the time the tracer reached the observation wells at a distance of about 50 feet, it had spread from this two-inch diameter well over a lateral distance of about 100 feet under natural, ambient gradients. In addition, this was for a tracer that was migrating partially downward yet still could be observed horizontally from the tracer injection point. This spreading is encouraging for the design of a monitoring network that could detect the unlikely event of landfill liner leakage. If there had been little spreading, that would have suggested a need for closer monitoring well spacing.

As an aside, the location of the bedrock tracer test was positioned within the footprint of the original 106-acre expansion plan. The location was selected based on two criteria: first, a location away from the top of the drumlin was sought, where groundwater seepage would be horizontal (e.g., on the flank of the hill) and, second, to set the test up in an area of typically fractured bedrock. The down-hole geophysical logging suggested multiple fractures intersecting the injection well. Bedrock cores confirmed similar conditions at the observation wells. Thus, the second criterion was met. The test location selected was thought to have predominantly horizontally moving groundwater based on constructing a groundwater flow-net using groundwater level data from surrounding wells and piezometers. From the tracer data, it appears that the selected location may have a partially downward groundwater seepage gradient, which limited the amount of tracer moving horizontally at the observation well depths. However, there was enough horizontal tracer movement to register in the observation wells surrounding the injection well, and thus the first criterion was met, as well. The primary objective of the test was, therefore, accomplished: to see whether the tracer spread out, confirming a well interconnected fracture system.

DEP recommended collecting additional bedrock information to finalize/refine the design of a perimeter groundwater monitoring system for the Expansion. We recognize that localized fracture zones may control groundwater moving away from the landfill and, therefore, agree with DEP to collect additional data to check for localized fracture zones that may locally control groundwater movement around the Expansion in the site's bedrock. Such localized fracture zones were observed in the earth resistivity surveys previously conducted at the site. This issue is best addressed through refined delineation of major fracture systems using subsurface exploration techniques similar to those previously used, prior to monitoring well installation, and is discussed elsewhere in our responses.

<u>Pg 3-18, 3.2.8 Groundwater Age-Dating</u>. JRL used the tritium-helium groundwater age-dating methodology to estimate the age of two groundwater samples. Results from these tests may provide invaluable information if one accurately estimates the age of groundwater at multiple locations along a groundwater flow path. The difference in the estimated ages divided by the distance yields an average groundwater velocity between the two sample points. This approach provides an estimate of groundwater velocity independent of the aquifer characteristic data commonly used to estimate groundwater velocity. In this case, it may provide an independent estimate of groundwater velocity in bedrock. The calculated groundwater velocity between P-04-06A and P-04-07B was 140 feet per year. This estimated bedrock groundwater velocity (140 ft/year) is significantly lower than the velocities used in the time of travel calculations. It is important for JRL to explain why they used significantly faster bedrock velocities in the time of travel calculations.

JRL also used the age of the groundwater sample collected at P-04-06A (14 years) to estimate the travel time through the till to the shallow bedrock. Assuming a downward vertical flow path through roughly 29 feet of till, the apparent travel time significantly exceeds six years. Based on the estimated age (14 years) and distanced travelled, the groundwater velocity is about 2 ft/year. JRL states the seepage gradients were determined to be vertical but it is not clear how they made this determination. Potentiometric head data from the two wells does indicate the potential for a downward vertical flow. It does not, however, demonstrate groundwater follows a vertical flow path through the till. In fact, while I don't dispute a vertical downgradient exists in the vicinity of P-04-6A, it is unlikely the flow path is straight down.

With this uncertainty in mind, I recommend JRL calculate a range of estimated groundwater velocities based on alternate flow paths leading to the screened interval of P-04-06A.

It is also necessary for JRL to improve this section by including a brief discussion of the tritium-helium age-dating methodology. It would also be helpful if JRL included information regarding its prior use at other Maine sites. This section should also include appropriate peer reviewed technical references. Most importantly, my concerns regarding the validity of the results, as detailed below (Appendix I comments), must be addressed to the Department's satisfaction.

> Response: The groundwater velocities used in our travel-time calculations were estimated on the conservative side. That is, the velocities were biased towards higher velocities resulting in faster arrival times. The time-of-travel calculations assumed only horizontal flow in the bedrock. Not accounting for the vertical travel time effectively shortens the calculated times; therefore, the calculations under-estimate the travel-times and are conservative. In the contaminant transport analysis in Section 4 of Volume III of the Application, a similar assumption of only a horizontal flow path was applied and the velocity was assumed at 5 feet/day. Even with these conservative assumptions, the requirements of Chapter 401(1) (C) (c) and (d) (travel time and risk to sensitive receptors) were met.
>
> A location for age-dating of groundwater was sought to estimate the vertical travel-time through the glacial till. Based on groundwater levels measured in wells and piezometers, groundwater flow-nets were constructed to estimate where groundwater seepage would be vertical or nearly vertical. As can be seen on Figure 5-2, Profile C-C,' the equipotential contours at P-04-06A and -B are nearly horizontal except for the more weathered, permeable till at the ground surface. This is why this location was selected for age-dating the groundwater at two depths across the till. The assumed seepage pathway through the till can be adjusted to remain more perpendicular to the interpreted equipotential contours. This would lengthen the flow path through the till by possibly 20 to 40 percent. The longer flow path results in a 20 to 40 percent increase in the estimated groundwater velocity. Assuming the hydraulic conductivity of the upper weathered five or so feet of till is likely somewhat more permeable than the

unweathered, deeper till, the calculated average vertical hydraulic conductivity through the till ranges from the previously calculated 1.3 x 10^{-6} cm/sec up to 1.8 x 10^{-6} cm/sec (see Page 5-10 of the Site Assessment Report). This is a relatively small change given the natural range of hydraulic conductivities measured for the till. This result does not affect any of our travel-time calculation results or conclusions.

SME has applied the tritium-helium age-dating methods on numerous sites inside and outside Maine over the past twenty years. It has been used to estimate the rate of groundwater travel, to examine aquifer vulnerability to surface contamination, to determine potential sources of groundwater contamination to water supply wells, and to estimate if a solvent groundwater plume is still expanding or near steady-state. It has proven to be a useful tool when used along with the other investigatory techniques.

The tritium-helium age-dating method is a relatively simple method to collect data (Clark, I. D. and P. Fritz, 1997. Environmental Isotopes of Hydrogeology, Lewis Publishers; Aeschbach-Hertig, W., Groundwater Sampling for Helium/Noble Gases Using Copper Tubing, Institute of Environmental Physics, University of Heidelberg, Germany). A liter sample of groundwater is collected in a plastic bottle for the tritium analysis. A 10 to 40 milliliter sample of groundwater is collected in a copper tube, being careful to continually tap the tube to remove air bubbles. Once the air bubbles have been completely removed from the tube, each end is sealed by pinching the copper. This tube sample is used to measure inert gases in the sample. The tritium is measured by the in-growth method, wherein all gases are removed from a specimen of the groundwater, the specimen is sealed and allowed to sit for two to three months as the tritium in the specimen decays to helium-3. The amount of helium-3 in the specimen is used to determine the tritium content of the groundwater at the time of sampling. The inert gases are measured by mass spectrometer from a specimen of groundwater taken from the copper tube. Some of the inert gases are used to estimate the precipitation recharge temperature of the specimen and others are used to estimate specimen total helium-3. The results are used to correct the helium-3 for excess air, atmospheric helium-3 and terragenic helium after which the tritium and corrected helium-3 concentrations are used to calculate groundwater age. By examining the various gas components, an evaluation of the utility and accuracy of the results can be made. The results of the analysis at the Expansion site proved to be useful but there is still a slight variability that must be recognized in applying the results (R. Poreda, 2002 through 2014, personal communications; USGS, The Reston Groundwater Dating Laboratory, Reston Virginia).

We used the tritium-helium age-dating method at the JRL Expansion site to corroborate groundwater velocities determined using the slug test data. The groundwater velocity is calculated from groundwater seepage gradients, hydraulic conductivities and effective porosity. Groundwater gradients are determined using wells and piezometers and can be calculated relatively precisely. Hydraulic conductivity of some soil and rock can range over several orders of magnitude and is typically resolved into a geometric mean or average hydraulic conductivity of the representative geologic formation. Effective porosities of fine grained soils and bedrock can be difficult to estimate. Therefore, estimating the groundwater velocity using the age-dating method described above, provides a check on all three inputs to the groundwater velocity calculation and provides confidence in the calculations.

<u>D 3-19, 3.2.9 Bedrock Pumping Test at MW-06-02</u>. JRL performed a short term pumping test in MW-06-02 roughly two years before conducting the tracer test in the same well. This well is located nearly 700 feet north of the northern edge of the proposed landfill expansion boundary. Unfortunately, JRL initiated the test before conducting a step draw down test to determine a sustainable pumping rate. Consequently, without prior knowledge about the well's sustainable yield, the initial pumping rate of 3.5 gpm turned out to be far too high and resulted in periodic adjustments throughout the test. The estimated average pumping rate during the eight hour pumping test was 0.19 gpm. Despite this misstep, it appears the pumping test produced some useful information about the nature of the bedrock aquifer in the vicinity of the proposed expansion.

Appendix J provides a detailed description and analysis of the resulting data. I have reviewed the data contained in Appendix J along with the data interpretation. Additional comments related to this test are found below following the Appendix J heading. I also asked a colleague, Gail Lipfert, to review and comment on both the pumping test and tracer test. I have attached Gail's comments with the expectations JRL will address them as well (DEP – Attachment B).

<u>Response</u>: We agree that the test yielded useful information that has been used to characterize the bedrock hydrogeology for the JRL site. We address DEP comments on Appendix J and DEP- Attachment B at the end of this response. Also see our response to Appendix B (i.e., G. Lipfert's comments on the pumping test) of Mr. Behr's memorandum at the end of this response letter.

<u>Pg 3-20, 3.2.10 Photolineament Survey</u>. JRL should also consider using the LIDAR imagery to identify photolineaments. This imagery is available through the Maine Office of GIS.

<u>Response</u>: Figure 3-2 has been updated with nine new photolineaments, based on SME's interpretation of the LiDAR image. None are within the Expansion or existing landfill footprint. These new photolineaments reinforce but, do not change any of our conclusions or the design of the Expansion. The updated Figure 3-2 is included in Attachment SME-1.

<u>Pg 3-21, 3.2.11 Bedrock Outcrop Survey</u>. JRL collected fracture orientation data from five outcrops surrounding the facility. One vertically orientated outcrop (OC-AG) was selected for detailed mapping. The data from the detailed analysis are summarized in this section and the tabulated strike and dip data are found in Appendix K. On a technical note, I found the total measurements tabulated in Table K-1 (68) differ significantly from the summary (81) included in Appendix U (Bedrock Fracture Interconnectivity).

This mapping effort produced some important information about the bedrock at this site. First, the outcrop selected for the detailed mapping, although relatively small, contained a large number of closely spaced fractures. Second, JRL found all the fractures on the outcrop are connected to one another.

The mapping summary did not discuss the degree to which the data from this outcrop is or is not representative of the general site conditions. For example, how does the fracture spacing observed on the outcrop compare to the fracture spacing in the four deep bedrock boreholes? Was the OC-AG outcrop similar to the other four outcrops? Although JRL did not complete detailed mapping of the remaining four outcrops, a careful visual inspection coupled with photographs may allow for a valid comparison.

Response: DEP noted that the number of fractures for OC-AG differs in Table K-1 and Figure U-3. For clarification, Table K-1 includes observations for 68 fractures that were identified within a discrete 6-foot by 4-foot area of the outcrop. These observations included the number of intersecting fractures for each identified fracture, and azimuth and dip measurements where planar measurements were possible (as indicated on Note 1 of Table K-1). Only a small portion of the 68 fractures within the 6-foot by 4-foot area of the outcrop chosen for observing fracture interconnectivity include azimuth and dip measurements because most of the exposed features were linear. The OC-AG outcrop is much larger than the 6 foot by 4 foot area chosen for observing fracture interconnectivity, and many azimuth and dip measurements were made across the entire outcrop exposure. Therefore, as is explained on paragraph 3 of page 5 of Appendix U, Figure U-3 includes "*bedrock fracture orientations from the 6-foot by 4-foot area of outcrop OC-AG and supplemental fracture orientations located elsewhere on an expanded area of the same outcrop*."

Outcrop OC-AG was useful for mapping of fractures because of its size and orientation. However, all five outcrops showed the same northeast-southwest mineral foliation and associated fractures. The nearly orthogonal secondary set of fractures was also apparent at all outcrops except OC-4, which was of limited size. OC-AG was mapped specifically for the Expansion investigations. The other four outcrops were measured in 1991 for the original JRL application. Average fracture spacing on OC-AG was in the order of a few tenths of a foot. In the boreholes, most fracture spacing was less than a few feet. Visually, the rock core fractures and the outcrops' fractures are similar in appearance and spacing, in a general sense. The outcrops mapped in 1991 were not mapped at the same level of detail as OC-AG due to their size and orientations. Although multiple fractures were observed as noted in Table 2-1, spacing measurements were not made. However, the corroboration between the fracture densities at OC-AG, rock cores and the geophysical survey support the conclusion that the site is relatively uniform with respect to fracture orientation, fracture density and spacing, and lithology. The data collected shows one bedrock unit at all locations where bedrock was exposed, composed of metagraywacke and phyllite with bands of siltstone and sandstone. All exposures show foliation of the clay minerals and all locations showed multiple fractures.

EXHIBIT B

<u>Pg 3-24, 3.2.13 Fracture Interconnectivity Pumping Test</u>. In addition to the previously discussed pumping test, JRL conducted five pumping tests in the four 200-foot open bedrock wells installed by Goodwin Well & Water, Inc. The results from these pumping tests generated invaluable information about the characteristics of the fractured bedrock underlying and adjacent to the proposed expansion.

This section includes a brief description of the results of all of the pumping tests. Appendix M includes additional details and discussion of the long-term pumping test conducted on PW-08-01 and PW-08-02. The long-term pumping test began by pumping PW-08-01 for about a week. At the beginning of the second week, JRL continued to pump PW-08-01 but also began pumping PW-08-02. It would be helpful if the report described the rationale for the dual well pumping tests. Specifically, the report should outline what additional qualitative and quantitative aquifer characteristic data were obtained from the combined test.

The report could be improved by providing the details about how each test was instrumented. For example, the report should identify all the wells where JRL measured hydraulic head using pressure transducers and the wells where manual water level measurements were made. I have not been able to locate the table(s) summarizing all of the manual measurements. I will need this information to complete my data analysis.

I also recommend the pumping test discussion in Appendix M be expanded to include an analysis of the four short-term pumping tests conducted prior to the long-term test. A detailed discussion of each pumping test should include all of the relevant data. For example, Appendix U (Bedrock Fracture Interconnectivity) states that during the 24 hour pumping test conducted at PW-04-01, JRL collected water level information at 24 bedrock wells and 25 till wells. The summary reports the range of drawdowns observed in the bedrock and till wells but I have not located the summary tables. Further, Appendix U appears to include a more detailed summary of the four short term pumping tests than what JRL presents in this section. Revisions to the application must address these issues.

> <u>Response</u>: The primary purpose for running the combined well pump test was to examine control of groundwater collection at the Expansion if pumping of the bedrock should become necessary in the future in the unlikely event of a landfill liner leak. Although the individual pump tests on the deep boreholes demonstrated the ability to control groundwater flow in the bedrock, we wanted to examine if the drawdown behavior was linear with more pumping, or if the wells ran dry. The results showed that the drawdown behavior was more or less linear and no dewatering of the wells occurred. The zone of influence under pumping both wells simultaneously was similar to adding drawdowns from pumping the wells individually. The linear behavior provided more confidence that the bedrock behavior could be analyzed using common modeling or analytical methods. Overall, the pumping tests demonstrated that pumping from the bedrock would be an effective way to control groundwater flow in the bedrock, if

necessary. The pumping also corroborated the general interpretation that bedrock fractures are interconnected across the Expansion site.

Another objective of pumping both wells together was to evaluate recharge from the till. It was apparent that this recharge is capable of stabilizing drawdowns in the underlying bedrock, at least over the term of the combined pump test. This is consistent with the conceptual hydrogeologic setting discussed throughout the Site Assessment Report, that the till and underlying bedrock are hydraulically interconnected.

SME did not analyze each test for transmissivities and storage coefficients since that was not the purpose of the pump testing. However, such analysis was done at two pump tests on the Site (MW-06-01 and PW-08-01) to see if the results of the analyses corroborated the slug test data and interpretation of the principal directions of horizontal transmissivity or permeability. Appendices J and M discuss these detailed analyses of each of these two pump tests.

The principal purpose of pumping the deep boreholes was to examine the horizontal extent and distribution of drawdowns to confirm fracture interconnectivity that is suggested by other data collected on the bedrock fracture system at the Site (see Appendix U). This analysis is represented by Figures U-14 and U-15. These figures show that drawdowns occur in all directions away from the pumping wells. The figures also show that drawdowns can be observed as far away as a couple of thousand feet from the pumping wells. These observations suggest a well-integrated bedrock fracture system. Thus, monitoring well placement at the site is less critical, provided they are properly spaced, since groundwater moving away from the Expansion can, with some confidence, be predicted to follow the water table slope. DEP has raised the issue of the larger-scale bedrock heterogeneities in monitoring groundwater around the Expansion. This will be addressed prior to installation of any monitoring wells based on the findings of the work plan to refine the locations of monitoring wells (see Attachment SME-2).

In response to the question on the manual water level measurements, they are in the Application, in Appendix M, behind the transducer drawdown plots. In additional a table has been prepared that identifies all the wells where hydraulic heads was measured using pressure transducers, and the wells where manual water level measurements were made. This table is in Attachment SME-3.

<u>Pg 3-29, PW-08-01 and PW-08-02 (Combined) Long-Term Pumping Test</u>. During the two week pumping test, precipitation totaled 1.15 inches. JRL believes recharge occurred due to the snowmelt and precipitation. Given the reported slow rate of groundwater movement through the till, I believe it is important for JRL to explain why potentiometric head levels may rise relatively rapidly in response to precipitation events. A similar explanation should be provided for the rebound in water levels observed in the observation wells during the MW-06-02 pumping test.

Response: The rapid water level response to precipitation is due to the weight of the precipitation entering the soil. The precipitation is a structural loading similar to the weight of soil fill, a building or a vehicle. This is similar to when a barometric change occurs in the atmosphere, the pore water pressures change in response to any variation in barometric pressure. This phenomenon is well understood and documented (Walton, W.C. 1970. Groundwater Resources Evaluation, McGraw-Hill Inc.; Anochikwa, C.I., G. van der Kamp, and S.L. Barbour, 2012. Interpreting Pore-Water Pressure Changes Induced by Water Table Fluctuations and Mechanical Loading Due to Soil Moisture Changes, Canadian Geotechnical Journal). In the case of the pumping test at MW-06-02, the rise in the water levels in observation wells and the pumping well is mostly due to adjustments of the pumping rate during the latter stage of the test. At the point where the pump rate was being reduced, sufficient data had been collected to interpret transmissivities and storage coefficients, and the test was being shut down.

Pg 3-30. Not surprisingly, JRL observed declining pumping rates (gpm) during these tests. The pumping rates are expected to decrease as the head on the pump decreases not "increases" as stated in the report.

Did JRL also analyze the recovery data collected during each of the five pumping tests? If not, please explain why the recovery data wasn't also examined.

Response: The report is correct as written. The decreasing pump rate is due to the head on the pump "increasing" since the pump has to work harder and pump capacity decreases. This is common with such pumps and is referred to as the "pump curve."

The water level recovery data was not analyzed except to observe where the level returned to. In essentially all cases the water level fully recovered suggesting recharge to the bedrock system. For the two tests where transmissivities and storage coefficients were calculated, the recovery data adds little to the drawdown analysis. Again, the principal purpose of all pump tests was to qualitatively evaluate fracture interconnectivity.

Pg 3-37, 3.3.6 Effective Porosity. Effective porosity data are needed to estimate groundwater velocity in the till, marine clay and bedrock. JRL conducted laboratory tracer tests to estimate the effective porosity of the basal till. Presumably the procedure is described in Appendix R. I have reviewed Appendix R and find that it provides insufficient information to properly document the experimental procedure used to estimate the porosity. It appears that the estimated effective porosity is based on a single experiment. If so, JRL must justify how a single measurement can be used to adequately describe the entire site.

Response: The reason why an effective porosity measurement was conducted on the till is because of its fine grained texture. Some clayey soils exhibit a significantly lower effective porosity than total porosity due to their clay content. This is due to the "double-layer" effect associated with clay particles. Because a lower effective porosity results in a greater seepage velocity, SME wanted to see if the clay content of the till was reducing the till's total porosity for seepage calculations. Only one test was run because the results showed no impact and total porosity could be used to estimate the effective porosity for the till. The test methodology can be found in Attachment SME-3.

Pg 4-4, 4.1 Surficial Geology. Figure 4-2, the isopach map, depicts the thickness of surficial sediments within and beyond the proposed expansion. As we previously discussed with JRL, the accuracy of this map could be improved if additional bedrock explorations are completed within the proposed expansion. I am particularly concerned about the relative absence of bedrock explorations within the eastern half of the proposed expansion. On DEP Figure 4, I have depicted all of the bedrock explorations within and surrounding the proposed expansion. I understand the soil depths depicted on Figure 4-2 are based on a variety of data sources, including the modelled vertical resistivity profiles. To that end, JRL should augment this section with additional information about how the resistivity data was interpreted to refine the isopach map. This discussion could also include a discussion about how soil depths derived from the resistivity surveys compared to data obtained from explorations that penetrated the underlying bedrock.

Response: There are 35 soil borings and test pits within the Expansion footprint and another 7 along the edge of the Expansion. There are two bedrock outcrops outside the western edge of the Expansion. In addition, there is several thousand lineal feet of resistivity profiling within and along the edges of the Expansion area. Existing borings were used to calibrate the resistivity profiles in terms of soil thickness. Even with calibration, it is common practice to estimate the error for the bedrock elevation from electrical resistivity profiles to be about plus or minus ten percent of the soil overburden thickness. For the Expansion area, this error is typically in the order of a few feet, plus or minus, from the position shown on the resistivity profiles prepared by Northeast Geophysical Services. Care should always be exercised in utilizing such maps and if a specific area is in question, additional borings should be made. DEP has requested additional information on the depth to bedrock prior to siting monitoring wells outside the perimeter of the Expansion. This will be addressed through additional geophysical surveys and borings, within and outside the Expansion, which will be used to refine the final location of the new monitoring wells. The work plan that describes both the locations and timing for completing the additional investigations is found in Attachment SME-2.

Pg 4-4, 4.1.1 Basal Till. JRL describes the sand and gravel deposits located along the Stillwater River as outwash deposits formed in depositional environments beyond the ice margin. The Maine Geological Survey maps I have reviewed depict ice contact deposits (i.e., eskers) along the Penobscot River (DEP – Attachment C). This section may require some clarification.

Response: As illustrated in the LiDAR imagery there are both well-defined esker segments and broader sand and gravel outwash areas associated with the eskers. This is a common relationship, particularly near glacial ice margins. In either case, how it is described is less important than the fact that sand and gravel deposits exist local to Route 16.

<u>Pg 4-6, 4.2 Bedrock Geology</u>. As we discussed during the December 2, 2015 meeting with JRL and its consultant, SME, I am concerned that a sufficient number of bedrock explorations have not been completed within the eastern half of the proposed expansion. My specific concern relates to the absence of bedrock explorations within at least 50% of the proposed expansion (DEP – Figure 4). DEP – Figure 4 depicts all the bedrock explorations within and adjacent to the proposed expansion boundary. There are no bedrock explorations within the eastern half of the proposed expansion located north of the existing landfill.

Information obtained from surficial explorations, including borings, monitoring wells and test pits, appears to provide sufficient data regarding the thickness of surficial sediments for landfill design purposes. However, additional bedrock explorations are needed to refine the interpreted bedrock surface figure (i.e., Figure 4-5). I further contend that additional information about the nature of groundwater flow within the fractured bedrock is required to develop a defensible environmental monitoring program.

This section includes photographs (Figure 4-3) of the three prominent rock types encountered during the drilling program. Providing photographic documentation is an excellent idea but the photographs are too small and dark to be useful to the reviewer. Larger photographs, perhaps 8" x 10", would provide adequate detail. Larger photos would also permit JRL to annotate the photos with some of the important characteristics (e.g., foliation, calcite and quartz veins, relic bedding and fractures).

<u>Response</u>: During the January 29, 2016 meeting with DEP, the issues of depth to bedrock and groundwater flow pathways through the bedrock were discussed. Although the SME bedrock investigations focused on demonstrating that the bedrock was sufficiently fractured to transport groundwater similar to a porous medium, SME also recognizes that fracture zones were identified on the former resistivity transects. In recommending positions for these monitoring wells, SME attempted to focus on these less resistive zones with the idea that they may be more likely to concentrate and transmit groundwater than the surrounding, more resistive rock. However, we also recognize that additional information would be useful prior to finalizing the monitoring well locations. As a result, we are proposing that well installation be preceded by additional geophysical and boring investigations to refine the number, location and depths of monitoring wells for the Expansion.

In order to respond to this comment, SME has prepared a work plan to refine our proposed monitoring well locations as presented in the Application. The work plan includes additional surficial resistivity surveys to search for more permeable zones in the bedrock and collect additional information on bedrock depth. Additional large-diameter borings are planned to allow down-hole geophysical mapping of fractures, and to measure bedrock depth. This data will allow refined siting of well screens for monitoring groundwater around the Expansion. The work plan is included in Attachment SME-2.

The requested photos have been enlarged and are included in Attachment SME-3.

<u>Pg 4-8</u>. The bedrock investigation identified two primary fracture sets. One fracture set strikes northeast-southwest and the other northwest-southeast. According to the text both fracture sets are steeply dipping but no information about a predominant dip direction, if one exists, is given.

<u>Response</u>: Both sets are typically steeply dipping. Dip measurements can be found in Table 2-11 for outcrops OC-1 through OC-4, Appendix F for individual boreholes, and Appendix K for OC-AG and on rock core logs in Appendix C. A discussion of fracture dips is also presented on Page 4-8.

<u>Pg 5-1, 5.1 Groundwater in Soils</u>. Appendix D contains tables of monthly water level data for select wells. In addition to the data tables found in Appendix D, I recommend JRL graphically depict the water level information for a representative selection of monitoring wells. This information could be used to supplement the groundwater depth discussion in Section 5.1.3.

<u>Response</u>: The groundwater trend plots for the measured groundwater levels for representative wells are included in Attachment SME-3.

<u>Pg 5-2, 5.1.1 Horizontal Groundwater Flow Through Soils</u>. JRL's interpreted phreatic surface (Figure 5-1) demonstrates flow directions do not change significantly between seasonal high and low groundwater levels. However, what happens as liner construction reduces groundwater recharge? Will a decrease in the elevation of phreatic surface alter groundwater flow directions? Will it alter the location of the groundwater divide?

<u>Response</u>: This topic is discussed in Section 5.4 of Volume II of the Application. Groundwater will lower with the liner in place and groundwater flow will shift to the west. Appendix V shows that groundwater recharge beneath the liner will come from the north causing groundwater to exit from the northeast side of the landfill and towards the southwest (also see response to question on Section 5.4 below).

<u>Pg 5-4, 5.1.3 Groundwater Depth</u>. Construction of portions (12.7 acres) of the proposed expansion will require an underdrain because the base grade are expected to be below the water table. The text states, "….this will induce upward groundwater seepage into the excavations…." This description is misleading based on the interpretive vertical equipotential profiles. The profiles indicate groundwater movement is not upward throughout most of the underdrain. Rather, the excavation base grade simply extends beneath the surface of the water table. It's best to simply view the excavation as creating a groundwater outcrop. In fact, if JRL's interpretive vertical equipotential profiles

accurately represent in-situ conditions, I expect flow in the underdrains will be short lived as recharge decreases with construction of cell 13.

> Response: During the initial excavation into the till, even though there are downward gradients in the area of the underdrain, they are less than unity. For a short period upward seepage gradients will exist and may locally hamper construction in the fine grained till. The degree of upward gradient will depend on the rate of excavation, weather conditions and local heterogeneities in the till. These should dissipate fairly quickly providing there is not excessive traffic on the excavation surface.

Pg 5-23, 5.3 Regional Hydrologic Setting. JRL's conceptual model of groundwater flow in the vicinity of the proposed expansion and existing landfill is consistent with my understanding of the expected regional groundwater flow in the area. Due to the existence of the till ridge trending northward beyond the proposed expansion boundary, JRL expects the identified north-south oriented groundwater divide to cause groundwater beneath the northern edge of expansion to flow away from the divide (i.e., toward the northeast or northwest). I believe the report mistakenly stated groundwater west of the divide flows in a southwesterly rather than in a northwesterly direction. The interpreted potentiometric surface depicted on Figure 5-8 indicates a northwesterly flow.

> Response: We concur with DEP's comment and the report should refer to flow to the northwest and not southwest under existing site conditions. However as the Expansion is developed and the recharge is cut off from the Expansion Area, modelling shows that the groundwater flow direction will have a more southwesterly component than northwesterly, as is shown in Figure V-6 of Appendix V of the Application.

Pg 5-26. JRL's conceptual model of regional groundwater flow, based on the site's hydrogeologic setting and supported by the hydrogeological investigations, along with the computer simulations of regional groundwater flow, demonstrate the private water supplies located along routes 16 and 43 are isolated from groundwater flow paths originating in the vicinity of the JRL facility. I therefore agree with the concluding statement that there is little risk the water quality of the existing water supplies would be compromised in the unlikely event of a failure of the proposed secure facility.

> Response: We agree with this comment and see it as an independent confirmation that the Expansion is located in a hydrogeologic setting that is protective of existing water supplies.

Pg 5-26, 5.4 Post-Construction Groundwater Flow Directions. As groundwater recharge is gradually eliminated as the facility expands, the elevation of the water table surface will decrease. JRL expects the water table surface will also flatten as recharge decreases. Are these changes expected to alter current flow directions? This section could be improved by augmenting the verbal description of the anticipated future groundwater flow directions, with a figure depicting current and future flow directions. The computer model used to simulate current groundwater flow in the vicinity of the

landfill could be used to further refine our understanding of groundwater flow directions and how they may change when recharge is ultimately reduced to zero beneath the entire 122 acres. These simulations may help us determine how the location and orientation of the groundwater divide may change in the future. Knowledge regarding the location of the groundwater divide is particularly important to the design of the facility's long term monitoring program.

> Response: See our response to DEP comment on Section 5.1.1 above. Future interpreted groundwater flow directions are also addressed in our response to Appendix V of Volume II below. Note that the proposed monitoring well locations will be refined in concert with NEWSME, SME and DEP based on the findings of the work plan described in Attachment SME-2.

Pg 5-27, 5.5 Protection of Off-Site Groundwater and Surface Water. The results of the pumping tests definitely demonstrate a relatively well connected bedrock fracture system. Like JRL, I too interpret this as an important finding since it certainly suggests that pumping wells could be used to capture contaminants in the unlikely event of a liner failure.

The long-term pumping test conducted using PW-08-01 and PW-08-02 produced measureable drawdown in many of the observation wells, some located a considerable distance from the pumping wells. However, it is not accurate to equate drawdown with groundwater capture. For example, the roughly 7.0 feet of drawdown measured in P-04-07A, located 1,900 feet from PW-08-01, does not imply groundwater from this location will be captured. The apparent interconnected bedrock fracture system does suggest appropriately located bedrock recovery wells could be used to control and capture contaminants at this site.

The Department has consistently encouraged JRL to use the surface geophysical technique (2-D electrical resistivity) to identify potential transmissive bedrock fracture zones. Given the success of this technique at this site, it would be prudent to complete additional geophysical lines to identify additional fracture zones before further site development reduces the technique's effectiveness.

> Response: SME concurs; see our response to DEP's comment on Section 4.2 above.

Pg 6-1, 6.1 Expansion Water Quality Monitoring Locations. This section provides an overview of the Environmental Monitoring Plan (EMP) for the proposed expansion. The complete EMP is included in Volume IV of the application. My comments about the EMP are included here and following the Volume IV heading.

As currently proposed, the EMP described will include the addition of 23 monitoring wells, two new surface water sample locations and several leak detection and underdrain locations. JRL states that many of the proposed new well locations would not be installed until JRL constructs the cells they are intended to monitor. This is a commonly

accepted approach for an expansion of this size. In large part I agree with this approach but I contend some of the proposed bedrock wells should be installed as soon as possible. On DEP Attachment D I have highlighted the proposed wells that I recommend JRL install as soon as possible to further refine our understanding of groundwater flow in the underlying fractured bedrock. To maximize the usefulness of these explorations, I also recommend extending the target depth of the proposed wells. All of the proposed bedrock wells should extend 200 feet into bedrock. Data from the traditional suite of borehole geophysical tools can be used to determine the appropriate well screen intervals. Because JRL has completed few bedrock explorations within the proposed footprint, it may be prudent to locate some of the additional bedrock borings within the footprint. I would like to have a detailed discussion with JRL about the locations I propose for additional bedrock exploration/observation wells.

It is important for JRL to recognize that information gathered during the installation of these wells may ultimately result in further refinements to the EMP.

> Response: SME has discussed this comment in detail with Mr. Behr and has included as Attachment SME-2 a work plan that outlines the scope and schedule for a program to supplement the understanding of groundwater flow in the underlying bedrock, as presented in the Application, and refine the future placement of monitoring wells, also as presented in the Application.

Pg 6-2, 6.1.1 Leachate Monitoring for the Expansion. Leachate characterization at the existing licensed landfill calls for the collection of three samples per year from the leachate storage tank. The current parameter list includes: field parameters, geochemical parameters (i.e., Detection parameters) and volatile organic compounds. This program has successfully characterized the bulk leachate but it yields little information about how the leachate chemistry evolves as the waste volume within a cell accumulates and matures. In an effort to assess any significant difference in leachate character between the existing leachate stream and the leachate generated by the expansion, I recommend JRL also sample the leachate generated by the first cell (Cell 11) of the expansion. Initially I expect the chemical leachate characteristics of Cell 11 will differ markedly from the mature leachate generated by the existing landfill.

> SME Response: Because the existing site leachate sampling location is at the onsite leachate storage tank, which receives leachate from all the JRL cells, we agree that collecting a discrete sample of the leachate from the first expansion cell (i.e., Cell 11) would be useful to determine if a difference exists between the Cell 11 leachate and the combined JRL leachate collected in the tank. We propose to sample the Cell 11 leachate three times during the first year of operations in a manner consistent with the proposed sampling of leak detection and underdrain monitoring locations described in Section 3-3 of the proposed Environmental Monitoring Plan found in Appendix I of Volume IV of the Application to evaluate if the leachate within the new landfill cell is substantially different from the combined site leachate. At the end of the first year an evaluation of the difference between the two leachates would be completed as part of

the Annual Report and recommendations made as to any modification to the site monitoring program. These recommendations would be reviewed with the DEP and only implemented upon the DEP's approval.

Pg 6-2, 6.1.2 Leak Detection and Underdrain Monitoring for the Expansion. JRL clearly recognizes routine monitoring of the leak detection systems represents the primary method to evaluate liner performance. The current monitoring program for the existing landfill with leak detection includes monthly measurements of specific conductance and flow. JRL also collects samples for the full suite of laboratory and field parameters three times per year. The EMP for the expansion calls for monthly flow and specific conductance as well. I propose increasing these measurements from monthly to every two weeks. I also think it would be instructive to be prepared to measure the head in the leak detection system if the flow measurements warrant. Based on discussions with the Department's project engineer, Steve Farrar, I understand it would not be difficult to place pressure transducers in the lower portion of the leak detection system.

Response: The Liner Leakage Action Plan included in Volume IV Appendix P outlines the frequency of sampling of the leak detection layer for different conditions. The frequency begins at bi-weekly during the baseline period, then transitions to monthly provided the leak detection action level I (LDSAL-I) is not exceeded. If this level is exceeded, the sampling would be expanded to weekly, and potentially even to daily if the leak detection action Level II was exceeded. This flexible program provides a robust approach to monitor the leak detection system and we propose not to change to the program.

The operation of the leak detection pump is controlled by a transducer, which is placed in the leak detection sump. While a transducer could be placed hydraulically upgradient within the leak detection layer, potential flow within this layer would be controlled by the overlying primary composite liner system. The leak detection system is designed to limit head build up within the leak detection layer. Therefore, placing a transducer in the leak detection layer would not provide any better information on the performance of the liner system than is obtained by measuring the flow rate in the leak detection layer.

Pg 6-3, 6.1.3 Groundwater Monitoring Locations around the Expansion. As discussed earlier in this memorandum, I do not agree with portions of JRL's interpretation of the bedrock tracer test. I do not dispute that the introduced tracer was detected in the downgradient observation wells nor that the results demonstrate that the bromide tracer spread out over a wide arc as remnants of the injected tracer travelled toward the observation wells. I understand JRL currently contends the majority of the bromide tracer "dropped" out of the injection well due to the initial density of the tracer solution. Despite the significant loss of tracer, JRL believes the remaining tracer travelled horizontally toward the observation wells. It is also possible the tracer may have followed hydraulically transmissive fractures that pass beneath the downgradient fence of observation wells. My calculations support the contention that the observation wells virtually failed to detect the plume as far less than 0.1% of the expected bromide was

observed. If my interpretation is correct, it has important ramifications for locating downgradient bedrock wells in the flow path contaminants may follow in the event of a release.

> Response: Please see our response above to comment pg. 3-18 – 3.2.7 Groundwater Tracer Test in Bedrock. As we indicate in response to that comment, the bedrock tracer test obtained its goal of showing the interconnectedness of fractures by demonstrating the spreading of the tracer, regardless of where most of the tracer ended up. As we have already mentioned, we agree there is a benefit to collecting additional information on the bedrock beneath and around the Expansion to examine for fracture zones that may be important for monitoring groundwater around the Expansion (see the work plan included in Attachment SME-2).

Pg 6-6. In the discussion of the rationale for well placement JRL refers to large spreading of the tracer plume within 50 feet of the injection point. Specifically, JRL contends the solute spreading observed during the tracer experiment justifies spacing downgradient wells at distances ranging from 350 to 2,000 feet. To their credit, JRL has reduced the well spacing to 500 to 600 feet. However, all parties must recognize that dilution and dispersion of a contaminant plume will significantly reduce the concentration of the primary indicator parameters. The resulting "signal" in the observation wells may be difficult to observe above the groundwater quality changes resulting from site development. With this reality in mind, I would like to discuss the possibility of further decreasing the spacing of monitoring wells.

In recognition of the importance of monitoring background groundwater quality, JRL has included four wells in its proposal. Two of the wells/piezometers are located south of the existing landfill and are included in the EMP. The two existing piezometers that are new to the program are located north of the proposed expansion (MW-04-09A/P-04-09A and MW-04-09B/P-04-9B). With time, water quality data from these wells may be particularly useful as they appear to be located beyond the influence of all site activities with the exception of the access road. I am, however, concerned that 1-inch piezometers may not yield sufficient water. In fact, I recall the low yield from P-206A has made it difficult to collect sufficient water for all of the required analyses. Traditional 2-inch wells should serve as the standard monitoring well as required by Chapter 405 of Maine's Solid Waste Management Regulations.

> Response: The location of monitoring wells will be re-evaluated based on the findings of the work plan described in Attachment SME-2. The two proposed monitoring locations (MW-04-09A/P-04-09A and MW-04-09B/P-04-9B) will be upgraded to two-inch monitoring wells, if they remain a part of the final monitoring well network.

Pg 6-7, 6.2 Future Sampling Parameters. I recommend modifications to the initial characterization parameter list summarized in Table 6-2. Boron has seldom been monitored at this landfill, but it is commonly found at elevated levels in landfill leachate and it is a relatively conservative parameter.

Methane is another parameter I wish to add to the characterization parameter list summarized in Table 6-2. Because the wastes proposed for disposal will ultimately generate large quantities of methane, it is imperative to establish predevelopment levels of methane in groundwater in the vicinity of the proposed expansion. This is particularly important because methane is found occasionally in Maine's groundwater under natural conditions. In fact, JRL's current program has detected methane in the pore-water samples within the wetland west of the existing landfill.

> Response: Boron and methane have been added to the long-term monitoring program as requested. A revised Table 6-2 of Volume II is included in Attachment SME-4.

Pg 6-8. JRL has proposed an alternative analytical program for some wells. The proposal calls for sampling the wells designated with the prefix "OW" for field parameters twice each year and once for the complete list of laboratory parameters. This protocol will also be followed for the underdrain and leak detection sample locations. I approve of this approach. It will, however, be necessary to include a protocol (e.g., increasing parameter trends) that will trigger the collection of samples for laboratory analysis three times per year.

> Response: We agree and our recommendation is that the protocol for triggering collection of samples for laboratory analysis stipulates that changing the sampling program for these wells be based on a yearly evaluation of site water quality in these wells. The exception to this would be sudden and abrupt changes in water quality that cannot be explained by other site conditions. In the case of unexplainable sudden or abrupt changes in the water quality results from the well, the results would be immediately reviewed with the DEP and a supplemental monitoring program undertaken to assess the reason for the change in site water quality. A similar approach has been used in the past at the current site monitoring wells to the satisfaction of both NEWSME and the DEP.

Pg 6-10, 6.4 Groundwater Level Monitoring. In an effort to monitor the expected drop in the phreatic surface beneath the expansion footprint, JRL plans to install two vibrating wire pressure transducers. Providing the transducers operate reliably for the expected timeframe, the transducers will generate the empirical head data necessary to quantify how the phreatic water levels decrease with time. To ensure these measurements can be obtained for an extended time period, JRL may want to consider installing additional transducers to provide some redundancy in case of equipment failure.

> Response: As part of the Application two transducers have been proposed at the locations shown on Drawing C-102 in Appendix E of Volume III, to provide the redundancy discussed in this comment. To further ensure the long-term performance of these instruments, they will have a more rugged construction and cable than is provided with a typical pressure transducer.

EXHIBIT B

<u>Pg 7-1, 7.0 Travel Time Analysis</u>. This section outlines JRL's approach to conducting the required travel time analysis. The written summary is thorough and is supported by the spreadsheets included in Appendix X. In its response to comments, JRL should provide the Department with an electronic copy of the worksheets. I also recommend the revisions to this section include schematic cross-sections to illustrate the travel paths to each of the chosen sensitive receptors.

> <u>SME Response</u>: Per the direction of the DEP during the pre-hearing conference held on February 10, 2016, all documents that are part of the project record must be submitted in an unalterable form so Excel worksheets has not been included. However, the Excel worksheets will be made available for DEP review at the SME office in Cumberland, where they can be reviewed with the appropriate SME staff. Time of travel schematics are provided in Attachment SME-3, which illustrate the components of subsurface travel-time used in the analysis (e.g., vertically downward through the till, horizontally through bedrock, and for surface water receptors vertically upward through the till) for the various locations where the time of travel analysis were completed.

<u>Pg 7-2, 7.1 Selection of Site Sensitive Receptors</u>. JRL's analysis of potential sensitive receptors for the time of travel calculations identified the following receptors: three locations for potential future private water supplies; one location characterized with saturated sandy zones within the glacial till; and three locations where groundwater discharges to the surface water. The seven locations are shown on Figure 7-1. I generally concur with the sensitive receptors JRL has identified for the analysis. One might reasonably argue that the sandy zones within the glacial till represent a marginal sensitive receptor given its limited extent and the fact it is not connected to the mapped sand and gravel deposits. However, based on data obtained during the pumping tests, some of the wells (e.g., MW-06-01) screened in the sandy till are hydraulically connected to the fractured bedrock. Given the potential connection between the sandy till and a future private water supply (location B on Figure 7-1), including the sandy till as a sensitive receptor represents a level of conservatism in JRL's time of travel analysis.

> <u>Response</u>: SME agrees that including the sandy till zones as a sensitive receptor represents a level of conservatism in this analysis. No changes to the travel-time analysis are required, based on this comment.

<u>Nearest Existing Water Supply</u>. Given the considerable distance between the closest water supply and the proposed expansion, I agree with JRL that the existing private water supplies do not represent sensitive receptors. I do, however, believe JRL's simplified description of the area providing water to a single family home is misleading. I don't disagree that there may be sufficient recharge from an area within 300 feet of a well but this assumes the borehole penetrates a homogeneous and isotropic bedrock aquifer. In most instances, the fracture characteristics of the primary water bearing fractures dictate the area of influence of a pumping well. The other important point relates to the position of the well in the hydrogeologic system. For example, bedrock wells located at the toe of a gentle slope may intercept groundwater that has travelled a considerable

distance from the point of recharge. In fact, JRL has identified wells located along the western edge of the expansion footprint that intercept groundwater that has travelled in excess of 1,000 feet. Providing my questions about the groundwater ages determined using the helium-tritium age-dating method are satisfactorily addressed, JRL will have provided an independent estimate of a substantial travel distance/time.

> Response: In light of the discussion both above and below about the helium-tritium age-dating method, we believe that no further response is required.

Pg 7-6, 7.2 Improvement Allowances. The improvement allowances for the liner design allows for a two year offset for the majority of the expansion footprint and three years for the two areas where the secondary liner includes a geosynthetic clay liner and one foot of compacted clay. The two areas with the augmented secondary liner are shown on Figure 7-1. JRL's proposal also includes 12-inch of compacted marine clay beneath the entire footprint which qualifies for an additional three years of travel time. In summary, the total offsets provide for either five or six years of travel time for the entire footprint.

> Response: We agree with this comment, and it highlights the high quality of the proposed liner system design for the Expansion as being protective of the groundwater resources of the State.

Pg 7-8, 7.4 Calculated Travel Time to Site Identified Sensitive Receptors. I have reviewed the travel time calculations summarized in this section and the worksheets provided in Appendix X. Overall the technical approach and the resultant calculations appear straightforward and logical. Perhaps more importantly, the input values for the calculations are based on well documented site specific information.

I identified one minor error in the offset credits included in Tables 7-3 and 7-4. After speaking to Mike Booth of SME, I have concluded the tables mistakenly included a three year rather than a two year offset for the travel time calculations from the Cell 13 Leachate Sump (Point C) to the surface water discharge point. Reducing the calculated travel time by one year isn't critical since the travel time calculations in the till and bedrock exceeds 35 years to the discharge point. All of the relevant tables, however, should be revised to include the correct offset value.

Notwithstanding the minor error, the calculated travel times range from 6.2 to 41.8 years. In summary, the calculated travel times to all of the identified sensitive receptors exceed the required six year time of travel required by the regulations.

> Response: We agree. One additional minor correction is needed to the offset credits presented in the Application: Cell 11 Southern End to the Southern Sandy Zone. Two years was used, where three years should have been used, due to the presence of the augmented liner at that location. The calculated travel time continues to exceed that required by the DEP Rules. Revised Tables 7-3 and 7-4 of Volume II, along with the updated Volume II, Appendix X printouts are included in Attachment SME-4.

EXHIBIT B

<u>Pg 7-12, 7.5 Sensitivity Analysis</u>. To provide additional information about the range of estimated travel times to the sensitive receptors, JRL has completed a sensitivity analysis. The sensitivity analysis has used a range of effective porosities and hydraulic conductivities for both the till and bedrock. I concur with the range of values used in the analysis but the explanation, presentation and documentation must be improved. I also believe it is necessary to expand the sensitivity analysis to include estimates of travel times while using a combination of the low range porosities along with the highest hydraulic conductivities.

The report indicates the results of the analysis can be found in Appendix X. It appears Appendix X does not contain spreadsheets for all of the sensitivity runs used to populate the table (Summary of Sensitivity Analysis, JRL Expansion Application) summarizing the results of the sensitivity analysis. Rather than outline the specifics for the additional analysis in this memorandum, I would prefer to discuss my objectives directly with JRL and its consultant.

> <u>Response</u>: It is not common practice to vary two parameters simultaneously in a sensitivity analysis, since the purpose of a sensitivity analysis is to assess the effect that varying each assumption over some reasonable range has on the result. To vary two parameters simultaneously is more a means of looking at two unlikely situations occurring simultaneously, which in our view is not a valid assumption. We have, however, prepared the requested evaluation. Attachment SME-3 includes the results of the evaluations when varying two parameters.
>
> Individual, complete printouts for the sensitivity analysis were not included in the Application for brevity, the results, however are included in Attachment SME-4. We have added notes to the printouts to improve the explanation and documentation of the format and values contained on the printouts.

JUNIPER RIDGE LANDFILL EXPANSION APPLICATION
VOLUME II, SITE ASSESSMENT REPORT – APPENDICES A-X

<u>Appendix H – Field-Scale Bedrock Tracer Test Results</u>

The results of the bedrock tracer test were first reported in SME's September 2008 Bedrock Tracer Report. I reviewed this report and outlined my comments in an October 15, 2008 memorandum. I believe the most significant finding of the tracer test was the relative absence of the bromide tracer in the downgradient observation wells. The absence of tracer in the downgradient observation wells indicated the bulk of the introduced tracer did not travel through the well screens of the observation wells. My memorandum included a couple of explanations for the relative absence of tracer in the

observation wells. First, the predominant flow direction in the fractured bedrock may not be horizontal. Rather, it is possible groundwater flow in the shallow bedrock may have a significant vertical component of flow and the tracer simply travelled beneath the observation wells. Another plausible explanation is the tracer traveled vertically through the bottom of the well as a result of density driven flow. This may have occurred because the mass of bromide introduced into the injection well resulted in an initial salinity close to seawater.

The revised report contained in Appendix H concludes the majority of bromide was lost due to the density of the tracer slug introduced into MW-06-02. Just the same, JRL believes the "residual" tracer remaining in the injection well (MW-06-02) ultimately moved downgradient through the fence of observations wells. I agree that density driven flow helps explain the observed results. However, I am not convinced the tracer test data demonstrate groundwater flow is predominantly horizontal between the injection well and the observation wells. In fact, I believe the pumping test results revealed, at best, a relatively poor connection between the pumping (injection) well and the downgradient observation wells. Regardless of the correct explanation, it is possible the tracer's predominant flow path was toward the observation wells but the mass travelled beneath the observation wells. This is based on a series of calculations (DEP – Attachment A) used to provide a rough estimate of the expected bromide concentration one would expect to observe within the test volume. If the tracer's path was directly intersected by the observation wells, one would expect to measure bromide levels in excess of 100 mg/L, perhaps as high as 1,000 mg/L. In fact, the highest bromide concentration measured was 0.095 mg/L, a level far lower than the value I estimated. The bromide measured in the observation wells may represent the upper portion of the tracer plume as it travelled beyond and largely below the observation wells. Again, I don't dispute that the tracer travelled in the direction of the observation wells. The point of dispute relates to the tracer's trajectory. The data may, in fact, demonstrate a significant downward component of flow. Regardless, the uncertainty regarding the tracer's path underscores the importance of using nested monitoring wells (completed at varying depths) to detect possible leachate releases.

> Response: SME's interpretation of the pumping test is consistent with Mr. Behr's, as documented in Appendix H of Volume II. The principal direction of the relatively-dense bromide tracer was downward and that is how we modeled the tracer plume in the Application. The primary tracer flow direction was rotated downward by adjusting the relative position of the observation wells to simulate the density driven flow component. The observation wells intercepted the edge of the plume and provided useful data against which to calibrate the analytical model to estimate dispersion and groundwater velocity. The spreading of the tracer in all observation wells over an arc of at least 90 degrees downgradient of the injection well demonstrates the well-interconnected nature of the bedrock fractures. Had the fracture system not been well interconnected we would not have recorded the tracer or we may have only recorded it in one observation well.

The pumping test that was done on the well also demonstrates a well-interconnected fracture system in the vicinity of the test. This is evident from the fact that water level drawdowns were observed in all observation wells over a spread of around 100 degrees from the pumping well. In a poorly interconnected fracture system maybe only one or two observation wells would have recorded drawdowns. This integration of fractures is consistent with the tracer test in that five of the six observation wells, spread over an angle of about 90 degrees all intercepted the bromide plume. As discussed in our response to DEP's comments on Section 3.2.7 above, the test provides useful qualitative information about the interconnection of fractures that have practical applications for locating monitoring wells in the bedrock with confidence for detecting the unlikely event of a landfill liner leak.

The need for nested wells will be considered based on the findings of the Work Plan described in Attachment SME-2.

Appendix I – Helium-Tritium Groundwater Age Dating Results

As I have noted earlier in this memorandum, additional information must be included to support the use of this technique. In addition to providing relevant peer reviewed references on the subject, JRL should provide details about the sampling protocol followed to ensure the collection of representative samples for age-dating groundwater using the helium-tritium method. The chain-of-custody sheets for the samples collected are also needed.

The analyses were performed by the University of Rochester's Noble Gas Laboratory. Appendix I contains one laboratory sheet for each of the groundwater samples. The laboratory report for the sample collected from P-04-06A includes a comment stating the "Correction is too large to provide valid age. Large amount of terragenic helium - may be mixed water." This comment suggests the age determination is not valid. I also find the tritium data puzzling as the tritium activity (TU) of the sample collected from P-04-07B is higher than that of P-04-06A. Given tritium's 12.3 year half-life, the older sample (P-04-07B) should be characterized by a lower tritium activity than that of P-04-06A. JRL must clarify these apparent discrepancies so the Department can determine if the age estimates are valid.

Response: SME responded to DEP's questions on test protocol and methodologies above when we addressed questions on Section 3.2.8. The chain-of-custody forms are not available; however, the Monitoring Well Sample Purging Forms are attached in Attachment SME-3.

The comment about the terrragenic helium in the sample from P-04-06A was a cautionary statement by Poreda since he did not know where the sample came from. However, in comparing the initial tritium content of the sample with the historical precipitation tritium for the Ottawa, Canada monitoring station, the sample is consistent with the precipitation tritium for the estimated sample age. The initial tritium content is the sum of the measured tritium and the tritiogenic helium-3. Helium-3 is the by-product of tritium decay. This implies the sample is not mixed with older groundwater. The data from P-04-07B and P-04-06A is consistent with the estimated ages if one examines the initial tritium in the samples. The initial tritium is a sum of the sample tritium plus the tritiogenic helium-3 and is the tritium content of the precipitation. The initial tritium of the older sample (P-04-07B) is about 26 TUs. The younger sample is 15 TUs. This is consistent with the decay of tritium in the atmosphere resulting in less tritium in precipitation over time. Thus, the data is internally consistent.

Appendix J – MW-06-02 Groundwater Pumping Test Results

Pg 1, 1.0 Purpose. I understand the primary purpose of this test was to determine the interconnectivity of the fractures intersecting the pumping well (MW-06-02) and the downgradient observation wells. Presumably information about the fracture network helped JRL design and implement the bedrock tracer test. It is not evident, however, how JRL used the results from this pumping test to design and implement the tracer test.

Response: The pumping test results showed that the fracture system around the pumping well was hydraulically interconnected to the observation wells through a well-integrated fracture system. This qualitative finding (along with all the other bedrock data collected on-site) suggested a tracer test should demonstrate the same finding, that the bedrock fractures were well interconnected and we should observed tracer in most downgradient monitoring wells.

Pg 2, 3.0 Test Data. The graph in Attachment B depicts the pumping rates throughout the pumping test. This figure should be revised to include the initial pumping rate of 3.5 gpm that was subsequently determined to be too high.

Please provide an explanation for the Telog data displayed on the drawdown versus time for the pumping well. Specifically, there is a considerable amount of Telog data collected between 200 and 500 minutes that is not correlated with the manual measurements.

Response: The Telog data scatter between 200 and 500 minutes is not uncommon with pressure transducers. The cause is uncertain but likely has to do with a transient electrical issue. We have discussed this effect numerous times over the years with technical representatives of the transducer manufacturers; they have never been able to point to a specific cause. This is the reason that manual measurements are made, particularly in critical applications.

Pg 4, 4.0 Analysis of Results. As the report notes, the time-drawdown graphs for three of the observation wells indicate the water levels began to recover before the pumping test ended. The water level data for OW-06-08 clearly illustrate this phenomenon. The report mistakenly describes this as a decrease in drawdown rather than recovery of water levels (i.e., increase in head). This distinction is important as water levels in three of six wells began to recover as pumping continued. JRL believes the afternoon rain event provides

an explanation for the recovering water levels. However, based on the estimated slow travel time through the till, these shallow bedrock wells are not expected to respond so quickly to a rain event. Please provide further explanation.

My synthesis of the pumping test data suggests the pumping well is at best poorly connected to the observation wells. The relatively long lag period between the on-set of pumping and observed drawdowns in the observation wells indicates a less than well connected fracture system. Further, while drawdown in the pumping well ranged from 10 to 15 feet, maximum drawdown in the observation wells did not exceed 1.0 ft during the eight hour pumping test. Contrast these results with the drawdowns observed during the 24-hour pumping test performed on PW-08-01. Within 20 minutes of the 24-hour pumping tests, drawdowns were observed in an observation well located more than 1,200 feet from the pumping well. Overall, in my view, the results of the pumping test on MW-06-02 did not suggest it was well suited for the subsequent tracer test.

> Response: The reason SME describes the water level response after 400 minutes as a decrease in drawdown is that the pump rate is decreasing. The drawdowns are responding to lowering of the pump rate in the later part of the test as we began to shut it down. This drawdown recovery due to the lessening pump rate is the significant part of the water level response, not the precipitation. There is likely some water level change due to the precipitation event, but is overwhelmed by the declining pump rate in the later stages of the test. As stated in our response to DEP comment on Page 3-29, precipitation events will cause an immediate rise in groundwater levels due to the weight of the precipitation in the ground. The barometric efficiency of the specific portion of the groundwater system affected can be used to correct for this effect if significant.
>
> The lag in water level response has to do with the pump rate, storage coefficient, and transmissivity of the formation, not necessarily the degree of interconnection of pore spaces. For instance, in a fine grained soil the pore spaces are intimately connected but it takes some time for the drawdowns to expand away from the well. The degree of interconnectedness is demonstrated here by the fact that all observation wells over an arc of at least 100 degrees around the pumping well had measurable drawdowns. If the fractures were poorly interconnected some wells would drawdown and others would not.
>
> A direct comparison of drawdowns observed during the MW-06-02 pump test and the large-diameter wells is inappropriate. MW-06-02 was pumped at a time-weighted average rate of about 0.2 gallons per minute over an 8 hour period. Total volume of water removed from the bedrock was about 94 gallons. Drawdown in the pumping well averaged about 12 to 13 feet. By comparison, the approximately two-hundred-foot deep, large diameter wells that were positioned in the bedrock fracture zones (PW-08-01, PW-08-02, and PW-09-04) were pumped at between 32 and 96 gallons per minute for 24 to 50 hours with pumping well drawdowns of about 59 to 77 feet. Between approximately 52,000 and 276,000 gallons of water was withdrawn from each of these wells, compared to the 94 gallons withdrawn from MW-06-02. The longer pumping periods allowed for the cone-of-drawdown to extend further from the pumping well than

at MW-06-02, which is what was being sought. The MW-06-02 pump test was considered a local test to examine rock that was known to be well fractured based on the downhole geophysics results. It is interesting to note that the hydraulic conductivities and orientation of the principal directions of hydraulic conductivities calculated for MW-06-02 and PW-08-01 were similar. In addition, the tracer test in the bedrock showed the tracer to be entering the downgradient observation wells over an arc of at least 90 degrees. If the fractures were poorly interconnected, we would have expected to see no tracer at all or maybe one random observation well detecting the tracer. We continue to conclude that the fractures surrounding MW-06-02 were well interconnected and this is qualitatively supported by all the available data.

Appendix M – Hydraulic Analysis of Data from Long-Term Bedrock Pump Test at PW-08-01

Pg 3, 3.0 Pump Test Analysis. Water levels in some of the wells screened in the till responded to pumping PW-08-01. Although the hydraulic conductivity of the till is generally significantly less that the underlying bedrock, it is capable of supplying water. JRL's revised report should specify the wells where this occurred. Likewise, the shallow till wells where they observed little change in water level should also be noted.

I believe the data presentation would be improved if JRL summarized the pumping test data by depicting the maximum drawdown data observed at each well on a site plan. Later in this memorandum I outline suggested additional data analysis.

Response: As requested, we have summarized the till observation wells where drawdowns were observed. The range of drawdowns for each well during each pump test is shown on Figures U-14 and U-15 in Appendix U of Volume II of the Application.

Appendix U – Bedrock Fracture Interconnectivity

Pg 4, 4.0 Detailed Description of Bedrock Fracture Features at the Expansion Site. This section summarizes the bedrock characterization data collected in and around the expansion. At this time it bears repeating that JRL has only completed five bedrock explorations within the proposed expansion footprint (DEP – Figure 4). Further, only one (PW-08-02) of the four 200-foot bedrock borings is located within the footprint. A detailed justification for the relatively small number of borings within the 56 acre expansion is required. The degree to which the data collected beyond the footprint adequately characterize the bedrock underlying the proposed expansion is not adequately addressed in the current application.

Response: The justification for fewer borings is based on the several thousand lineal feet of earth resistivity that captures the bedrock surface and bedrock fracture zones. Having said this, we agree with DEP that supplemental data would be useful to refine our currently proposed monitoring well locations and we are proposing a work plan contained in Attachment SME-2, to collect supplemental data.

<u>Pg 5, 4.1 Bedrock Fracture Orientation</u>. Figure U-2 provides a rose diagram containing all of the orientation data for fractures observed on four bedrock outcrops and the "fracture" data obtained from the four bedrock borings logged using the optical televiewer. As JRL points out, there are two dominant fracture trends (i.e., northeast-southwest and northwest-southeast) and they are consistent with the results from the regional photolineament analysis.

Unfortunately, I was unable to locate the table containing the strike and dip data for three of the four outcrops.

> <u>Response</u>: The bedrock outcrop data is in Table 2-1 and Appendix K.

<u>Pg 8</u>. The fracture data collected with the optical televiewer for each of the four borings are depicted on Figures U-4 through U-7. The strike and dip data for the four boreholes are remarkably uniform. It is also noteworthy that the boring (PW-08-03) located on a resistivity high (i.e., low transmissivity) contained far fewer fractures than the three borings located on the resistivity anomalies.

> <u>Response</u>: We agree with the comment. The use of resistivity surveys are a valid technique at this site to identify highly transmissive bedrock, which in-turn can be used with confidence to locate bedrock wells for monitoring of landfill performance.

<u>Pg 13</u>. The two photographs (Figures U-8 and U-9) along with the fracture attitude data illustrate how two closely spaced fractures intersect to help create a relatively well interconnected fracture system.

> <u>Response</u>: We agree with the comment. The fracture system is interconnected, which shows that closely spaced wells (i.e., closer than 500 to 600 feet) around the landfill will adequately monitor the landfill.

<u>Pg 15</u>. JRL determined the fracture spacing for four bedrock cores (P-04-07, P-04-12, P-04-13 and P-04-14) collected from explorations outside the proposed expansion footprint. This section should also specify the total core length examined. I don't underestimate the importance of this data, but how do we know that it is representative of the bedrock underlying the proposed landfill?

> <u>Response</u>: A total of about 408 lineal feet of bedrock core was examined for these four borings; about one hundred feet per boring. The same type of bedrock was encountered in P-04-06 and PW-08-02, both of which are within the Expansion footprint. The same bedrock is also encountered beneath the existing landfill, which abuts the Expansion. The downhole geophysical logging for PW-08-02, within the Expansion footprint, shows relatively close fracture spacing, similar to the bedrock cores and outcrops. Thus, we believe this data is representative of the bedrock underlying the Expansion area.

<u>Pg 22, 5.0 Pump Test Proof of Bedrock Interconnectivity</u>. The pumping tests performed on the four 200-foot open bedrock boreholes certainly demonstrated the usefulness of conducting the 2-D Resistivity surveys to locate potential fracture zones. Interestingly, JRL suggests additional bedrock explorations may be located using this technique. After carefully reviewing the modeled 2-D resistivity lines, I urge JRL to consider locating additional bedrock explorations at several apparent anomalies. One of the additional bedrock explorations should target the low resistivity area identified on Line 6 (DEP – Attachment E). This apparent anomaly is located about 500 feet south of PW-08-01 along the eastern boundary of the proposed expansion. This is within the general area I have previously noted requires additional bedrock explorations and monitoring wells. Another apparent prominent low resistivity area appears on Line 8, roughly 500 feet south of P-04-09A,B (DEP – Attachment F).

> <u>Response</u>: We agree with the recommendations. MW-502 and OW-605 were located along the anomaly identified by resistivity Line 6, which falls between photolineaments. Line 8 is oriented approximately along the direction of flow from the northern end of the landfill and is approximately parallel the northeast-southwest trending bedrock fracture set, so this orientation was not considered conducive to monitoring. Lines 1 and 2 were used to attempt to align potentially highly transmissive zones when picking wells north of the Expansion. The work plan in Attachment SME-2 includes added lines, which may identify target locations for wells north of the landfill.

<u>Pg 25</u>. On Figures U-14 and U-15, JRL has illustrated the range of drawdowns observed in bedrock wells during each of the pumping tests performed on the four 200-foot bedrock boreholes. Additional illustrations are warranted to more fully convey the data collected during the tests. For example, the text states water levels were measured in 24 bedrock wells but the Figure only includes 20. Figure U-15 also appears to include drawdown data for some of the till wells although the Figure's title implies it is bedrock data only. This raises another point. It is also necessary to include figures illustrating the observed drawdown in the till wells during each of the pumping tests. The text states that significant drawdown occurred in some till wells during each pumping test. Comparing the drawdowns observed in both the till and bedrock wells during each test, may reveal locations where the hydraulic connection between the till and underlying bedrock is most pronounced.

As I have previously noted, I couldn't locate the tabulated drawdown data. It is important to obtain this data in an electronic format so the department can thoroughly analyze the data.

> <u>Response</u>: The manual water level measurements have been tabulated and were included in Appendix M of Volume II following the transducer drawdown plots.

<u>Pg 28</u>. JRL has combined all of the drawdown data (normalized to drawdown in the pumping well) collected during the five pumping tests to generate Figure U-16. This rose diagram provides an excellent illustration of the relatively uniform network of

transmissive fracture that exists on a site-wide scale. Transmissive fracture pathways appear to encompass all azimuths, albeit not from a single location.

JRL must include the tabulated data used to generate Figure U-16. Again, JRL should provide the data in an electronic format also.

> Response: The tabulated data is provided in Attachment SME-3. However per the direction of the BEP during the pre-hearing conference held on February 10, 2016, all documents that are part of the project record must be submitted in an unalterable form, so we are cannot provide the data in electronic format. It is in PDF form, however.

Pg 30, 6.0 Theoretical Confirmation of Bedrock Fracture Interconnectivity. In this section JRL makes the case the fracture density exceeds the so-called "percolation threshold" and therefore supports advective groundwater flow. I am concerned that JRL's analysis assumes the fracture network observed and mapped at the OC-AG outcrop is representative of the entire site. It is not clear to me how one extrapolates the findings from a single outcrop to an entire site. Please elaborate.

> Response: SME did not base its conclusion only on outcrop OC-AG. The conclusion is based on all the outcrop mapping for the Site, all the downhole geophysical fracture mapping, all the bedrock cores, the photolineament mapping, and MGS regional mapping. The data collectively indicate there are numerous fractures at relatively close spacing of a few feet or less that occur in fractures sets that intersect one another and the fracture lengths are greater than the fracture spacing. Therefore, on the scale of the Expansion, with fractures intersecting at distances of less than a foot, it is reasonable to conclude there is significant fracture interconnectivity. The pump tests performed confirm this interconnectivity from a hydraulic perspective by demonstrating drawdown in all directions away from the pumping well for distances of up to a couple thousand feet. The bedrock tracer test results are consistent with well interconnected fractures as stated above and are inconsistent with limited or no interconnection due to the observed tracer spreading. The data collectively are the basis for our conclusion that bedrock fractures on the scale of the Expansion are well interconnected (see Appendix U).

Pg 31, 7.0 Conceptualization of the Bulk Bedrock Groundwater Flow. JRL, in my view, makes a compelling argument for treating the fractured bedrock, at least on a site-wide scale, as an equivalent porous medium. Therefore, JRL has reasonably chosen to model groundwater flow in the surficial and bedrock aquifers using the USGS' MODFLOW numerical model. MODFLOW can be expected to model current conditions and evaluate future scenarios. An important future scenario includes an evaluation of how groundwater flow directions may change once recharge is reduced to zero beneath the landfill's footprint.

> Response: Appendix V includes a scenario of elimination of recharge from the Expansion footprint and its effect on groundwater flow directions beneath the Expansion.

<u>Appendix V – Groundwater Simulation Juniper Ridge Landfill Expansion Old Town, Maine July 2015</u>

<u>Pg 15, 5.0 Simulation Results</u>. The last sentence in the first paragraph states, in part, "….that the anisotropy of groundwater flow through the shallow and deep bedrock is evident." Without additional explanation this statement has no significance.

Model simulations included reducing the recharge to zero over the existing facility and the proposed expansion. These simulations incorporated particle tracking to determine the potential fate of groundwater originating in the vicinity of the landfill. The particle tracking simulation is shown on Figure V-6 and demonstrates groundwater originating from beneath the landfill ultimately discharges to the surrounding streams.

I strongly recommend JRL expand this aspect of the modelling to include pre and post equipotential head data and the estimated groundwater flow directions. Using the model to quantitatively determine how the water table changes in response to reducing recharge to zero seems like a particularly important question to address. As stated previously, predicting the future location of the drainage divide is important to the facility's long-term environmental monitoring plan. I recognize it may require a finer discretization of the model domain to produce output meaningful at the scale of interest.

<u>Response</u>: The partial sentence quoted at the outset of this comment is poorly worded. It is intended to mean that if one examines the simulated groundwater flow directions and compares them to the equipotential contours, they are not exactly perpendicular like they would be in an isotropic medium; in an anisotropic medium they are not perpendicular.

Regarding DEP's recommendation to include pre- and post-equipotential head data and the estimated groundwater flow directions (relative to recharge cutoff changes), Section 5.0 of the Model Simulation includes: (1) Figure V-5, which illustrates the groundwater head equipotential contours for model layer 2 (i.e., near the phreatic surface) based on approximate recharge cutoff conditions for the period selected for calibration (i.e., April 2009); and (2) Figure V-6, which illustrates groundwater particle pathways away from the existing landfill and expansion area with recharge cutoff over both the existing landfill and expansion area.

Based on DEP's recommendation, two supplemental figures are provided in Attachment SME-3. Figure V-5S supplements Figure V-5 and includes groundwater particle pathways away from the existing landfill and Expansion area with approximate recharge cutoff conditions for the period selected for calibration (i.e., April 2009). Figure V-6S supplements Figure V-6 and includes groundwater phreatic surface contours in the area of the existing landfill and Expansion with recharge cutoff over both the existing landfill and expansion area.

Based on a comparison of Figures V-5 and V-6S, the phreatic surface elevations decrease in the area of the existing landfill and Expansion as a result of the simulated recharge cutoff. Groundwater heads were compared at 29 locations at equal spacing within the expansion area for pre- (i.e., April 2009 conditions) and post-expansion development in the model. Post-expansion development recharge cutoff results in an average decrease in head of 23 feet at those locations in the model, with a maximum decrease of 33 feet in the interior of the expansion and a minimum decrease of 8 feet along the northern perimeter of the expansion.

Figures V-5S and V-6 illustrate that the divide of the groundwater particle pathway flow directions (i.e., the groundwater divide) shifts to the east as a result of the recharge cutoff.

SME further discretized the model in the area of the existing landfill and expansion by refining the cell spacing from 100 feet by 100 feet to 25 feet by 25 feet. The changes in simulated groundwater particle pathways and groundwater heads were negligible.

JUNIPER RIDGE LANDFILL EXPANSION APPLICATION
VOLUME III, DESIGN REPORT

<u>Pg 4-1, 4.0 Contaminant Transport Analysis</u>. As required by the Solid Waste Regulations (401.2 G), an expansion application requires a contaminant transport analysis. This analysis is required to evaluate the potential of a variety of hypothetical failure scenarios to pose an unreasonable threat to the identified sensitive receptors. In my view, information obtained regarding the potential threats to sensitive receptors is conservatively addressed by the completion of a thorough time of travel analysis which JRL has completed. Regardless, this section describes the hypothetical failure scenarios evaluated, the analytical methods used for the analysis and the results.

Based upon my review, it appears JRL has completed a satisfactory contaminant transport analysis. The failure scenarios evaluated do not reveal an unreasonable risk to the sensitive receptors.

<u>Response</u>: We agree with DEP finding that the contaminant transport analysis is adequate to meet the Rules and demonstrates the proper siting and design; and that no unreasonable risk to the sensitive receptors will exist for the Expansion. Hence the Expansion meets the Performance Standard Specific in Chapter 401(1)(d).

<u>Pg 4-9, 4.4 Hypothetical Failure Scenarios</u>. This section describes the three failure scenarios, along with a summary table (Table 4-3) of the contaminant transport analysis. Table 4-3 contains a portion of the summary data for the analytical solute transport equation used in each of the failure scenarios. In its current form Table 4-3 includes the alkalinity, arsenic and nitrate data. Table 4-3 should be revised to include the analytical solutions for all six of the leachate constituents in Table 4-1.

Response: Included in Attachment SME-4 is the updated table as requested.

JUNIPER RIDGE LANDFILL EXPANSION APPLICATION
VOLUME IV, OPERATIONS MANUAL

Appendix I – Environmental Monitoring Plan

I have completed a comprehensive review of JRL's Environmental Monitoring Plan (EMP) for the proposed expansion. In the following section I have outlined a number of comments related to the proposed EMP but perhaps more importantly I have outlined a variety of alternatives to the generally accepted approach used to monitor potential releases from secure facilities. Because the JRL facility is State owned and privately operated, it represents a unique opportunity to cooperatively explore one or more alternative monitoring approaches. Once JRL and its consultant, SME, have an opportunity to consider my suggestions, I recommend we meet to discuss the potential to implement one or more of the alternative approaches.

Response: NEWSME is willing to entertain other approaches to environmental monitoring for the site, however these approaches should be discussed outside of the permitting of the Expansion since they go beyond what is required by the Rules that govern the permitting of this facility.

Pg 3-1, 3.1 Groundwater Monitoring. In general, I agree with both the number and locations of the proposed new wells. Based upon my earlier comments, it will not come as a surprise that I recommend deeper bedrock explorations and wells along the eastern boundary of the proposed expansion. To provide a couple of specific examples, OW-604A and OW-605A should be paired with deeper bedrock wells. The use of air rotary drilling techniques would enable JRL to cost effectively complete boreholes extending to target depths in the neighborhood of 200 feet below the bedrock surface. The subsequent characterization of the bedrock explorations will enable JRL to screen the appropriate fracture systems.

Response: We agree and will work collaboratively with the DEP to optimally locate the site monitoring wells using both known site characteristics and the supplemental information collected from the completed work discussed in the work plan contained in Attachment SME-2.

Pg 3-1, 3.2 Surface Water Monitoring. The expansion will include two additional surface water monitoring locations. Because flow in these headwater streams is maintained, in part, by discharging groundwater, I strongly recommend JRL consider installing permanent pore-water samplers to monitor the quality of discharging groundwater at each of these locations.

<u>Response</u>: We concur with this suggestion and have added a number of permanent pore-water sample locations to the Expansion monitoring locations. These locations are shown on Figure 6-1 of Volume II and Figure 3-1 of Appendix I in Volume IV, included in Attachments SME-1. Table 3-2 in Appendix I of Volume IV has also been updated and is included in Attachment SME-4.

<u>Pg 4-1, 4.0 Selection of Monitoring Parameters</u>. The parameter list summarized in Table 4-1 should be revised to incorporate the comments contained in this memorandum. At this time I recommend the addition of the following parameters: boron, methane and tritium.

<u>Response</u>: We have added boron and methane to the proposed monitoring program and Table 4-1 has been updated to reflect these parameters, as provided in Attachment SME-4. We have not included tritium in the program, however, but understand that both BGS and NEWSME would not be opposed to discussing alternate sampling programs, such as described in your initial comment of Volume IV, independent of the Expansion's permitting process.

EXHIBIT B

Recommended monitoring alternatives for evaluation

1) Researchers have found leachate generated by municipal solid waste may contain significant tritium.[3] A preliminary survey completed by the Department found Maine's landfill leachate was characterized by tritium activity[4] in excess of expected background. In fact, the Department's survey found JRL's leachate contained significant tritium activity. Tritium may therefore serve as a valuable tracer. To evaluate the potential usefulness of tritium, I recommend JRL determine the tritium content of the current leachate. JRL can initiate this characterization in 2016.

> Response: We understand that both BGS and NEWSME would not be opposed to discussing alternate sampling programs such as you described independent of the Expansion's permitting process.

2) On several occasions during the past year I have suggested the possibility of incorporating a tracer into the protective base layer of the liner system. Because the proposed expansion will be constructed in phases, we will have an opportunity to explore this possibility using a variety of approaches. For example, JRL could incorporate a tracer into cell 11. Once waste disposal begins, JRL could analyze both the leachate generated by this cell and its leak detection system for the introduced tracer. An ideal tracer will be soluble, conservative and not generally detected in Maine's groundwater. During the past several years researchers have developed techniques that embed synthetic DNA in polylactic acid microspheres.[5] These techniques are in their infancy but hold tremendous promise in part because the particles can be uniquely labeled, detected at extremely low levels and are not prohibitively expensive. Since the JRL facility is a privately operated state owned facility, it is a particularly good site for which to evaluate the usefulness of tracers.

> Response: We understand that both BGS and NEWSME would not be opposed to discussing alternate sampling programs such as you described independent of the Expansion's permitting process.

3) Historically, monitoring well networks have been successfully used to detect and monitor the level of contamination downgradient of *unlined* landfills. Today we routinely characterize downgradient groundwater at double–lined secure landfill facilities, but the traditional downgradient fence of monitoring wells no longer represents the initial means to detect a liner failure from a secure double-lined landfill. JRL's proposed liner design incorporates a leak detection layer positioned between a primary and secondary liner system. Today, robust monitoring of the leak detection system represents the primary method of detecting a failure in the primary liner. In the event of a significant leachate

[3] Hackley, K.C., C.L. Liu, and D.D. Coleman. 1996. Environmental Isotope Characteristics of Landfill Leachates and Gases. Groundwater: Vol. 34, No 5.

[4] Behr, R.S. and R Heath. December 2010. Tritium activity in landfill leachate and contaminated groundwater in Maine

[5] Sharma, A. N., D. Luo, and M.T. Walter. 2012. Hydrological Tracers Using Nanobiotechnology: Proof of Concept. ES&T. Vol 46 (16) pp 8928-8936.

release, I expect the most soluble components would be detected by the downgradient groundwater monitoring well network. However, long before there are any indications of contamination in downgradient groundwater, monitoring data from the leak detection system will provide an early warning.

> Response: We agree that based on the proposed expansion design the site monitoring wells are not the "initial" means of monitoring landfill liner performance. The proposed secondary liner and leak detection system provides both a means to monitor the performance of the Expansion's primary liner (i.e., the system that provides for the containment and collection of landfill leachate) and the initial means to detect and implement corrective actions due to a liner failure. The early warning afforded by the monitoring of the leak detection layer allows for a response action to be implemented before the groundwater monitoring network would detect such a leak. The approach used to monitor and respond to results from the leak detection monitoring is described in Volume IV, Appendix P of the Application.

Appendix B of Review Memorandum
January 14, 2016 Memorandum from Gail Lipfert Re: Juniper Ridge Landfill Pumping and Tracer Test Evaluation.

1. The purpose is stated as determining to what extent bedrock fractures are integrated or hydraulically connected. It is not clear if they mean to assess the nature of bedrock fractures across the site or only those between the pumping well and the observation wells involved in this test

> Response: The purpose of the pump test was to qualitatively examine the bedrock fracture interconnectivity in the vicinity of MW-06-02. The bedrock pumping test was used as a means to corroborate earlier conclusions that the bedrock fractures were, in general, well interconnected. Previous data collected from bedrock outcrop mapping, bedrock core samples and Maine Geological Survey mapping showed that the bedrock was commonly fractured. The data showed fracture spacing of less than a foot in most areas, fracture lengths typically greater than the fracture spacing, and fractures oriented in virtually all azimuths and dips (although there were two prominent fracture sets). These conditions were identified for the existing landfill back in the early 1990s, the proposed Expansion area and the areas surrounding the existing landfill and proposed Expansion. These repetitive findings, along with our experience with similar bedrock at other sites, led us to conclude the bedrock fractures were well interconnected. The groundwater pumping test, as well as the tracer test, are believed to be a useful and efficient way of testing this conclusion. The test was planned to provide a qualitative indication of whether the fracture system in a localized area of "typical" Site rock would result in drawdowns of groundwater in observation points surrounding the downgradient side of the pumping well. If only one well or two non-adjacent observation wells drew down, there may be some question as to the interconnectedness conclusion. However, because all observation wells drew down, spread over an arc of almost 180 degrees, the fracture interconnectedness was confirmed. This achieved our goal of testing the

bedrock in a qualitative manner. The fact that the drawdown data from the test could be analyzed by common porous media methods to estimate bedrock hydraulic conductivities, and these results agreed with slug test results, provided further confirmation of our interpretations. Not only were we able to calculate the similar hydraulic conductivities, but the Papadopoulos method showed an azimuth of maximum hydraulic conductivity which aligns with the prominent bedrock fracture set. The same result was calculated for the PW-08-01 pump test, lending further credibility to the results. Appendix U explains our rationale for considering the bedrock fracture system as being generally well interconnected.

2. Pump Test Procedure:

2a: Groundwater elevations were recorded every 5 minutes, whereas it is recommended that pumping tests within fractured bedrock be monitored more frequently at the very beginning to see the effects of fracture control on drawdown, then monitored less frequently later on.

Response: Our interest was getting the semi-log straight-line drawdown data, which we did starting around 100 minutes.

2b: The initial pumping rate was only sustainable for 1 minute 20 seconds, which is not very long. They should have conducted a step-drawdown test first to establish the pumping rate.

Response: We were aware of the well's yield based on the downhole flowmeter data obtained during the geophysical logging of the well (see Appendix F of Volume II of the Application). The pump was simply started at full throttle and backed off as needed to sustain a pump rate. This approach has no effect on the purpose of the test, or on the transmissivity calculations.

2c: They only monitored wells immediately downgradient of the pumping well, but they could have monitored the surrounding wells to see if there was any effect.

Response: There are no other nearby wells in bedrock to monitor. The closest was about 1,000 feet away to the east. Drawdown in wells about 50 feet away were in the order of 0.1 to 0.8 foot, and so we concluded that at 1,000 feet, drawdowns would have been unmeasurable. Greater distance monitoring was necessary for the deep bedrock boreholes with pump rates ranging from about thirty to one-hundred gallons per minute and drawdowns at the pumping wells of sixty feet or more.

2d: There is no mention of borehole geophysical results to help understand the fracture system in any of the wells.

Response: The geophysical data is presented in Appendix F and discussed in Section 3.2.5 of Volume II. The geophysical data is typical of the rest of the bedrock on the site.

It shows the two principal fracture sets with typical dips in the order 40 to 50 degrees. Downhole flow meter data showed groundwater advection through the well under ambient natural gradients. No major fracture zone feature that would control the majority of groundwater flow into the well was observed in the geophysical log or core.

2e: They started monitoring one minute after pumping started instead of monitoring for a day or two before the test to establish any background water level changes and trends.

Response: This test was intended to last long enough to collect the semi-log, straight-line drawdowns (maybe up to 8 hours), which it did, and long-term trend data was not necessary. The straight-line portion of the drawdown curves lasted about five hours and would have been unaffected by typical long-term water table trends.

2.f: They conducted the test during a thunder storm. The responses at OW-06-08, OW-06-09 and OW-06-10 to the rain storm at 200 minutes are abrupt and almost instantaneous, which indicates poorly-constructed wells.

Response: As stated elsewhere, the changes in drawdowns at around 200 minutes are due to decreasing pump rates, not the precipitation. Furthermore, the monitoring wells have 20 feet or more of bentonite chips effectively sealing them from the ground surface.

2.g: The Telog and manual water level measurements do not match at MW-06-02 between 200 and 500 minutes in Attachment C.

Response: As indicated in response to the earlier DEP comment (Pg. 2, 3.0 Test Data), this was likely due to an intermittent electrical problem which these transducers are prone to. That is the principal reason why manual measurements are taken periodically and are considered more reliable in this particular instance.

2.h: They don't seem to have Telog data from a couple of the wells (OW-06-05 and -06)/

Response: We did not have enough transducers to instrument all wells, nor was it necessary to meet the objective of the test.

3.a.i: The time at which the observation wells responded to the pumping are in the following order, from shortest to longest: OW-06-09, -10, -05, -07, -08, and -06 (9, 20, 35, 45, 45, and 75 min, respectively). The wells with the shortest response time would be the wells with a more direct fracture pathway.

Response: We agree.

3.a.ii: The depth to which the water levels responded to the pumping are in the following order, from greatest to least: OW-06-07, 09, 05, 10, 06, and 05 (0.78, 0.65, 0.54, 0.43, 0.16, 0.15 ft). The wells with the greatest responses would be the tightest wells.

Response: The ones with the greatest responses may or may not be the “tightest wells”, but the data is the result of the complex, anisotropic, three-dimensional fracture system between the pumping well and observation wells. Even though the details of how best to analyze the data may be arguable, the test served our purpose and qualitatively demonstrated the interconnectedness of the bedrock fracture system in the vicinity of MW-06-02.

3.b: Table J-1. I don't understand what they mean by the "approximate radial azimuth for the various observation wells relative to the two predominant fracture set strike orientations (northeast/southwest and northwest/ southeast)". There is only one azimuth value listed, but there are two strikes that they are described as being relative to. I would be more interested in the azimuth of the strike between the observation wells and the pumping well relative to true north.

Response: Table J-1 is labeled incorrectly, as the azimuths listed are relative to True North as is the text. The text should say “Table J-1 lists the approximate radial azimuth of each observation well relative to True North.”

3.c: The analysis of maximum and minimum principal transmissivities using the Papadopoulos method has been presented only for five well groupings because these "provided meaningful results". How did they determine which results were meaningful?

Response: When a calculation is made, two observation wells are selected along with the pumping well. Because the method calculates the two principal horizontal transmissivities, observation wells with greater angular separations (up to ninety degrees) are more sensitive to observing anisotropic effects than wells that are lined up within a few degrees of one another. When the wells are closer together radially the calculation will not show much variation since both wells should theoretically have similar transmissivities. To determine which results were “meaningful,” we selected five sets of observation well pairs that we judged based on past experience with the method would provide a significant difference between the two calculated principal transmissivities and therefore, their azimuths.

3.d: Last paragraph states that the hydraulic conductivities estimated from dividing the transmissivities in Table J-1 by the well screen length are greater than measured at the observation wells. I do not understand this statement - what are the hydraulic conductivity values that were measured at the observation wells?

Response: The results of in situ slug testing (i.e., hydraulic conductivity values) of these observation wells are presented in Table 3-2.

4. Appendix H 4.0: second paragraph. OW-06-10 and OW-06-07 are aligned with the two dominant fracture orientations, but these wells have later arrival times (3 and 3.6 days, respectively) than OW-06-09 and OW-06-08, which received tracer after 0.8 and 1 days, respectively. SME interpret these results along with the fact that the wells with the

steepest groundwater gradients have the longest travel times, to indicate that the predominant fractures had more influence on tracer velocity than groundwater gradients. I don't see that the predominant fracture orientations have much influence at all. I would say that it appears that there are fractures outside the predominant orientations that are hydraulically connected between MW-06-02 and OW-06-08 and OW-06-09.

Response: In examining the tracer test results, the average direction of the groundwater flow gradient, based on Figure H-1, is to the east, even though horizontal seepage gradients are not uniform downgradient of the injection well. The strike direction of maximum fracture frequency is to the north-northeast/south-southwest. This is along the foliation pattern of the bedrock. Combining the gradient and fracture strike suggests to SME that the horizontal plume migration direction is more or less west-southwest from the injection well, if conditions were ideal and uniform (the tracer cannot move northeast or east since those directions are upgradient). Therefore, to observe the tracer first in the southwest quadrant is not surprising and might be expected if conditions were uniform. Movement of the tracer plume in other directions would be delayed. This is essentially what is observed and the reason for our conclusions as stated in Section 4.

5: Appendix H 4.0 third paragraph. This paragraph suggests that the early arrival of tracer at OW-06-09 is consistent with the interplay between the principal hydraulic conductivity orientation (along predominant fracture sets) and the hydraulic gradient. I agree that the interplay between the principal hydraulic conductivity orientation and the hydraulic gradient controls plume direction, but using this logic, the tracer should arrive at OW-06-07 first instead ofOW-06-09. This paragraph doesn't really explain why tracer arrived at OW-06-09 first.

Response: See response to Comment 4 immediately above.

6: Overall conclusions. One of the major assumptions in this analysis is that there are two principal transmissivities along two axes of an ellipse, but examination of the drawdowns at 200 minutes (before recharge affected the drawdowns) shows that the pattern of drawdowns is very irregular and cannot be described as an ellipse of anisotropy. The drawdowns also clearly indicate that the site is heterogeneous, which negates an underlying assumption for Papadopoulos's method. In general, it appears that the interconnectivity of the observation wells to the pumping well is quite variable and cannot be explained by the predominant fracture orientations or principal hydraulic conductivity orientations.

Response: The bedrock in the vicinity of MW-06-02 contains fractures in various orientations. When pumping on this well, drawdowns are observed in all radial directions where observation wells are located. This shows that all the fractures within about fifty feet of the pumping well are integrated with the pumping well and interconnected with other fractures. This was our objective for the test. These observations suggest to us that the bedrock fractures are well integrated and interconnected. The test, therefore, corroborates the interpretation that this should be the case based on the vast amount of bedrock data collected around the Expansion Site and existing landfill (see Appendix U).

We appreciate the opportunity to clarify and expound on the information contained in the Expansion Application in response to the comments and recommendations provided in the January 15, 2016 memorandum. The JRL site has been extensively studied since the early 1990s. Site investigations conducted to-date at the JRL site include the installation of over 80 borings, 94 test pits, seismic refraction surveys (approximately 34,000 lineal feet of transects), photolineament mapping, bedrock outcrop mapping, in situ hydraulic conductivity testing, groundwater measurements (wet- and dry-season), groundwater age-dating, groundwater tracer test analysis, numerous bedrock pumping tests, and water quality sampling and analysis. The information contained in these responses and the additional supplemental investigation associated with the proposed work plan to optimally locate the expansion site monitoring wells will further supplement this geologic and hydrogeologic information on the site. In total, these investigations and analysis support the fact that the proposed Expansion meets all of the relevant siting and operational criteria outlined in DEP Chapter 400 and Chapter 401.

<u>LIST OF ATTACHMENTS</u>

SME-1 VOLUME I AND II UPDATED FIGURES

- VOLUME I, APPENDIX M, SITE SURROUNDINGS MAP
- VOLUME II, FIGURE 2-8, BEDROCK WELL YIELDS
- VOLUME II, FIGURE 3-2, PHOTOLINEAMENTS AND FRACTURE TRACES
- VOLUME II, FIGURE 6-1, PROPOSED MONITORING LOCATIONS FOR GROUNDWATER AND SURFACE WATER
- VOLUME VI APPENDIX I, FIGURE 3-1 ENVIRONMENTAL MONITORING LOCATIONS

SME-2 VOLUME II WORK PLAN FOR REFINING LOCATION OF MONITORING WELLS AT THE JUNIPER RIDGE LANDFILL EXPANSION

SME-3 VOLUME II APPENDICES, SUPPLEMENTAL INFORMATION AND FIGURES

- APPENDIX M, SUMMARY OF INSTRUMENTED WELLS AND PIEZOMETERS DURING PUMPING TESTS
- APPENDIX R, TEST METHODOLOGY FOR BASAL TILL LABORATORY COLUMN TRACER TEST.
- FIGURE 4-3, ENLARGED PHOTOGRAPHS
- APPENDIX D, GROUNDWATER TREND PLOTS AT REPRESENTATIVE LOCATIONS.
- APPENDIX X, TIME OF TRAVEL SCHEMATICS
- APPENDIX X, TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS
- APPENDIX I, FIELD DATA FORMS
- APPENDIX U, TABULATED DATA USED TO PRODUCE FIGURE U-16

- APPENDIX V, FIGURE V-5S, GROUNDWATER PARTICLE PATHWAYS FOR MODEL CALIBRATION
- APPENDIX V, FIGURE V-6S, GROUNDWATER TABLE WITH RECHARGE CUTOFF

SME-4 VOLUME II, VOLUME III AND VOLUME V UPDATED TABLES AND APPENDICES

- VOLUME II, TABLE 6-2, ANALYTICAL PROGRAM
- VOLUME II, TABLE 7-3, CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS- EXISTING CONDITIONS
- VOLUME II, TABLE 7-4, CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS-FUTURE CONDITIONS
- APPENDIX X, UPDATED PRINTOUTS FOR THE TRAVEL TIME ANALYSIS
- APPENDIX X, ADDITIONAL PRINTOUTS AS REQUESTED BY DEP COMMENT ON PAGE 7-12, 7.5 SENSITIVITY ANALYSIS
- VOLUME III, TABLE 4-3, SUMMARY OF CONTAMINANT TRANSPORT ANALYSIS
- VOLUME IV, APPENDIX I TABLE 3-2, SURFACE WATER, PORE-WATER, LEACHATE, UNDERDRAIN, AND LEAK DETECTION MONITORING LOCATIONS
- VOLUME IV, APPENDIX I, TABLE 4-1, ANALYTICAL PROGRAM

- APPENDIX V, FIGURE V-5S, GROUNDWATER PARTICLE PATHWAYS FOR MODEL CALIBRATION
- APPENDIX V, FIGURE V-6S, GROUNDWATER TABLE WITH RECHARGE CUTOFF

SME-4 VOLUME II, VOLUME III AND VOLUME V UPDATED TABLES AND APPENDICES

- VOLUME II, TABLE 6-2, ANALYTICAL PROGRAM
- VOLUME II, TABLE 7-3, CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS- EXISTING CONDITIONS
- VOLUME II, TABLE 7-4, CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS-FUTURE CONDITIONS
- APPENDIX X, UPDATED PRINTOUTS FOR THE TRAVEL TIME ANALYSIS
- APPENDIX X, ADDITIONAL PRINTOUTS AS REQUESTED BY DEP COMMENT ON PAGE 7-12, 7.5 SENSITIVITY ANALYSIS
- VOLUME III, TABLE 4-3, SUMMARY OF CONTAMINANT TRANSPORT ANALYSIS
- VOLUME IV, APPENDIX I TABLE 3-2, SURFACE WATER, PORE-WATER, LEACHATE, UNDERDRAIN, AND LEAK DETECTION MONITORING LOCATIONS
- VOLUME IV, APPENDIX I, TABLE 4-1, ANALYTICAL PROGRAM

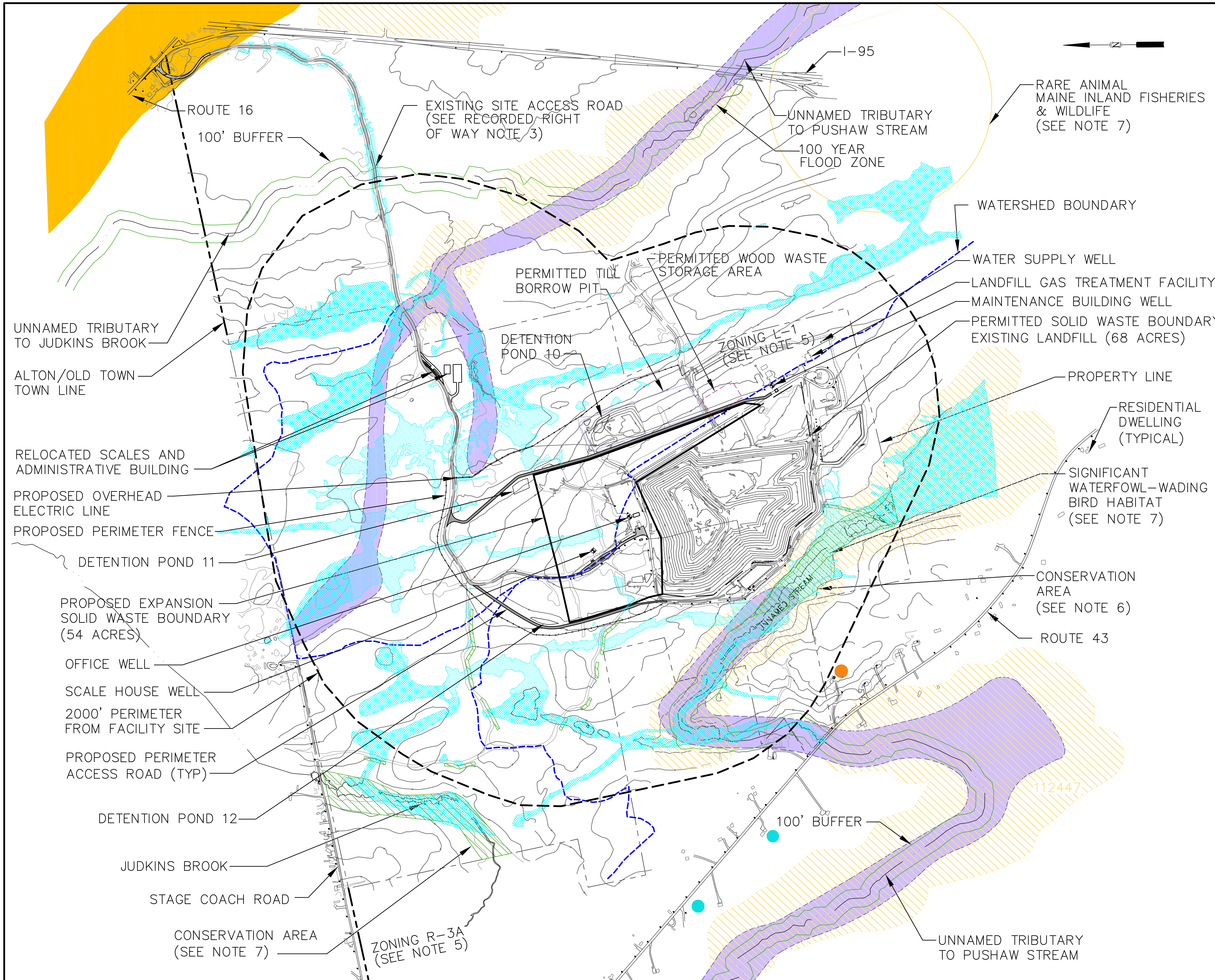
SME-1

VOLUMES I, II AND IV UPDATED FIGURES

- VOLUME I, APPENDIX M SITE SURROUNDINGS MAP
- VOLUME II, FIGURE 2-8 BEDROCK WELL YIELDS
- VOLUME II, FIGURE 3-2 PHOTOLINEAMENTS AND FRACTURE TRACES
- VOLUME II, FIGURE 6-1 PROPOSED MONITORING LOCATIONS FOR GROUNDWATER AND SURFACE WATER
- VOLUME IV, APPENDIX I FIGURE 3-1 ENVIRONMENTAL MONITORING LOCATIONS

VOLUME I

APPENDIX M
SITE SURROUNDINGS MAP



NOTES:

1. EXISTING GROUND CONTOURS FROM JULY 31, 2014 AND APRIL 17, 2004. AERIAL SURVEY PERFORMED BY AERIAL SURVEY AND PHOTO, INC. OF NORRIDGEWOCK, MAINE.

2. PROPERTY LINE LOCATIONS ARE A RESULT OF FIELD SURVEY PERFORMED BY HERRICK AND SALSBURY, INC. LAND SURVEYORS, ELLSWORTH, MAINE FOR TRYTON TREE FARM PROJECT, PATTEN CORPORATION-DOWNEAST, OLD TOWN, MAINE, FEBRUARY 23, 1988, REVISED APRIL 7, 1988.

3. RIGHT OF WAY FOR ALL PURPOSES OVER THE ACCESS ROAD (50) FOOT WIDE, AS DESCRIBED IN EXHIBIT A TO QUITCLAIM DEED WITH COVENANT (BOOK 9188, PAGE 154, #3751 – PENOBSCOT REGISTRY OF DEEDS).

4. THERE ARE NO HISTORICAL OR ARCHAEOLOGICAL SITE IDENTIFIED WITHIN THE 2000 FOOT PERIMETER FROM THE FACILITY SITE.

5. ZONING, AS DESCRIBED BY THE CITY OF OLD TOWN CODE OF ORDINANCES, FOR PROPERTY OWNED BY THE STATE OF MAINE IS L-1 (LANDFILL ZONE). ALL OTHER PROPERTY SURROUNDING THE SITE IS R-3A (RESIDENCE AND FARMING).

6. CONSERVATION AREAS SHOWN ARE WETLAND AREAS PREVIOUSLY PRESERVED AS DESCRIBED IN DECLARATIONS OF COVENANTS AND RESTRICTIONS BY JAMES RIVER PAPER COMPANY (REVISED PLAN AUGUST 10, 1995). LOCATIONS ARE APPROXIMATE.

7. THERE ARE NO RARE BOTANICAL FEATURES DOCUMENTED WITHIN THE PROJECT AREA BASED UPON A REVIEW OF THE NATURAL AREAS PROGRAM'S BIOLOGICAL AND CONSERVATION DATA SYSTEMS FILES BY THE MAIN DEPARTMENT OF AGRICULTURE AND FORESTRY (OCTOBER 7, 2014 CORRESPONDENCE).
THE MAINE DEPARTMENT OF INLAND FISHERIES AND WILDLIFE, WILDLIFE DIVISION, IDENTIFIED POTENTIAL SIGNIFICANT WATERFOWL –WADING BIRD HABITATS AND A WOOD TURTLE OBSERVATION BUFFER NEAR THE SITE (OCTOBER 6, 2014 AND NOVEMBER 5, 2014 CORRESPONDENCE). THERE HAVE BEEN NO OTHER UNIQUE AREAS IDENTIFIED WITHIN THE GENERAL VICINITY OF THE SITE.

8. THERE ARE NO INDUSTRIAL OR PUBLIC WATER SUPPLY WELLS, PUBLIC WATER SUPPLY WATERSHED AREAS, WELLHEAD PROTECTION AREAS OR SIGNIFICANT SAND AND GRAVEL AQUIFERS LOCATED WITHIN 2000 FEET OF THE PROPOSED EXPANSION. ALL RESIDENTIAL DWELLINGS SHOWN ARE ASSUMED TO HAVE A DOMESTIC WATER SUPPLY WELL.

LEGEND

MAINE INLAND FISHERIES AND WILDLIFE SIGNIFICANT WILDLIFE HABITAT SOURCE: NOVEMBER 5, 2014 CORRESPONDENCE WITH THE MAINE DEPARTMENT OF INLAND FISHERIES AND WILDLIFE

100 YEAR FLOOD ZONE (FEMA OLD TOWN QUAD PANEL NUMBER 2301120002A DATED APRIL 1978)

STANTEC WETLANDS (2004, 2008, 2014 AND 2015 DELINEATION)

GROUNDWATER YIELDS GREATER THAN 10 GPM SIGNIFICANT SAND & GRAVEL AQUIFER MGS SURVEY OPEN FILE 08-07 BY TOLMAN AND LANCTOT, 2008.

CONSERVATION AREAS

YIELD BETWEEN 5 AND 10 GALLONS PER MINUTE

YIELD BETWEEN 10 AND 15 GALLONS PER MINUTE

MAPPING SOURCE: MAINE GEOLOGICAL SURVEY WATER WELL DATABASE, DATED AUGUST 28, 2014.

250 0 500 1000 FEET

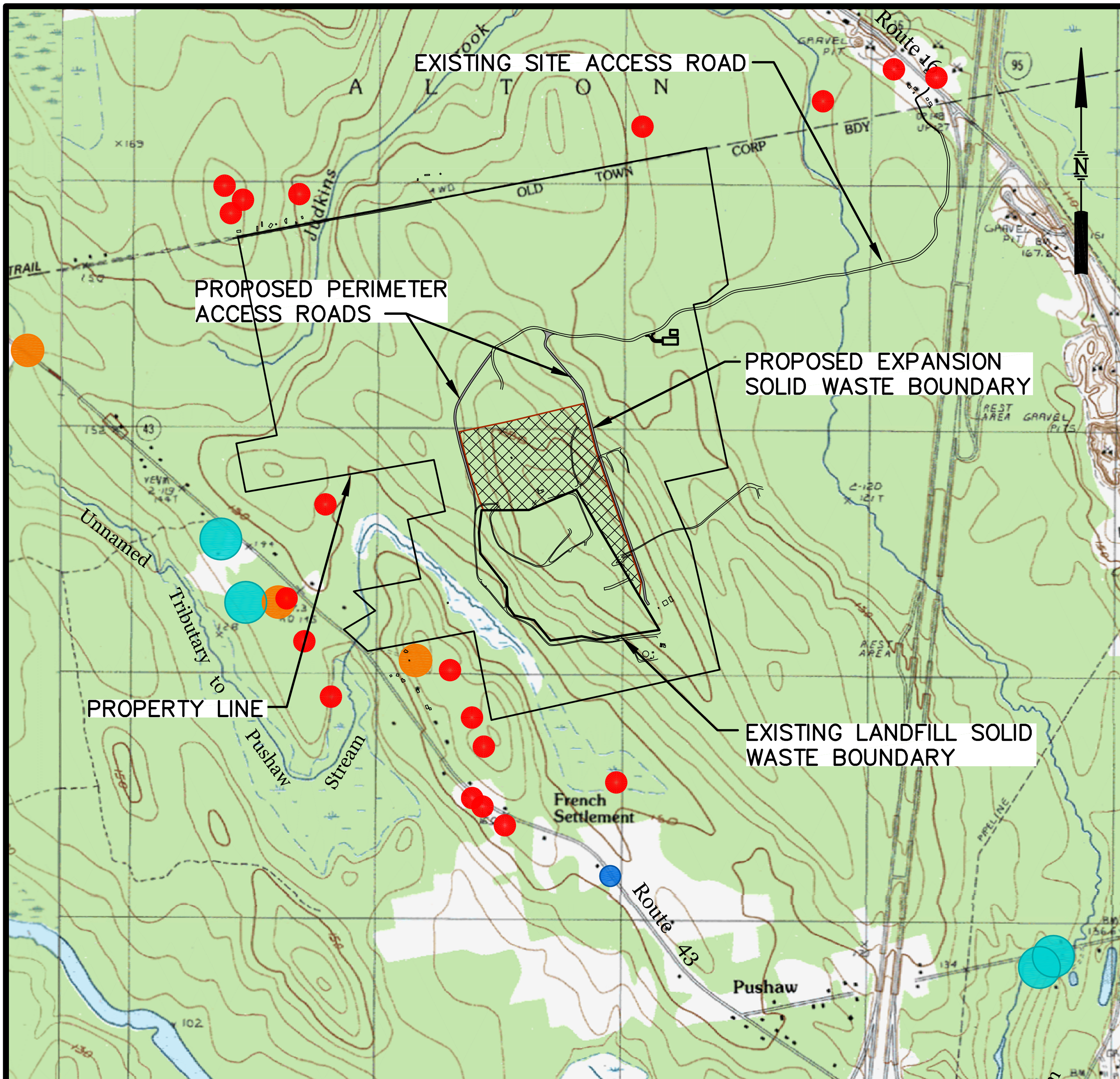
SITE SURROUNDINGS MAP
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DWG: BUFFERPLAN LMN: SITE Surroundings CTB: SiteSurroundings REV: 2/22/16

VOLUME II

FIGURE 2-8
BEDROCK WELL YIELDS



BASE MAP ADAPTED FROM 7.5 MIN USGS TOPOGRAPHIC QUADRANGLE OLD TOWN, MAINE–1988

MAPPING SOURCE

MAINE GEOLOGICAL SURVEY
WATER WELL DATABASE,
DATED AUGUST 28, 2014.

LEGEND

- REPORTED WELL YIELD LESS THAN 5 GALLONS PER MINUTE
- REPORTED WELL YIELD BETWEEN 5 AND 10 GALLONS PER MINUTE
- REPORTED WELL YIELD GREATER THAN 10 AND LESS THAN 20 GALLONS PER MINUTE
- WELLS SAMPLED BY SME IN 2004 (LOCATION APPROXIMATE)

1000 0 2000 FEET

FIGURE 2–8
BEDROCK WELL YIELDS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

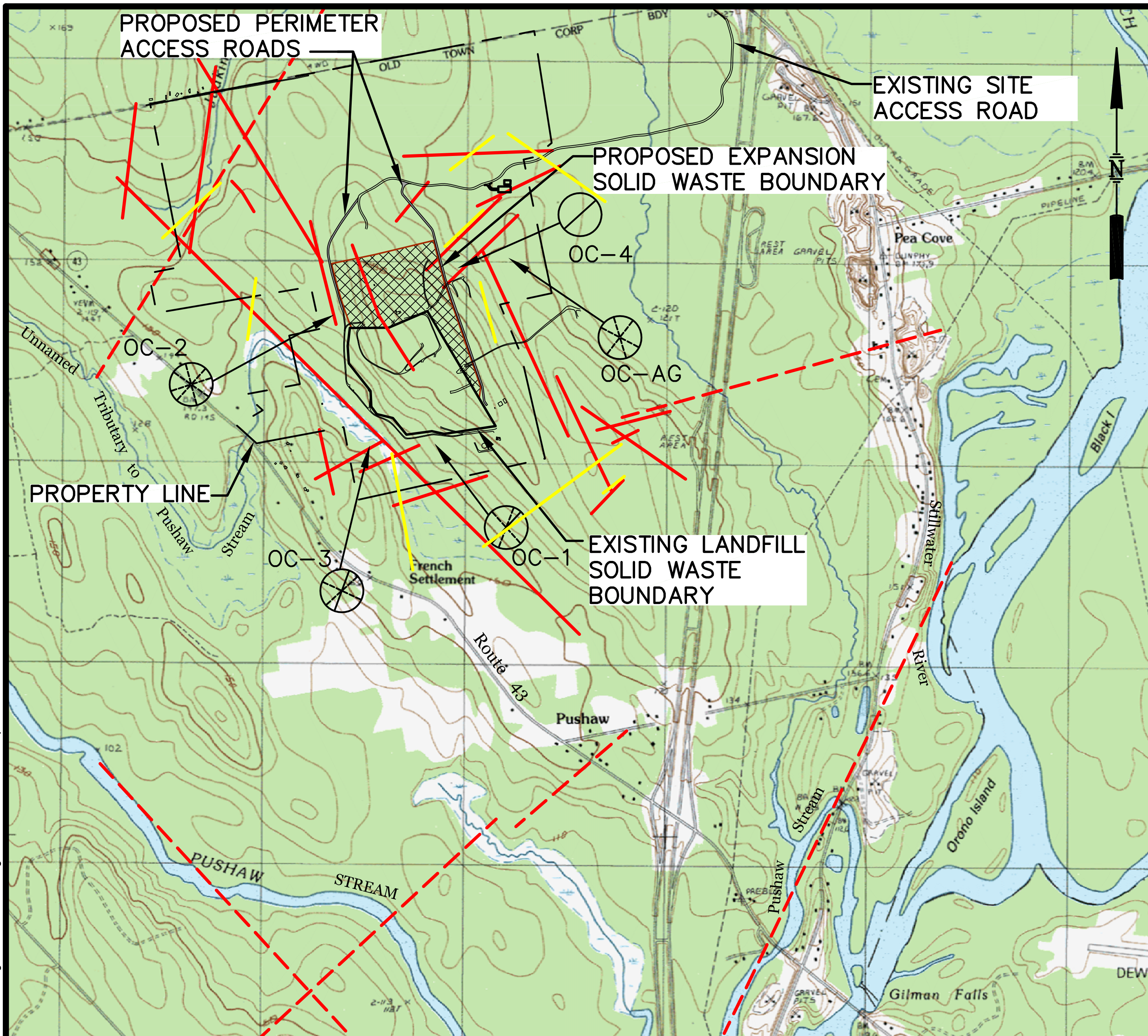
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: SITELOC LMN: BR–WELLYIELDS CTB: SITELOC REV: 2/11/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\SITELOC.dwg, 2/11/2016 10:14:19 AM, jrl

VOLUME II

FIGURE 3-2
PHOTOLINEAMENTS AND FRACTURE TRACES



BASE MAP ADAPTED FROM 7.5 MIN USGS TOPOGRAPHIC QUADRANGLE OLD TOWN, MAINE–1988

NOTES

LOCATIONS OF OUTCROPS ARE APPROXIMATE.

LEGEND

HIGH ALTITUDE PHOTOLINEAMENTS (CASWELL, EICHLER & HILL, 1986)

LOW ALTITUDE PHOTOLINEAMENTS (SEVEE AND MAHER 1991, UPDATED 2008)

PHOTOLINEAMENTS (SEVEE AND MAHER 2016) INTERPRETED FROM MAINE OFFICE OF GIS LIDAR MAP

BEDDING (———) AND FRACTURE (-----) ORIENTATIONS MEASURED AT OUTCROPS (LOCATION APPROXIMATE)

1250 0 2500 5000 FEET

FIGURE 3–2
PHOTOLINEAMENTS AND FRACTURE TRACES
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: SITELOC LMN:PHOTOLIN CTB:SITELOC REV: 3/3/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\SITELOC.dwg, 3/3/2016 8:46:11 AM, paf

VOLUME II

FIGURE 6-1
PROPOSED MONITORING LOCATIONS FOR GROUNDWATER AND SURFACE WATER

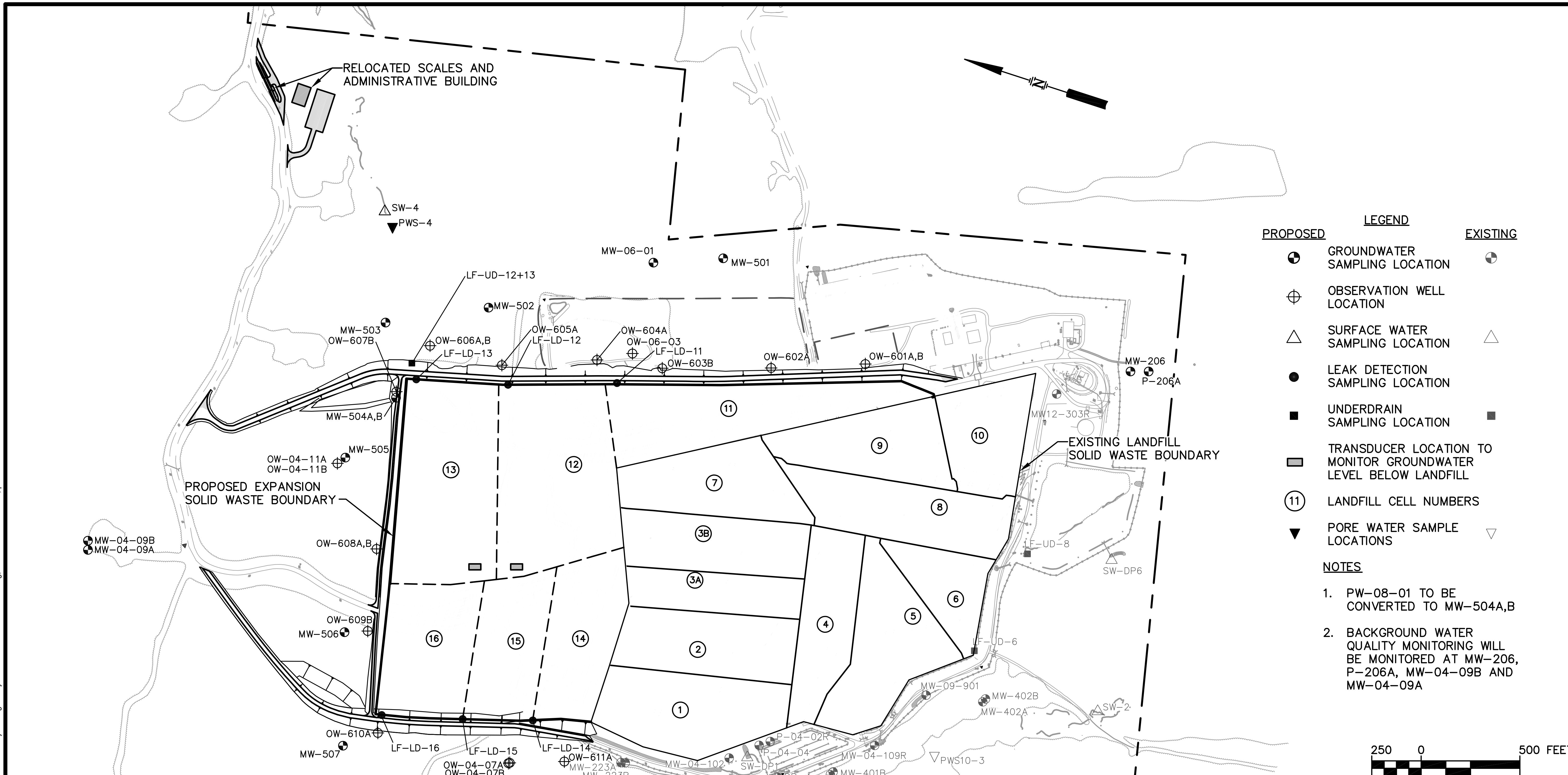


FIGURE 6-1
PROPOSED MONITORING LOCATIONS FOR GROUNDWATER AND SURFACE WATER
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: BASE LMN: FIG-SITEPLAN-MONITORING CTB: SME-STD REV: 2/22/2016

\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Plans\BASE.dwg, 2/22/2016 4:01:20 PM, paf

VOLUME IV

APPENDIX I FIGURE 3-1
ENVIRONMENTAL MONITORING LOCATIONS

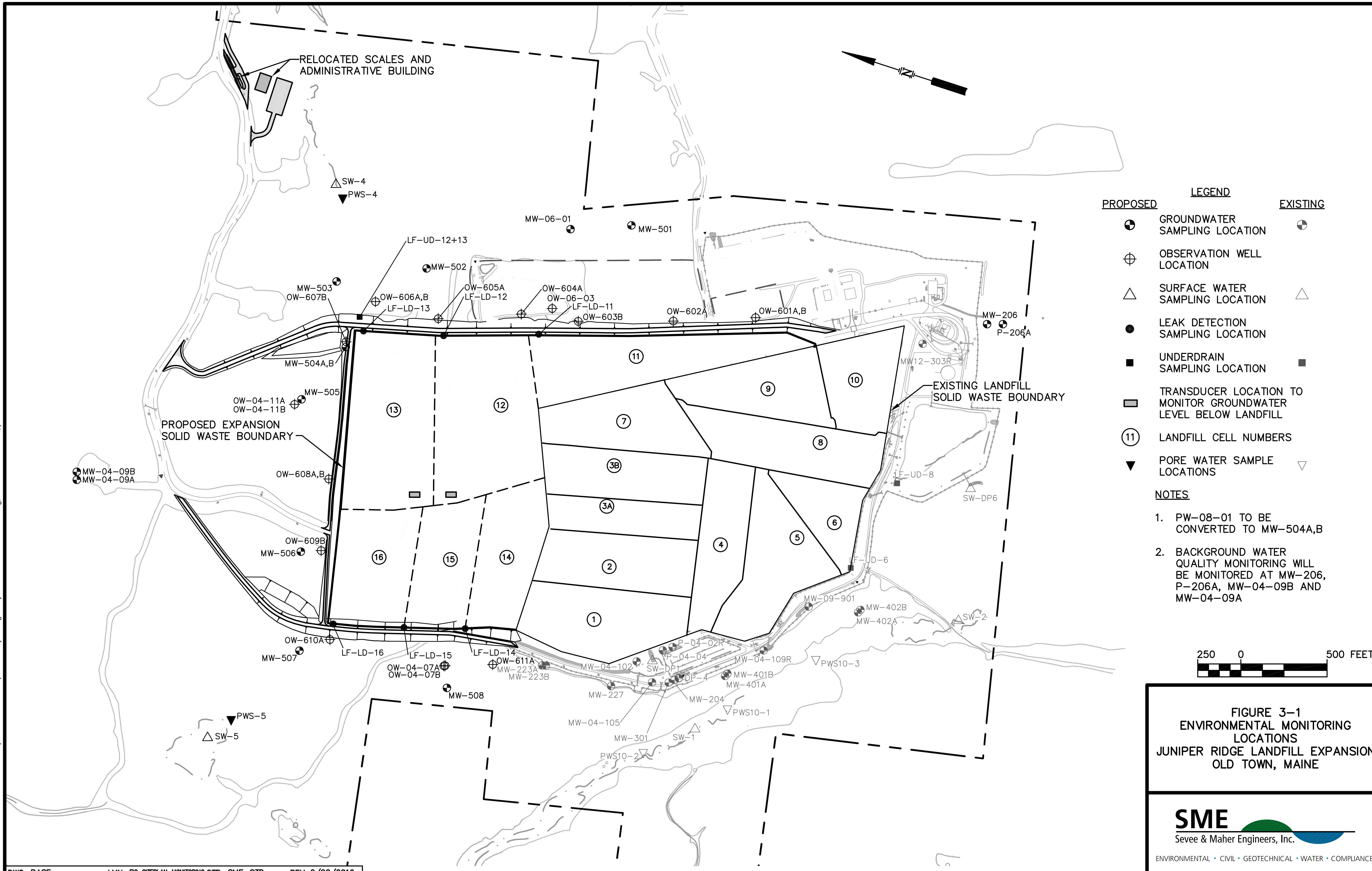


FIGURE 3-1
ENVIRONMENTAL MONITORING LOCATIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: BASE LMN: FIG-SITEPLAN-MONITORING CTB: SME-STD REV: 2/22/2016

\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Plans\BASE.dwg, 2/22/2016 3:39:48 PM, paf

SME-2

VOLUME II
WORK PLAN FOR REFINING LOCATION OF MONITORING WELLS AT THE JUNIPER RIDGE LANDFILL EXPANSION

WORK PLAN FOR
REFINING LOCATION OF MONITORING WELLS
AT THE
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

Prepared for

BUREAU OF GENERAL SERVICES
AND
NEWSME LANDFILL OPERATIONS, LLC

March 2016

STATE OF MAINE
SEVEE
3/2/16
GEOLOGIST

SME
Sevee & Maher Engineers, Inc.

ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

4 Blanchard Road, PO Box 85A, Cumberland Center, ME 04021 • Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

TABLE OF CONTENTS

WORK PLAN
FOR
REFINING LOCATION OF MONITORING WELLS AT THE
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

1.0 PURPOSE

The purpose of this Work Plan is to present an approach for refining/finalizing the locations for new groundwater monitoring wells around the perimeter of the JRL Expansion for operational and long-term monitoring of the landfill. The Environmental Monitoring Plan, includes establishment of a total of 45 monitoring locations consisting of: (1) background and downgradient piezometers and wells; (2) additional surface water and pore water sampling points; and (3) leak detection and underdrain monitoring points. The proposed monitoring locations associated with the Expansion are as shown on Figure 6-1 of Volume II of the Application. Since the Expansion will be developed in a series of cells beginning in 2018 with the construction of Cell 11, and continuing for a period of about 12 years, the installation of the monitoring wells included in the monitoring program will be phased as landfill development proceeds as proposed in the Application. However, in discussions with DEP, we agreed that a work plan outlining an approach to refine the locations for the proposed monitoring wells should be provided as part of the Expansion application, to obtain DEP approval prior to beginning field work.

During the development of this work plan, and in discussions with DEP, we agreed that there would be an advantage to gathering additional data now to confirm geologic features identified during the site assessment that will be relevant to siting the individual wells. This will be, therefore, a refinement of the information already submitted in the Application.

Thus, we have prepared a staged approach to gather this data as described in this work plan, with some additional data being collected in the near-term (i.e., spring 2016), and with the input of DEP[1], to help plan for what data may be necessary for final siting of the monitoring wells. This approach will help to fine-tune the geologic data that already exists for the Expansion site, which, in turn, will help to guide the eventual siting process for the monitoring wells needed prior to operation of the Expansion.

[1] MEDEP (Mr. Richard S. Behr) provided comments on the Draft Work Plan for Refining Location of Monitoring Wells at the Juniper Ridge Landfill Expansion Old Town, Maine, in a memorandum dated February 25, 2016. Those comments have been incorporated into this Work Plan.

2.0 APPROACH

The investigations conducted as part of the Expansion Application and documented in Volume II - Site Assessment Report show that the bedrock at the site consists of fractured metasediments, which are typical of this area of Maine. The investigations found that the bedrock fracturing is on the scale of inches to a few feet. The borehole and surficial geophysical surveys completed onsite demonstrated that there are also localized, more densely fractured zones within the bedrock.

Information to be collected during execution of this Work Plan will supplement the available geologic data and be used to inform placement of the proposed observations and monitoring wells outside the perimeter of the Expansion. In part, this work will help to ensure more densely fractured zones have not been overlooked in siting the observation and monitoring wells. The data will be used to establish the final well locations and the screen depths within the bedrock. This Work Plan utilizes the same methodologies utilized during the previously completed site investigations, which has demonstrated that the site meets the requirements contained in DEP Chapter 401 for landfill siting, design and operations.

Supplemental geophysical survey work is included in this Work Plan, as is installation of boreholes into the bedrock to confirm the geophysical and photolineament studies already completed. Each new borehole, as well as two existing boreholes (i.e., the water supply wells for the office and scale house) within the footprint of the Expansion, will be examined using geophysical borehole logging methods to establish fracture depths and possible fracture continuity between boreholes using surficial geophysical methods. Boreholes will be drilled within the Expansion footprint and along the Expansion's perimeter. Boreholes that do not become part of the groundwater monitoring plan will be decommissioned and sealed with grout. The outcome of this supplemental data gathering program will be the basis to refine the Expansion's groundwater monitoring system.

The work plan has been subdivided into two parts: (1) an early phase - Phase 1- which would be done now, and (2) a later phase - Phase 2- that would be done at least one year before the beginning of Expansion development (i.e., Cell 11), or tentatively during the summer of 2017 providing all required project permits are obtained. The Work Plan is designed to be completed in close cooperation with DEP, to streamline decision-making.

3.0 SCOPE OF WORK

3.1 Phase 1-Background Information for Planning and Confirmation

The purpose of Phase 1 is to collect data for planning and confirmation.

Task 1-Downhole Geophysical Survey of Existing Water Supply Wells. Task 1 of Phase 1 includes conducting downhole geophysical surveys of two existing water supply wells within the footprint of the Expansion. The pumps will be removed from the existing two water supply wells (i.e., the scale house and office) at least one day before the geophysical survey begins. Each well will be logged with a suite of downhole geophysical instruments to examine bedrock fracture locations, sizes, orientations and fracture water yield. The geophysical logging parameters are listed in Table 1, along with a brief explanation of the logging objective relative to identification of bedrock fractures.

Borehole diameter and fracture width data from caliper logs will be used to make preliminary estimates of fracture depths with the potential for water flow. Fluid resistivity and temperature are often useful in identifying zones where groundwater is seeping into the borehole. Vertical flow measurements between transmissive fractures can be evaluated with a heat-pulse flowmeter. Ambient and induced groundwater flows from fractures will also be measured using the downhole flowmeter. The acoustic and optical televiewer data will be used to identify planar features (e.g., fractures, joints, bedding, and foliation) that intercept the borehole wall and measure their strikes and dips. Results from the downhole geophysical logging will be plotted as stereo nets, rose diagrams and an image of the borehole wall. The strike and dip data along with fracture width, will provide a qualitative sense of hydraulic conductivity anisotropy in the bedrock. The borehole fracture orientations will be compared with those previously measured at bedrock outcrops, bedrock cores, and existing downhole geophysical studies performed for the Expansion application. The geophysical survey will be conducted by Northeast Geophysical Services (NGS) of Bangor, Maine.

TABLE 1

BOREHOLE GEOPHYSICAL PARAMETERS

Instrument/Parameter	Objective
Caliper (Borehole Diameter)	Fractures are often indicated by widenings along the borehole wall.
Fluid Temperature	Changes in fluid temperature can indicate water entering a borehole through fractures.
Fluid Resistivity	Changes in fluid resistivity can indicate water entering the borehole through transmissive fractures.
Single Point Resistance	Electrical resistance between instrument and a surface electrode. Water-filled fractures often are characterized by low resistance.
Spontaneous Potential (SP)	Electrical voltage between the instrument and a surface electrode. SP sources can include lithologic changes and water movement in or out of a borehole through fractures.
Gamma	Provides lithologic/formation information. Clay-filled fractures can be characterized by gamma spikes.
Acoustic Televiewer	Oriented acoustical image of the borehole wall, including identification of strike and dip directions of planar features such as fractures and foliation.
Optical Televiewer	Oriented optical image of the borehole wall, including identification of strike and dip directions of planar features.
Heat-Pulse Flowmeter	Measures the vertical flow of water in the borehole, under ambient and pumping (stressed) conditions. Vertical flow indicates two or more transmissive fractures intersecting the borehole, at hydraulic disequilibrium.

Task 2-Borehole Drilling Within Expansion Footprint. Task 2 is to conduct additional borehole drilling within the footprint of the Expansion site. There are several geologic features along the east side of the Expansion that may be appropriate locations for monitoring wells. Three new boreholes (B16-101 through B16-103) within the Expansion footprint would be useful in finalizing the later elements of this work plan. Therefore, the three boreholes would be drilled at the approximate locations shown on the attached Figure 1 within the eastern side of the Expansion footprint. Two of these locations (B16-101 and B16-102) have been proposed along the alignments of previously identified photolineaments and should help resolve their importance for monitoring. Prior to drilling, the locations of existing photolineaments and denser fracture zones in the bedrock will be located in the field from the existing mapping. The intent is to drill along these features (accounting for the interpreted dip of the bedrock structures). A third borehole (B16-103) will be drilled within the footprint in an area not aligned with a photolineament to provide a point to compare the bedrock structure to that investigated with the other two boreholes. The approximate locations of these boreholes are shown on the attached Figure 1.

All three of the boreholes in Task 2 will extend at least 200 feet below the bedrock surface and will be drilled using air-rotary methodology. The soil overburden will be cased during advancement of the borehole into the bedrock. Soil and rock cuttings will be spread around each borehole. SME will observe the drilling and will classify rock chip samples obtained from the boreholes. Soil and rock cuttings will be spread around each borehole.

Each borehole will be developed by pumping and/or surging techniques to remove fine-grained sediments after the completion of drilling. The recovery rate of water levels will be recorded to estimate the borehole water yield. Static water levels in each boring will be recorded after levels have stabilized.

Task 3-Downhole Geophysical Survey of New Boreholes. Task 3 of Phase 1 is to conduct downhole geophysical surveys of each of the new boreholes. Each of the three boreholes described in Task 2 will be logged with the same downhole equipment and methodologies as described in Task 1 to examine structure locations, sizes, orientations and fracture water yield.

Task 4-Data Compilation and Review. Task 4 will be data compilation and review of the information gathered in Tasks 1 through 3. DEP will be notified of the specific schedule for the various work elements of Phase 1 and will be kept abreast of the results of the investigations. The data compiled from the investigations will be reviewed with DEP and it is anticipated at least one meeting with DEP will be held to review the results of the Phase 1 investigations. The results of the investigations will be reviewed in terms of (1) the voluminous existing data; (2) the understanding of both the bedrock depth and structural features, as they relate to locating, both horizontally and vertically, zones to be screened for the Expansion's monitoring wells; and (3) the interpretation of the groundwater flow paths beneath the Expansion footprint. These findings will be presented in a written report to supplement the information contained in the Expansion application. The report will include borehole logs; the geophysical report; survey data, a map showing the locations of the Phase 1 boreholes; and a summary of the supplemental field investigation work. Any appropriate refinements to the Phase 2 program, discussed below, will also be included. The schedule for completing Phase 1 is discussed in Section 4.0 of this Plan, below.

3.2 Phase 2-Locating Monitoring Wells

The objective of the Phase 2 program is to optimally locate the Expansion's observation and monitoring wells.

Task 5-Electrical Earth Resistivity. Task 5 involves performing earth resistivity transects as part of Phase 2 of this Plan. Electrical earth resistivity (resistivity) transects will be completed to supplement existing resistivity transects, as shown on Figure 1. This will include running one transect (Line S-1, on Figure 1) along the northern boundary of the Expansion and a second along the western boundary of the Expansion (Line S-2). Transect S-2 parallels existing Line 9 and will be used to help determine the azimuths of potentially interconnected anomalies, to aid in locating the well west of the Expansion. A third transect will parallel the eastern side of the Expansion (Line S-3) and pass through proposed monitoring locations OW-604A, OW-605A, and OW-06-03. Transect S-3 parallels existing Line 6 and will be used to help determine the azimuths of potentially interconnected anomalies, to aid in locating the well east of the Expansion. Finally, two transects will pass through the Expansion area, one with a northwest-southeast orientation (Line S-4) and one east-west (Line S-5). Line S-4 will pass nearby to two existing water supply well casings, which should not significantly impact the bedrock 2-D resistivity results since the casings are vertical and do not penetrate significantly into the bedrock. Line S-5 will pass through two of the proposed bedrock borehole explorations (B16-101 and B16-102) installed as part of Phase I. Over-head electrical lines in the vicinity of the scale house, office and access roadway to the highway may locally interfere with the resistivity transects.

The purpose of the resistivity transects is to further refine information from previous investigations on fracture zones in the bedrock, which will provide information necessary for optimally locating new Expansion observation and monitoring wells. The earth resistivity results will also provide additional data on the soil overburden thickness. The earth resistivity transects will be "calibrated" by passing them over existing site borings that extend beneath the bedrock surface. The preliminary locations of these transects are shown in Figure 1, pending DEP review. This resistivity work will be done in close coordination with DEP. The earth resistivity survey will be conducted by NGS.

Task 6-Additional Borehole Drilling. Task 6 of the Plan requires additional borehole drilling. Based on the results of the geophysical surveys and preliminary boreholes described in Phase 1, the six proposed monitoring boreholes (OW-602A, OW-605A, OW-606 A&B, OW-608A&B, MW-507 and OW-611A) will be drilled using the air-rotary hammer technique. The boreholes will be located outside of and along the northern (one), eastern (three) and western (two) boundaries of the Expansion. The approximate locations of these boreholes are as presented in the Expansion application, and are shown on Figure 1. The locations and depths of these wells will be finalized after the Phase 1 data has been analyzed. One of the boreholes will be intentionally located on a bedrock zone that indicates a relatively lower fracture density to aid in confirming and calibrating the earth resistivity survey data. Furthermore, prior to the beginning of drilling, SME and DEP will finalize the borehole locations and depths.

The new boreholes will allow access for downhole geophysical logging tools to the presence of fractures or fracture zones identified by the earth resistivity transects and photolineaments. The boreholes will be nominally six inches in diameter and drilled a minimum of 200 feet deep into bedrock. The soil overburden will be cased during advancement of the borehole through the bedrock. Soil and rock cuttings will be spread around each borehole. Rock chips will be visually logged.

Each borehole will be developed after the completion of drilling. The recovery rate of water levels will be recorded to estimate the borehole water yield. Static water levels in each boring will be recorded after levels have stabilized.

Site preparation for drilling will include clearing of brush and trees, and construction of access roads sufficient for a three-axle, water-well-style drill rig, support trucks, and equipment. Erosion control at these drilling locations will include installation of silt fencing between work areas and surface water streams (if any).

Task 7-Downhole Geophysical Survey. Task 7 will involve a downhole geophysical survey. Each of the six boreholes drilled in Task 6 will be logged with the same downhole logging probes utilized in Task 1 to examine fracture locations, sizes, orientations and fracture water yield.

Task 8-Location Survey. Task 8 of the Plan is to conduct a location survey. Once the boreholes and geophysical transects are completed, their horizontal and vertical locations will be measured by survey. Horizontal locations will be measured to the nearest one-foot and vertical locations measured to the nearest 0.1 foot.

Task 9-Data Review and Monitoring Well Identification. Task 9 will involve final data review and monitoring well identification. Once the Phase 2 field work is complete, the results of Tasks 1 through 8 will be provided to DEP in a summary report documenting what was done, how it was done, and the purpose of each Task performed. The collected information will be used to finalize the overall depth, location, and screen length for the Expansion's observation and monitoring wells, in cooperation with DEP. Available mapping provided in the Site Assessment Report will be updated to show the new boreholes and geophysical transects. The submittal will include the NGS report and logs for the boreholes. Groundwater elevations will be measured at the new boreholes and compared to those of existing surrounding wells and piezometers. Bedrock depth and fracture patterns will be compared with existing data. The report will include a description of the field work and an interpretation of the findings. The information gathered will be used to support SME's recommendations for final monitoring and observation well placement, design and construction. Well placement will focus on transmissive zones in the bedrock that can conduct groundwater from beneath the Expansion to its perimeter. DEP will approve each well location and screened interval, prior to installation.

Once the locations and designs of the monitoring wells are complete, they will be installed at least one year before the construction of the adjacent individual Expansion cells are complete. Attachment 1 contains a revised Table 3-1 from the Expansion's Environmental Monitoring Plan, contained in Volume IV of the Application. The revised Table 3-1 includes the tentative installation schedule for the proposed site monitoring wells. Once the wells are installed and have a chance to equilibrate with the adjacent formation, they will be sampled for at least four rounds to establish pre-Expansion water quality. Boreholes, piezometers, and wells within the

Expansion footprint will be grouted to eliminate open holes through the glacial till into the bedrock.

TABLE 3-1

EXPANSION GROUNDWATER MONITORING WELLS
(Revised 2/2016)

Landfill Expansion Boundary	Monitoring Well	Geologic Unit Screened	Tentative Screen Interval Depth[2] (feet bgs)	Tentative Installation Schedule[3]
Background	MW-206	Overburden	15 - 20	Presently Installed
Background	P-206A	Bedrock	85.5 - 90.5	Presently Installed
Background	MW-04-09A	Shallow Bedrock	36 - 39	One year before Cell 12 constr.
Background	MW-04-09B	Overburden	13 - 15.5	One year before Cell 12 constr.
Eastern	MW-501	Shallow Bedrock	57 - 67	One year before Cell 11 constr.
	MW-06-01	Overburden	10 - 20	Presently Installed
	MW-502	Bedrock	36 - 46	One year before Cell 12 constr.
	MW-503	Bedrock	65 - 75	One year before Cell 13 constr.
	OW-601A	Bedrock	88 - 98	One year before Cell 11 constr.
	OW-601B	Overburden	51 - 61	One year before Cell 11 constr.
	OW-602A	Bedrock	52 - 62	**Phase 2**
	OW-603B	Overburden	34 - 44	One year before Cell 11 constr.
	OW-604A	Bedrock	39 - 49	One year before Cell 11 constr.
	OW-605A	Bedrock	32 - 42	**Phase 2**
	OW-606A	Bedrock	44 - 54	**Phase 2**
	OW-606B	Overburden	7 - 17	One year before Cell 13 constr.
	OW-06-03	Overburden	10 - 15	One year before Cell 11 constr.
Northern	MW-504A	Bedrock	117 - 127	One year before Cell 13 constr.
	MW-504B	Bedrock	69 - 79	One year before Cell 13 constr.
	MW-505	Bedrock	76 - 86	One year before Cell 13 constr.
	MW-506	Bedrock	55 - 65	One year before Cell 13 constr.
	OW-607B	Overburden	61 - 71	One year before Cell 13 constr.
	OW-608A	Bedrock	69 - 79	**Phase 2**
	OW-608B	Overburden	32 - 42	One year before Cell 13 constr.
	OW-609B	Overburden	19 - 29	One year before Cell 13 constr.
	OW-04-11A	Overburden	48 - 49	One year before Cell 13 constr.
	OW-04-11B	Overburden	9 - 10	One year before Cell 13 constr.

Landfill Expansion Boundary	Monitoring Well	Geologic Unit Screened	Tentative Screen Interval Depth[2] (feet bgs)	Tentative Installation Schedule[3]
Western	MW-507	Bedrock	33 - 43	**Phase 2**
	MW-508	Bedrock	40 - 50	One year before Cell 14 constr.
	OW-610A	Bedrock	27 - 37	One year before Cell 14 constr.
	OW-611A	Bedrock	31 - 41	**Phase 2**
	OW-04-07A	Bedrock	73 - 83	One year before Cell 14 constr.
	OW-04-07B	Bedrock	24.5 - 25.5	One year before Cell 14 constr.

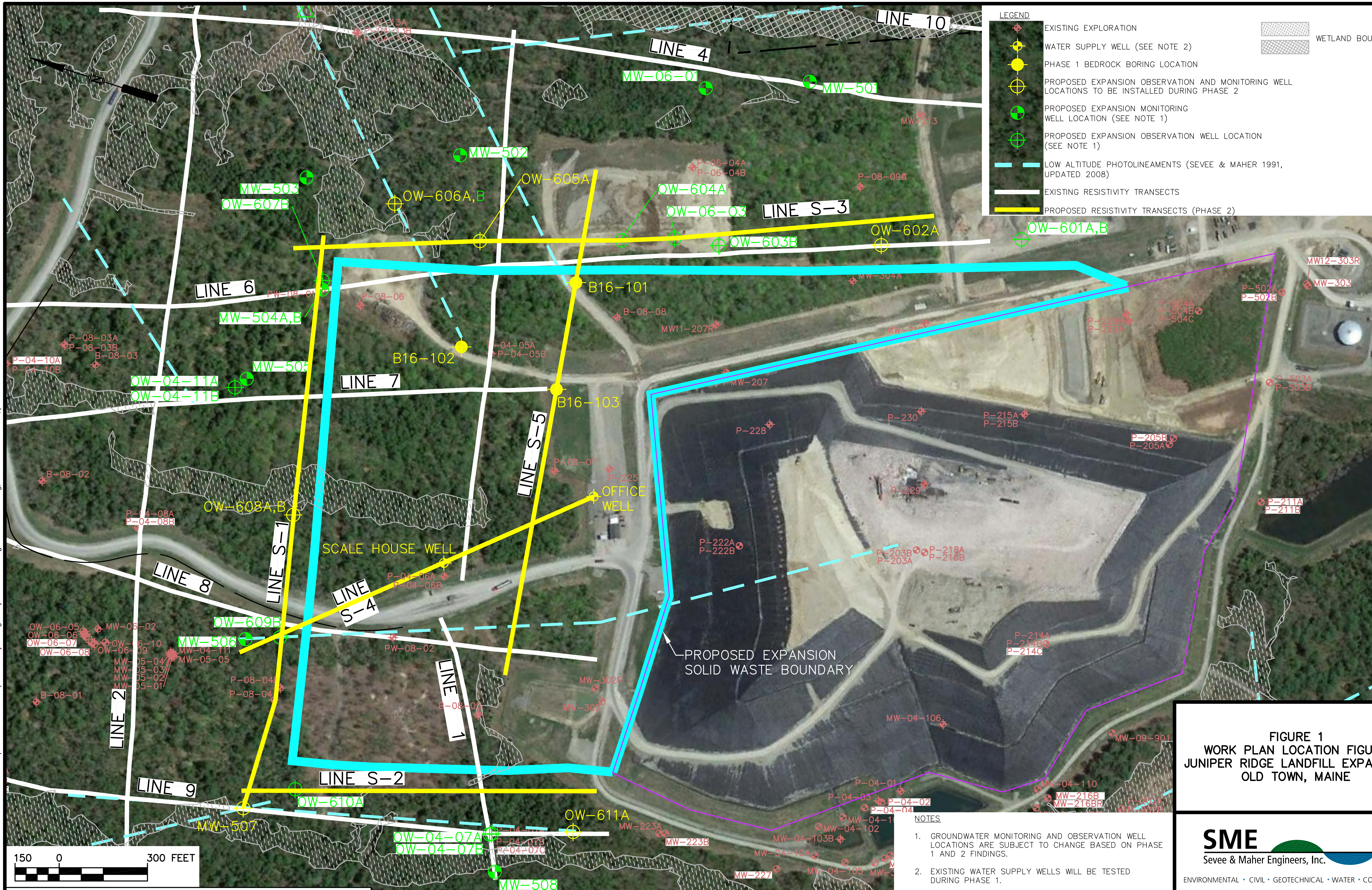
<u>Notes</u>:

1. Well screen intervals for new wells and piezometers are preliminary and based on: site lithology; 10-foot long screens; overburden screens are two feet above bedrock; and bedrock screens are 25 to 35 feet below bedrock surface.
2. Well Depths identified as being installed in Phase II will be drilled to a depth of 200 feet. Screen intervals will be determined based on Phase 1 investigation.
3. Bedrock wells installed during Phase 2 of the work plan for refining the location of the monitoring wells will be installed tentatively during the summer of 2017 provided the Expansion Application has received all require approvals.

4.0 SCHEDULE

The downhole geophysical logging of the existing two water supply wells, drilling of the three preliminary boreholes and the downhole geophysics in Phase 1 are scheduled to be initiated in March of 2016. It is expected to take up to two months to coordinate access, water pump removal and replacement, drill the boreholes, and get the data report from NGS. It is expected that our report to DEP will be submitted in May 2016. Weather and driller availability may affect this schedule.

For Phase 2, the resistivity survey will require about one week to clear the transects and up to two weeks to complete the field work. This work is scheduled for the summer of 2017, after the Expansion application is approved. Once started, the results should be available in near real-time for review with DEP. The borehole drilling will take about two to three days per location once access is provided. Access may take some time to complete since most of the boreholes are away from existing roads in heavily wooded areas and the potential impacts on habitat will need to be considered. Clearing and road building for the drilling may take a few weeks but could be on-going during the earth resistivity field work and the start of drilling. Downhole geophysics can be scheduled as soon as the wells have had a chance to rest for one or two weeks. It is not uncommon to complete the downhole work at a rate of two boreholes per day. Phase 2 may require up to four to six months to complete.



LEGEND

- EXISTING EXPLORATION
- WATER SUPPLY WELL (SEE NOTE 2)
- PHASE 1 BEDROCK BORING LOCATION
- PROPOSED EXPANSION OBSERVATION AND MONITORING WELL LOCATIONS TO BE INSTALLED DURING PHASE 2
- PROPOSED EXPANSION MONITORING WELL LOCATION (SEE NOTE 1)
- PROPOSED EXPANSION OBSERVATION WELL LOCATION (SEE NOTE 1)
- LOW ALTITUDE PHOTOLINEAMENTS (SEVEE & MAHER 1991, UPDATED 2008)
- EXISTING RESISTIVITY TRANSECTS
- PROPOSED RESISTIVITY TRANSECTS (PHASE 2)
- WETLAND BOUNDARIES

NOTES

1. GROUNDWATER MONITORING AND OBSERVATION WELL LOCATIONS ARE SUBJECT TO CHANGE BASED ON PHASE 1 AND 2 FINDINGS.
2. EXISTING WATER SUPPLY WELLS WILL BE TESTED DURING PHASE 1.

FIGURE 1
WORK PLAN LOCATION FIGURE
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: FIGURES LMN: SITEPLAN-AERIAL CTB: SME-STD REV: 2/26/2016

\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\FIGURES.dwg, 2/26/2016 1:54:11 PM, paf

SME-3

VOLUME II APPENDICES, SUPPLEMENTAL INFORMATION AND FIGURES

- APPENDIX M, SUMMARY OF INSTRUMENTED WELLS AND PIEZOMETERS DURING PUMPING TESTS
- APPENDIX R, TEST METHODOLOGY FOR BASAL TILL LABORATORY COLUMN TRACER TEST.
- FIGURE 4-3 ENLARGED PHOTOGRAPHS
- APPENDIX D, GROUNDWATER TREND PLOTS AT REPRESENTATIVE LOCATIONS.
- APPENDIX X, TIME OF TRAVEL SCHEMATICS
- APPENDIX X, TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS
- APPENDIX I, FIELD DATA FORMS
- APPENDIX U, TABULATED DATA USED TO PRODUCE FIGURE U-16
- APPENDIX V, FIGURE V-5S GROUNDWATER PARTICLE PATHWAYS FOR MODEL CALIBRATION
- APPENDIX V, FIGURE V-6S GROUNDWATER TABLE WITH RECHARGE CUTOFF

APPENDIX M

SUMMARY OF INSTRUMENTED WELLS AND PIEZOMETERS DURING PUMPING TESTS

Summary of Instrumented Wells and Piezometers During Pumping Tests

	Section 3.2.13 Fracture Interconnectivity Pumping Tests									
Pumping Well:	PW-08-01		PW-08-04		PW-08-03		PW-08-02		PW-08-01 & PW-08-02	
Test Start:	1/29/09 13:00		2/2/09 12:15		2/5/09 13:00		3/17/09 14:00		3/23/09 12:30	
	Water Levels were Measured at Each Location, Using Either a Transducer or Manually, as Indicated With an "X"									
Location	Transducer	Manually	Transducer	Manually	Transducer	Manually	Transducer	Manually	Transducer	Manually
MW-04-111		X		X		X				
MW-05-01		X		X		X				
MW-05-02		X		X		X				
MW-05-03		X		X		X				
MW-05-04		X		X		X		X		X
MW-05-05		X		X		X				
MW-06-01		X	X	X	X	X	X	X	X	X
MW-06-02	X	X	X	X	X	X	X	X	X	X
MW-207		X	X	X	X	X		X		X
MW-223A	X	X	X	X	X	X	X	X	X	X
MW-223B		X		X		X		X		X
MW-227		X		X		X		X		X
MW-302R	X	X	X	X	X	X		X		X
MW-304A	X	X	X	X	X	X	X		X	X
OW-06-05		X		X		X		X		X
OW-06-06		X		X		X		X		X
OW-06-07		X		X		X		X		X
OW-06-08		X		X		X		X		X
OW-06-09		X		X		X		X		X
OW-06-10		X		X		X		X		X
P-04-05A		X		X		X		X		X
P-04-05B		X		X		X		X		X
P-04-06A	X	X	X	X	X	X	X	X	X	X
P-04-06B		X		X		X		X		X
P-04-07A	X	X	X	X	X	X	X	X	X	X
P-04-07B	X	X		X	X	X	X	X	X	X
P-04-07C		X		X		X		X		X
P-04-08A		X		X		X		X		X
P-04-08B		X		X		X		X		X
P-04-09A	X	X	X	X	X	X		X		X
P-04-09B		X		X		X		X		X
P-04-10A	X	X	X	X	X	X		X		X
P-04-10B		X		X		X				X
P-04-11A	X	X	X	X	X	X	X	X	X	X
P-04-11B		X		X		X				X
P-04-12A		X		X	X	X	X	X	X	X
P-04-12B		X	X		X			X		X
P-04-12C				X		X		X		X
P-04-13A		X	X	X	X	X	X	X	X	X
P-04-13B	X	X	X	X	X	X		X		X
P-04-13C		X		X		X		X		X
P-04-14A	X	X	X	X	X	X	X		X	X
P-04-14B	X	X	X	X	X	X	X		X	X
P-06-04A		X		X		X		X		X
P-06-04B		X		X		X		X		X
P-08-03A		X		X		X				X
P-08-03B		X		X		X				X
P-08-04		X		X		X				X
P-08-06		X		X		X		X		X
P-08-07		X		X		X		X		X
P-08-09A										X
P-08-09B										X
P-08-09C										X
P-08-10A										X
P-08-10B										X
P-08-10C										X
P-213A										X
P-213B										X
PW-08-01	X	X	X	X	X	X	X	X	X	X
PW-08-02	X	X	X	X	X	X	X	X	X	X
PW-08-03	X	X	X	X	X	X	X	X	X	X
PW-08-04	X	X	X	X	X	X	X	X	X	X
Scale House							X	X	X	X
Office Supply								X	X	X

Response to MEDEP comments, page 3-24 section 3.2.13

APPENDIX R

TEST METHODOLOGY FOR BASAL TILL LABORATORY COLUMN TRACER TEST

TEST METHODOLOGY USED FOR BASAL TILL LABORATORY COLUMN TRACER TEST

1. Determine the sample properties, including specific gravity and dry density.

2. Re-compact soil to approximately match typical site conditions, place in triaxial cell and pressurize overnight to saturate.

3. Check saturation (B-value), apply a hydraulic pressure gradient of 2psi across the sample; and monitor hydraulic conductivity for consistency. Maintain cell and head pressures and close valves to isolate the sample.

4. Install a Bladder Accumulator, which contains a solution of 4,076 ppm Sodium Bromide (permeant), on the influent end of the sample.

5. Install (2) 24” lengths of sample tubing in series at the effluent end of the sample cell, which will allow for sample collection of un-altered effluent. Determine tubing volume. Quick disconnecting shutoff fittings were used at each end of the sampling tubes to maintain pressure and eliminate the loss of fluid when disconnected. Install another bladder accumulator on the effluent side of the sample tube chain, which contains Distilled De-ionized water (DI water).

6. Re-pressurize the sample and apply the 2 psi hydraulic pressure across the sample. At this time Sodium Bromide solution begins to enter the influent end of the sample cell.

7. After the tracer solution has been introduced, sampling occurs at approx. every 8-12 hours. The sampling tube closest to the sample is removed from the apparatus and drained into a vial, then rinsed and refilled using DI water and re-installed into the apparatus down gradient of the other sampling port.

8. The effluent sample in the vial is measured and diluted with DI water up to the 15ml level. That solution is then tested using HACH titration test kits for Chloride. Different test kit ranges should be used to define the full range of Sodium Bromide concentrations up to the influent concentration. (Note, for the JRL sample, three kits were used.)

9. Repeat sampling until the effluent concentration stabilizes at the influent concentration. (Note, for the JRL sample this took approximately 15 days, and resulted in passing approximately 2.2 pore volumes through the sample.

FIGURE 4-3
ENLARGED PHOTOGRAPHS

Figure 4-3 Enlarged Photographs

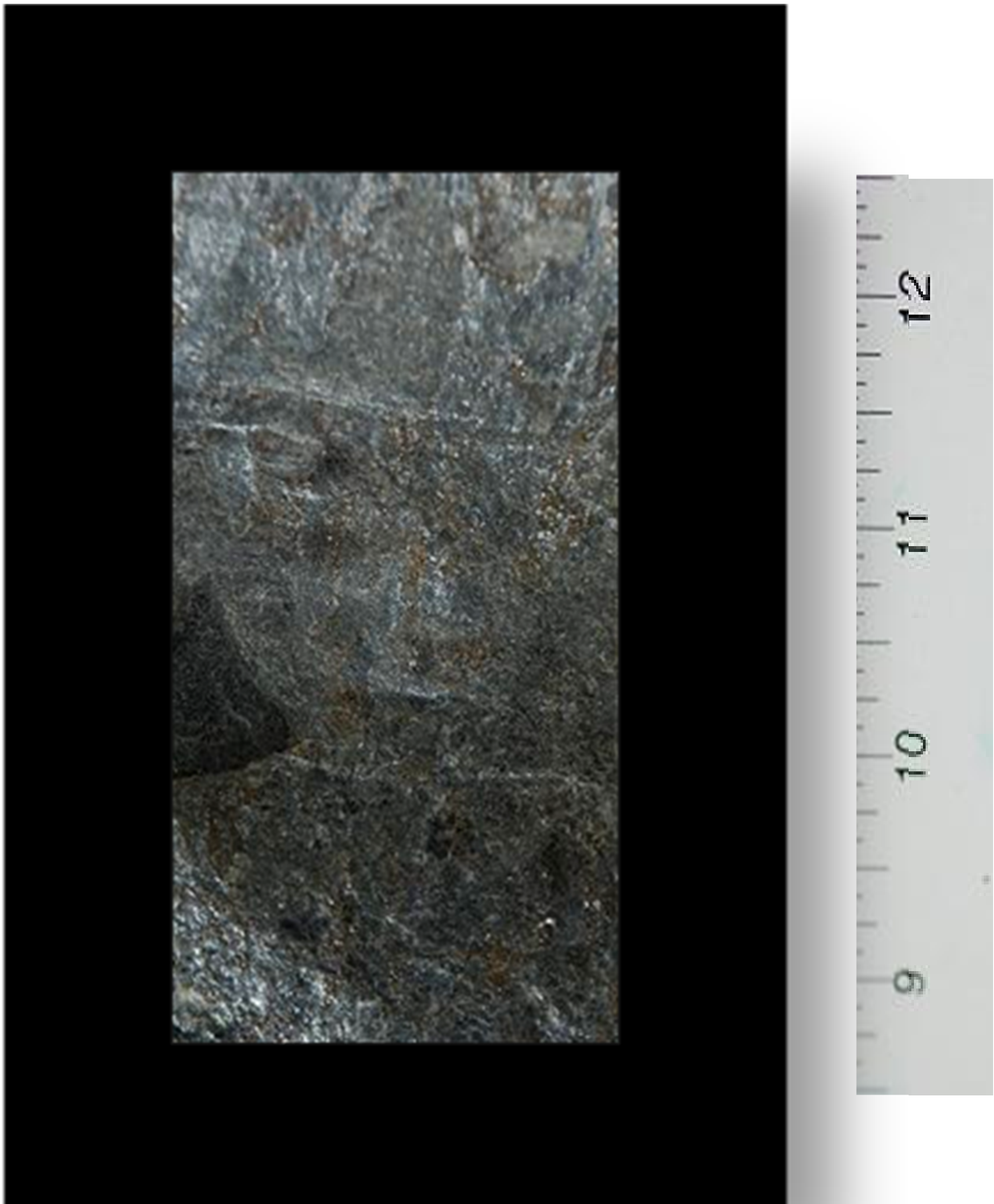
METASILTSTONE

Figure 4-3 Enlarged Photographs

METAGRAYWACKE

Figure 4-3 Enlarged Photographs

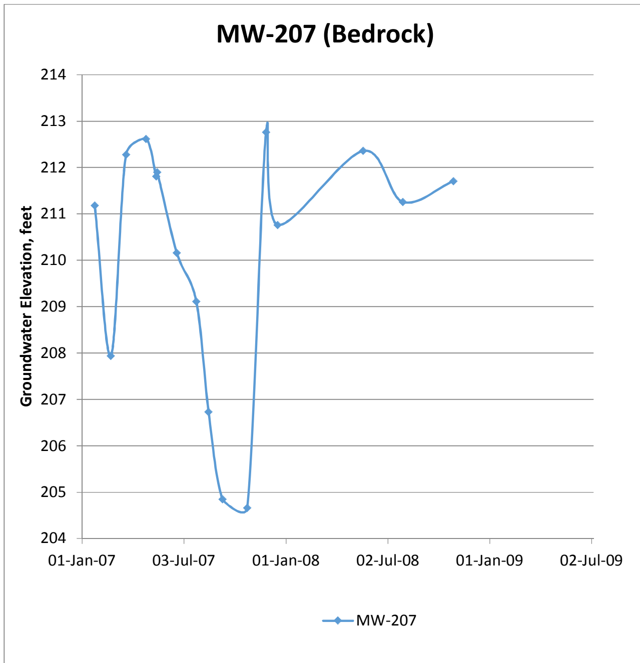
PHYLLITE

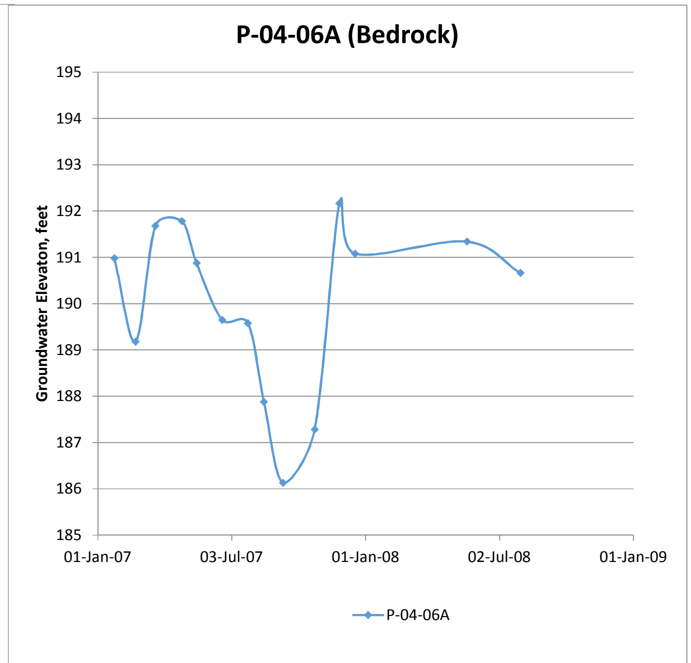


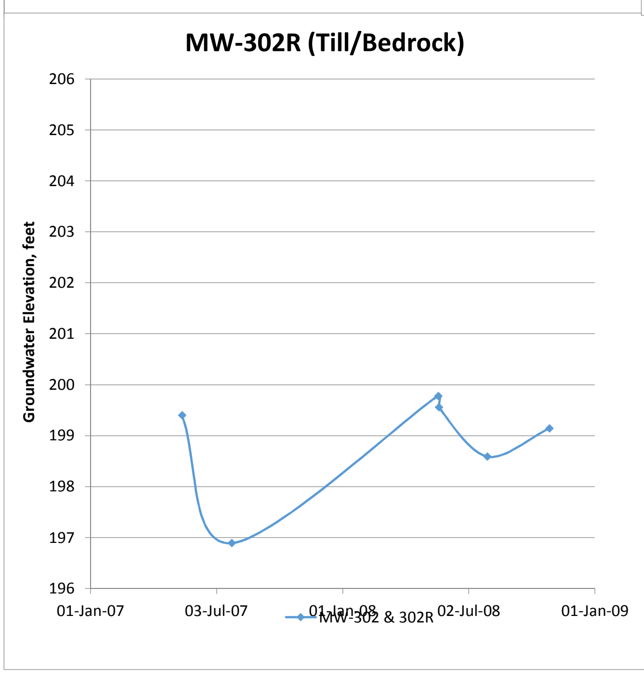
APPENDIX D

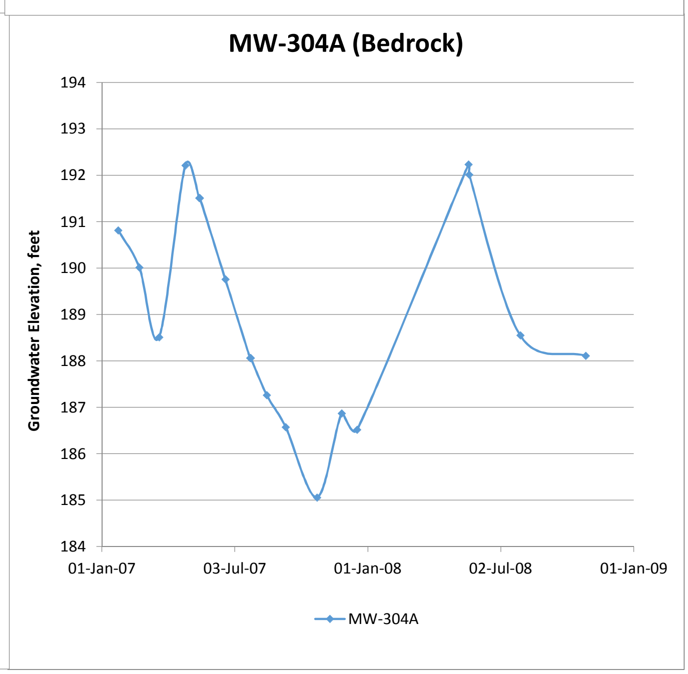
GROUNDWATER TREND PLOTS AT REPRESENTATIVE LOCATIONS

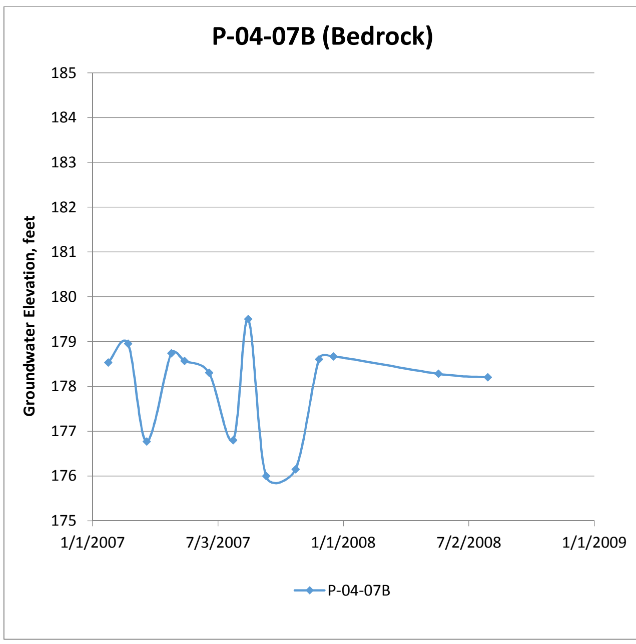
GROUNDWATER TREND PLOTS AT REPRESENTATIVE LOCATIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

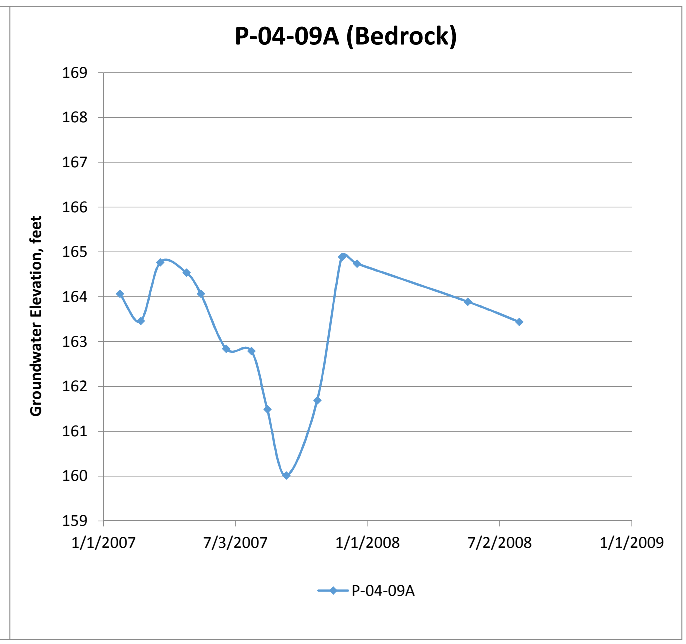




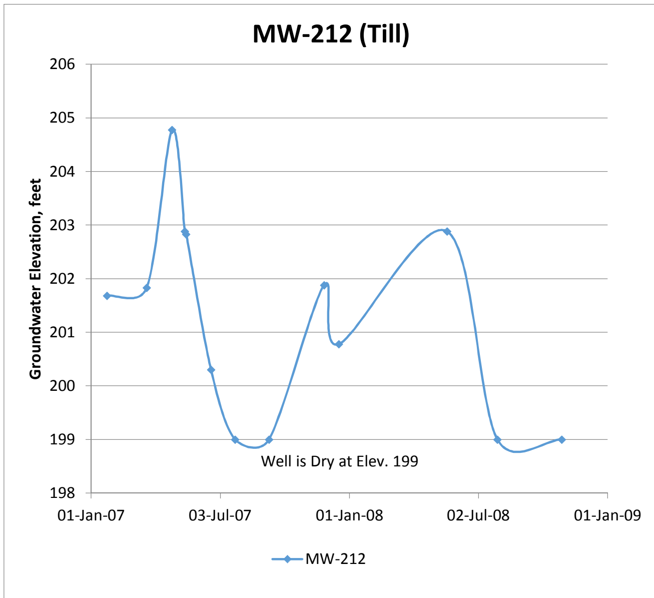


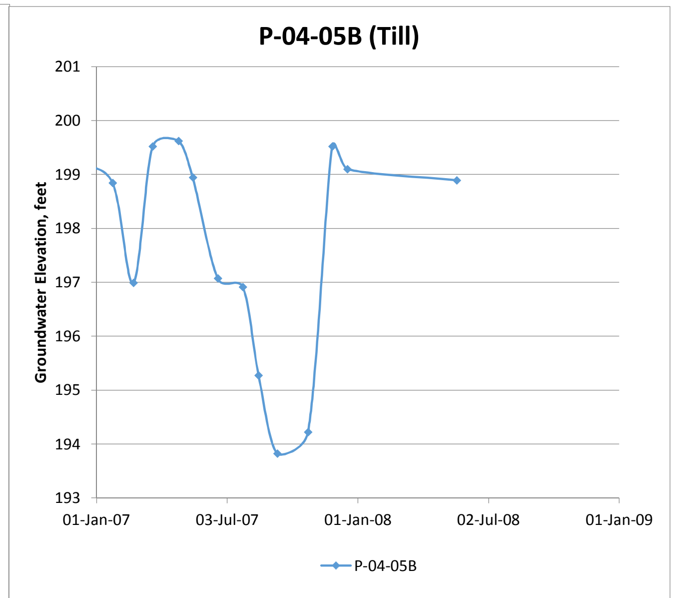


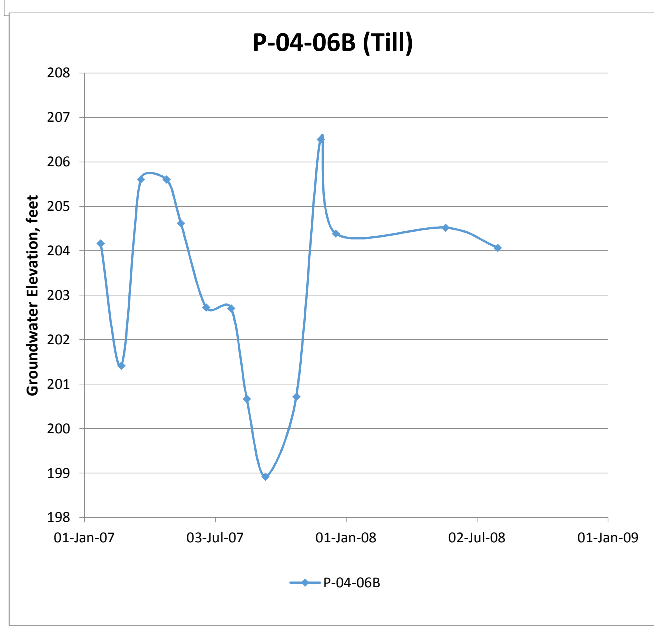

P-04-07B (Bedrock)
Groundwater Elevation, feet
185
184
183
182
181
180
179
178
177
176
175
1/1/2007
7/3/2007
1/1/2008
7/2/2008
1/1/2009
P-04-07B

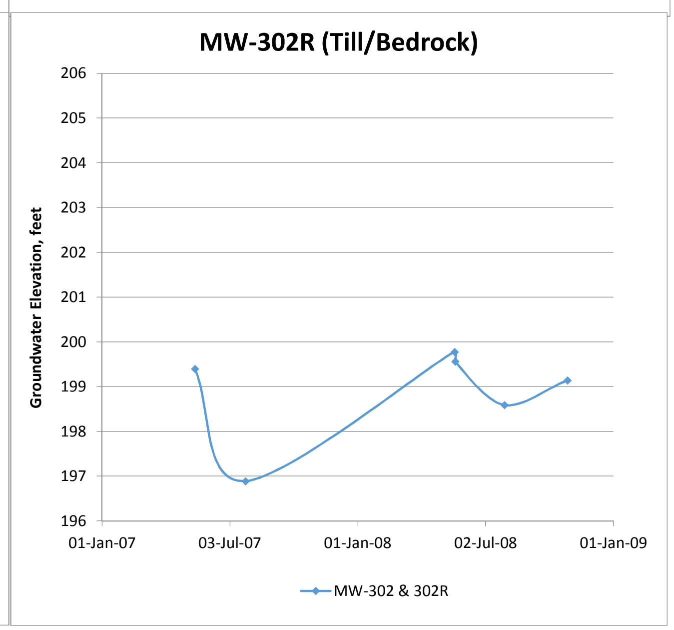

P-04-09A (Bedrock)
Groundwater Elevation, feet
169
168
167
166
165
164
163
162
161
160
159
1/1/2007
7/3/2007
1/1/2008
7/2/2008
1/1/2009
P-04-09A

GROUNDWATER TREND PLOTS AT REPRESENTATIVE LOCATIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

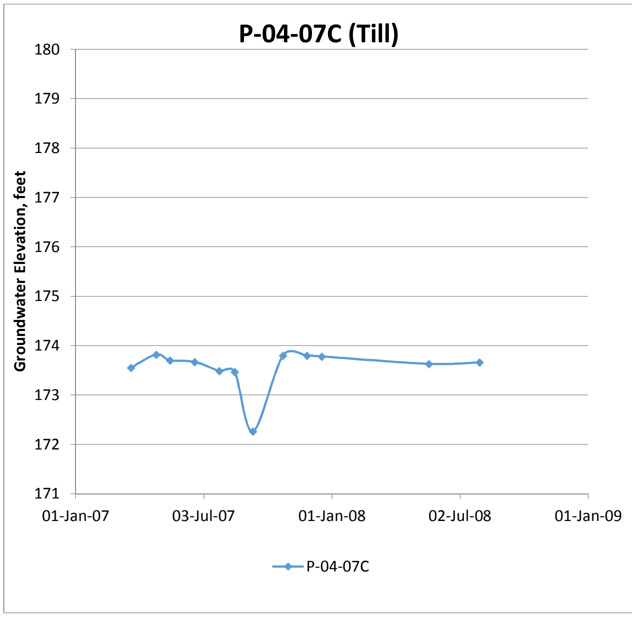


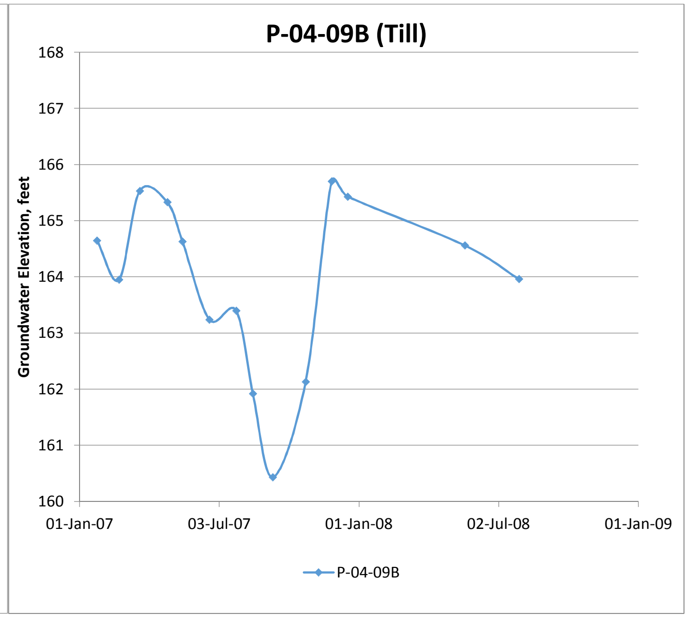


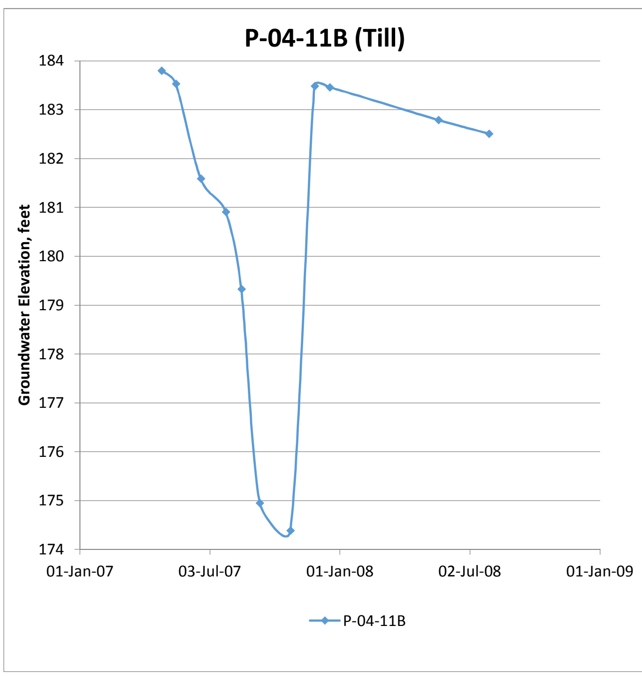


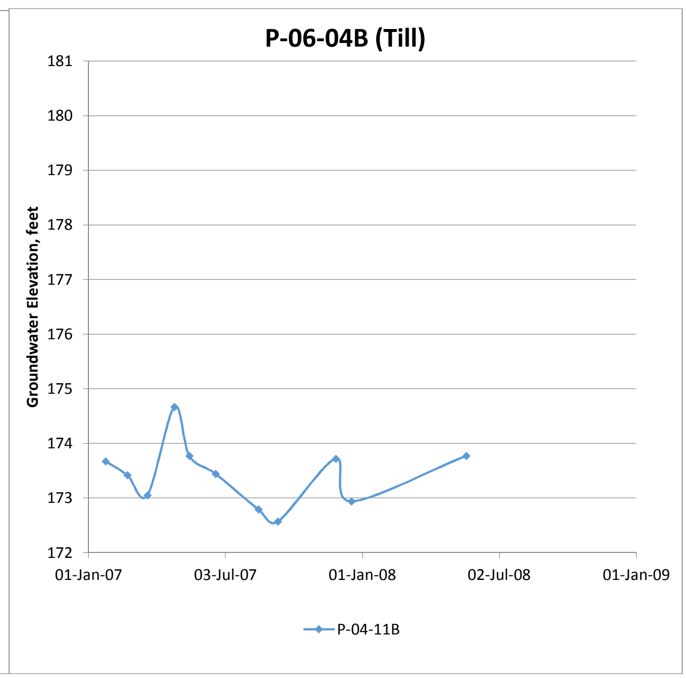


GROUNDWATER TREND PLOTS AT REPRESENTATIVE LOCATIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE



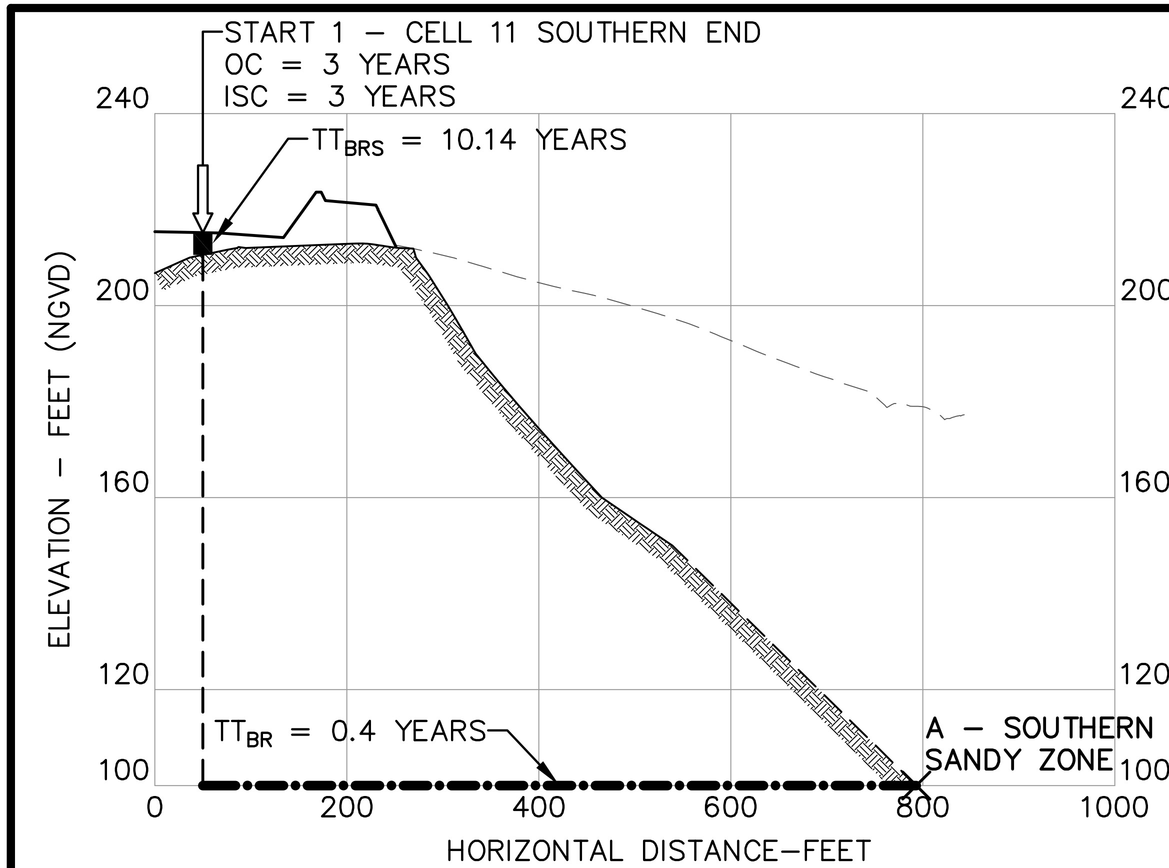




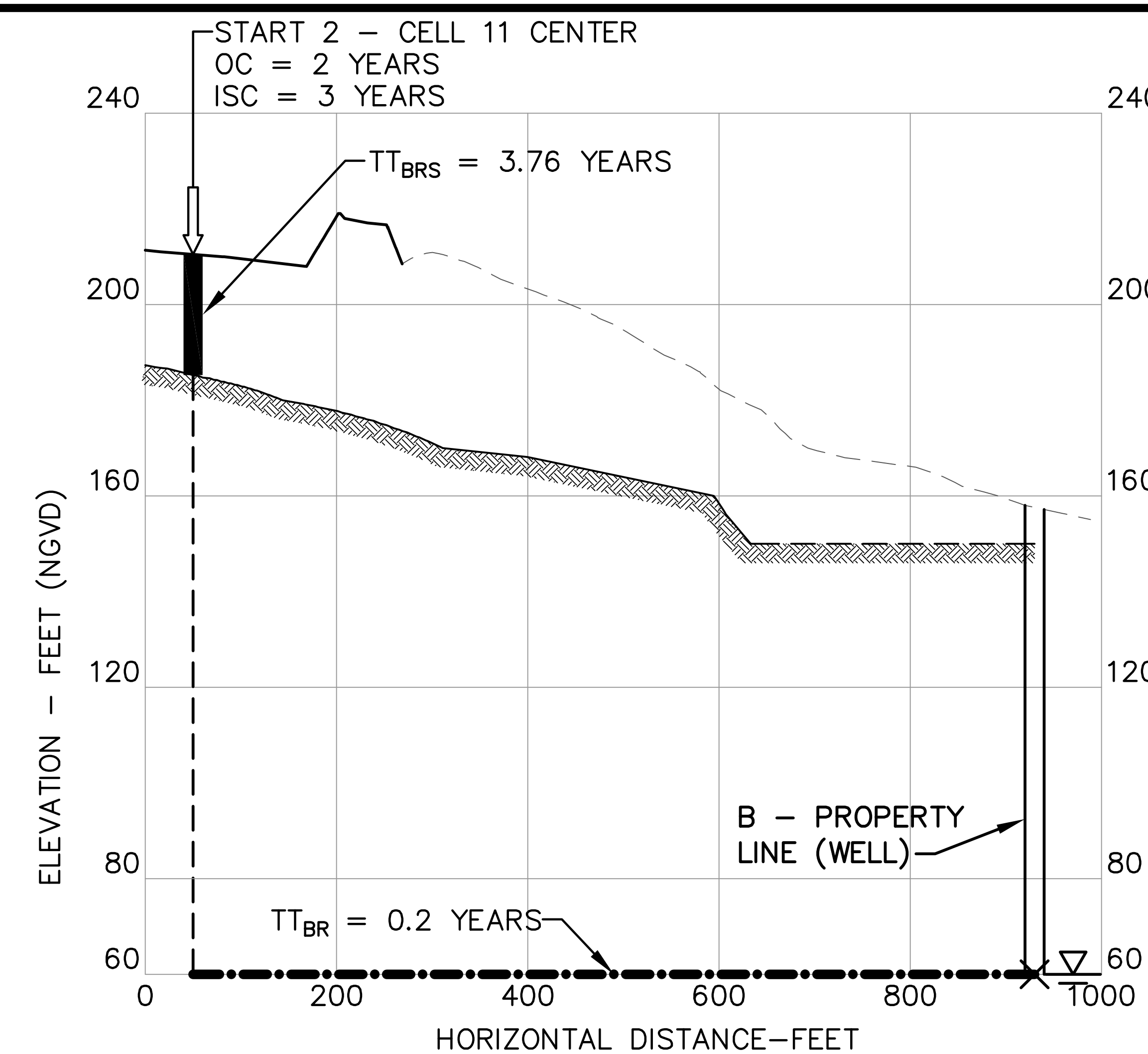


APPENDIX X

TIME OF TRAVEL SCHEMATICS



CROSS–SECTION 1–A



CROSS–SECTION 2–B

LEGEND

EXISTING GRADE

BASE GRADE

BEDROCK (DASHED IS INTERPRETED)

ASSUMED PHREATIC SURFACE IN WELL

TIME OF TRAVEL STARTS HERE AND INCLUDES:

OC — OFFSET CREDITS

ISC — IMPORTED SOIL CREDITS

TT_{BRS} — TIME OF TRAVEL TO BEDROCK SURFACE TT_{BRS}

TIME OF TRAVEL VERTICALLY IN BEDROCK (CONSERVATIVELY ASSUMED TO BE 0.0 YEARS)

TT_{BR} — TIME OF TRAVEL THROUGH BEDROCK TT_{BR} (HORIZONTALLY)

TIME OF TRAVEL ENDS AT SENSITIVE RECEPTOR LOCATION

NOTE:
1. TOTAL TRAVEL TIME INCLUDES FLOW THROUGH TWO OR THREE MATERIALS. TOTAL TRAVEL TIME IS THE SUM THROUGH ALL MATERIALS.

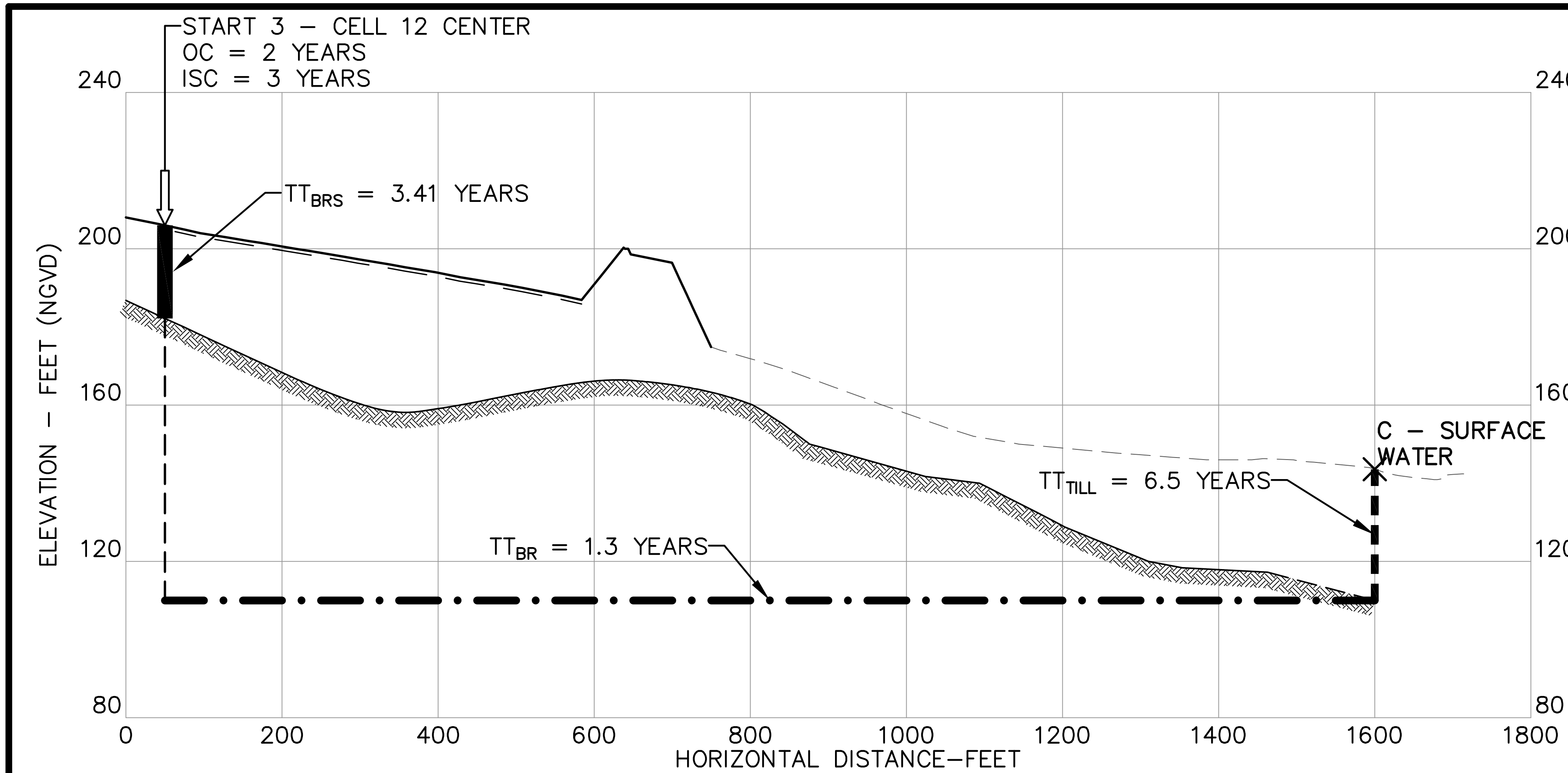
100 0 200 400 FEET (HORZ)
20 0 40 80 FEET (VERT)

FIGURE 3.4–1
SCHEMATIC TRAVEL TIME ANALYSIS CROSS–SECTIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: CROSS–SECTIONS LMN: SECTIONS CTB: SME–STD REV: 2/25/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\SECTIONS.dwg, 2/25/2016 10:00:44 AM, jrl



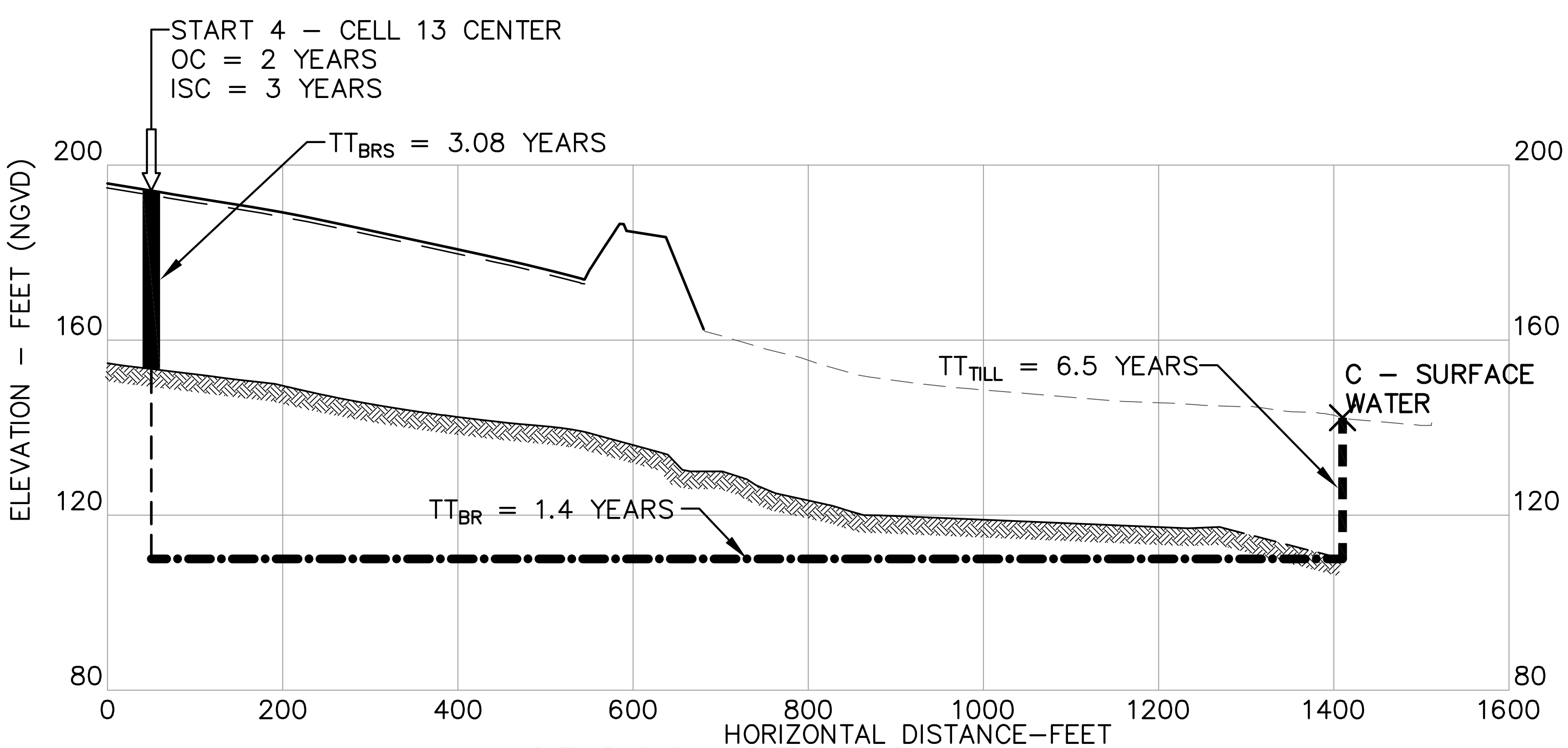
CROSS–SECTION 3–C

LEGEND

- EXISTING GRADE
- BASE GRADE
- BEDROCK (DASHED IS INTERPRETED)
- TIME OF TRAVEL STARTS HERE AND INCLUDES:
  - OC — OFFSET CREDITS
  - ISC — IMPORTED SOIL CREDITS
- TT_{BRS} — TIME OF TRAVEL TO BEDROCK SURFACE TT_{BRS}
- TIME OF TRAVEL VERTICALLY IN BEDROCK (CONSERVATIVELY ASSUMED TO BE 0.0 YEARS)
- TT_{BR} — TIME OF TRAVEL THROUGH BEDROCK TT_{BR} (HORIZONTALLY)
- TT_{TILL} — TIME OF TRAVEL THROUGH TILL TT_{TILL}
- TIME OF TRAVEL ENDS AT SENSITIVE RECEPTOR LOCATION

NOTE:

1. TOTAL TRAVEL TIME INCLUDES FLOW THROUGH TWO OR THREE MATERIALS. TOTAL TRAVEL TIME IS THE SUM THROUGH ALL MATERIALS.



CROSS–SECTION 4–C

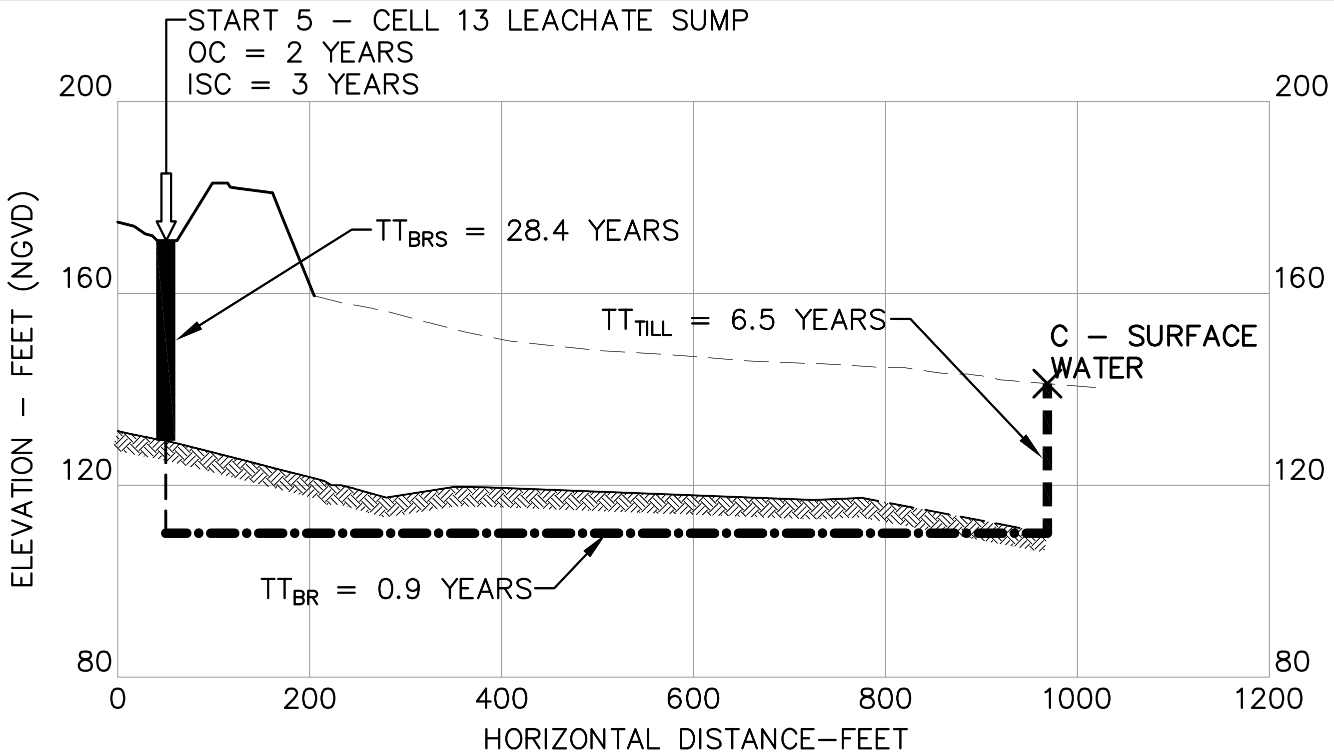
100 0 200 400 FEET (HORZ)
20 0 40 80 FEET (VERT)

FIGURE 3.4–2
SCHEMATIC TRAVEL TIME ANALYSIS CROSS–SECTIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

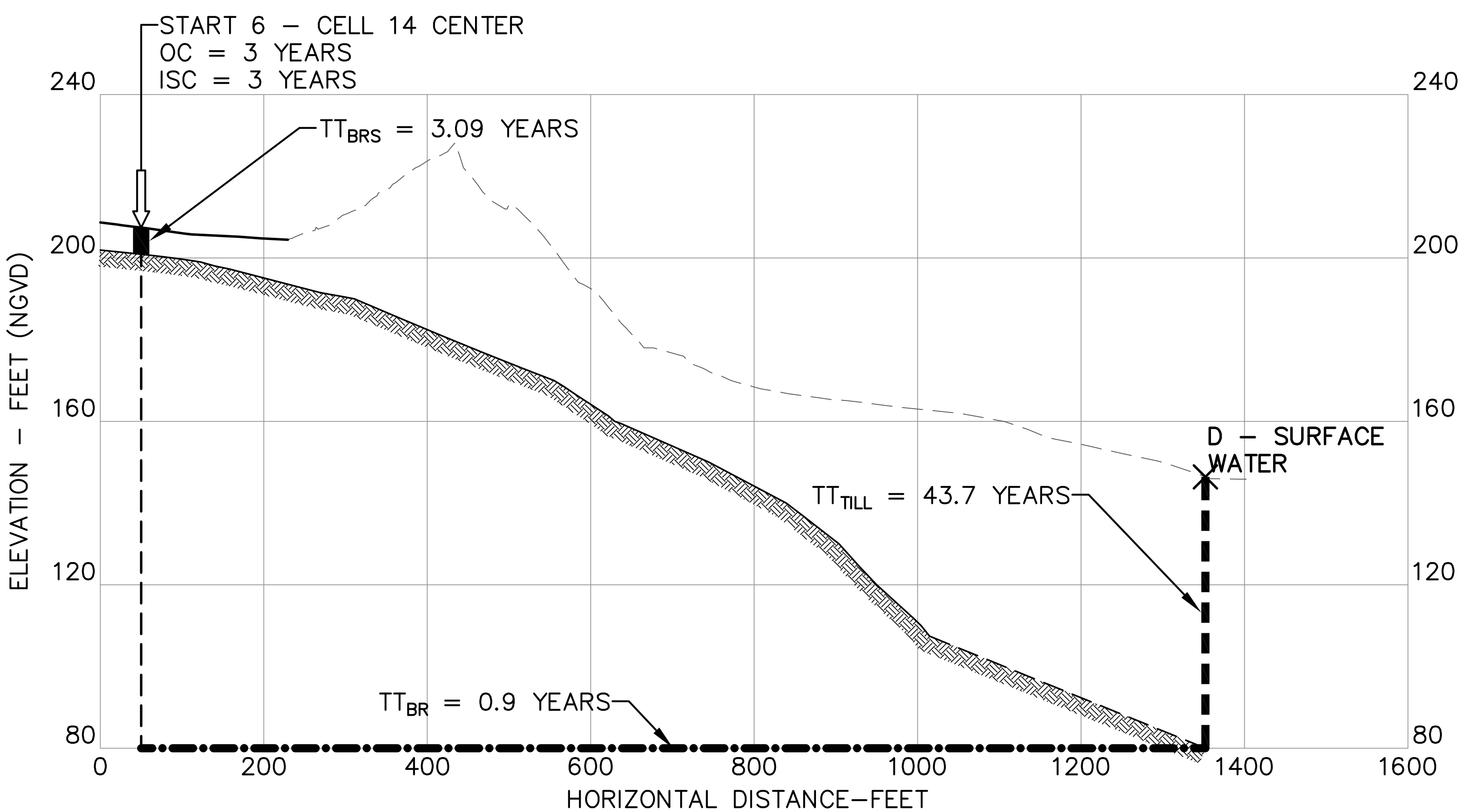
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: CROSS–SECTIONS LMN: SECTIONS CTB: SME–STD REV: 2/25/2016

\\Nserver\cls\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\SECTIONS.dwg, 2/25/2016 10:01:10 AM, jrl



CROSS–SECTION 5–C



CROSS–SECTION 6–D

LEGEND

EXISTING GRADE

BASE GRADE

BEDROCK (DASHED IS INTERPRETED)

TIME OF TRAVEL STARTS HERE AND INCLUDES:

OC — OFFSET CREDITS

ISC — IMPORTED SOIL CREDITS

TT_{BRS} — TIME OF TRAVEL TO BEDROCK SURFACE TT_{BRS}

TIME OF TRAVEL VERTICALLY IN BEDROCK (CONSERVATIVELY ASSUMED TO BE 0.0 YEARS)

TT_{BR} — TIME OF TRAVEL THROUGH BEDROCK TT_{BR} (HORIZONTALLY)

TT_{TILL} — TIME OF TRAVEL THROUGH TILL TT_{TILL}

TIME OF TRAVEL ENDS AT SENSITIVE RECEPTOR LOCATION

NOTE:
1. TOTAL TRAVEL TIME INCLUDES FLOW THROUGH TWO OR THREE MATERIALS. TOTAL TRAVEL TIME IS THE SUM THROUGH ALL MATERIALS.

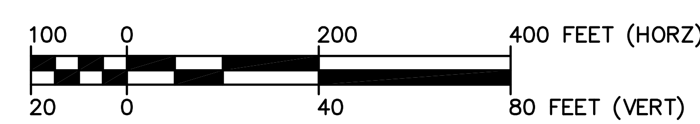
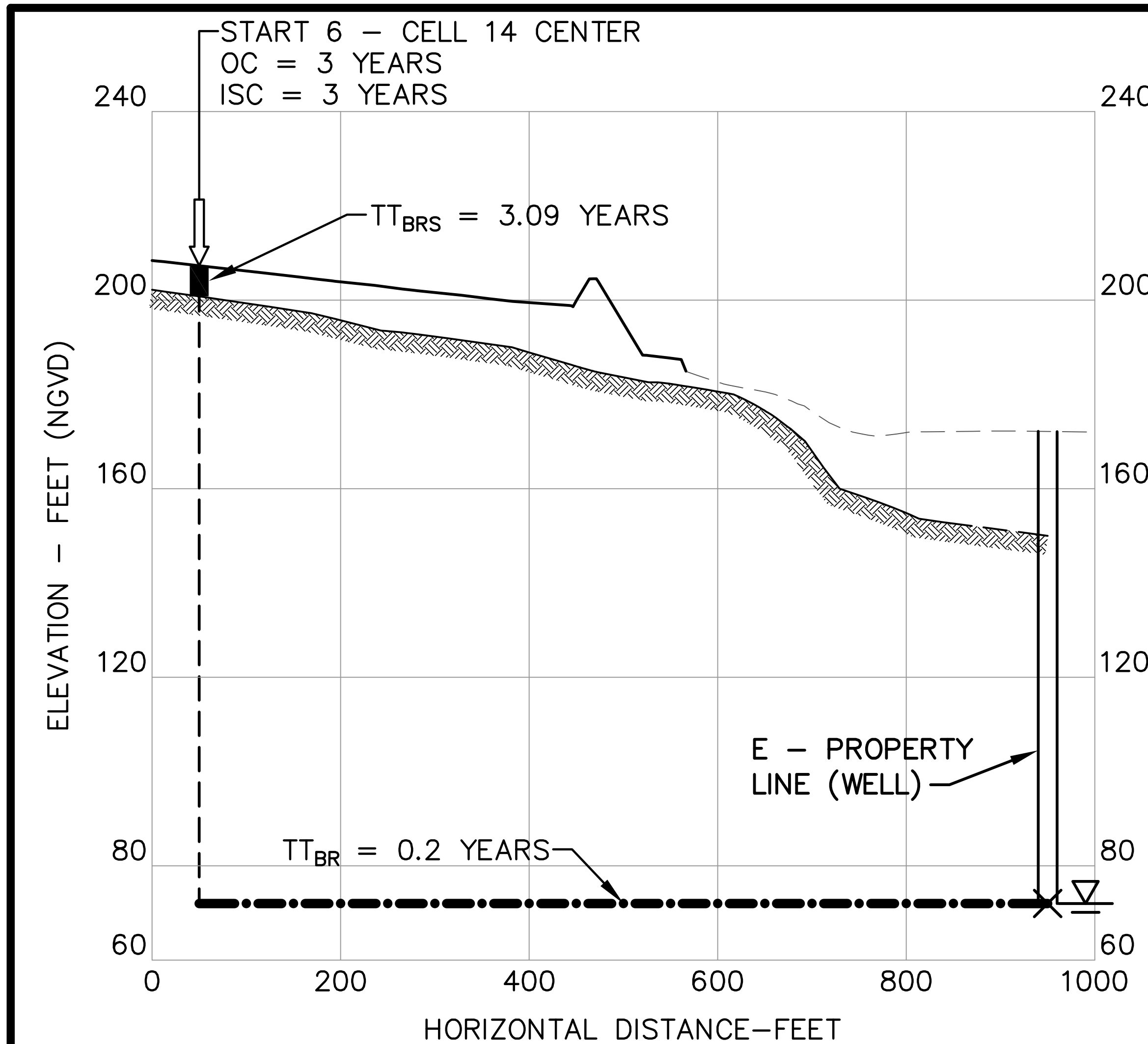


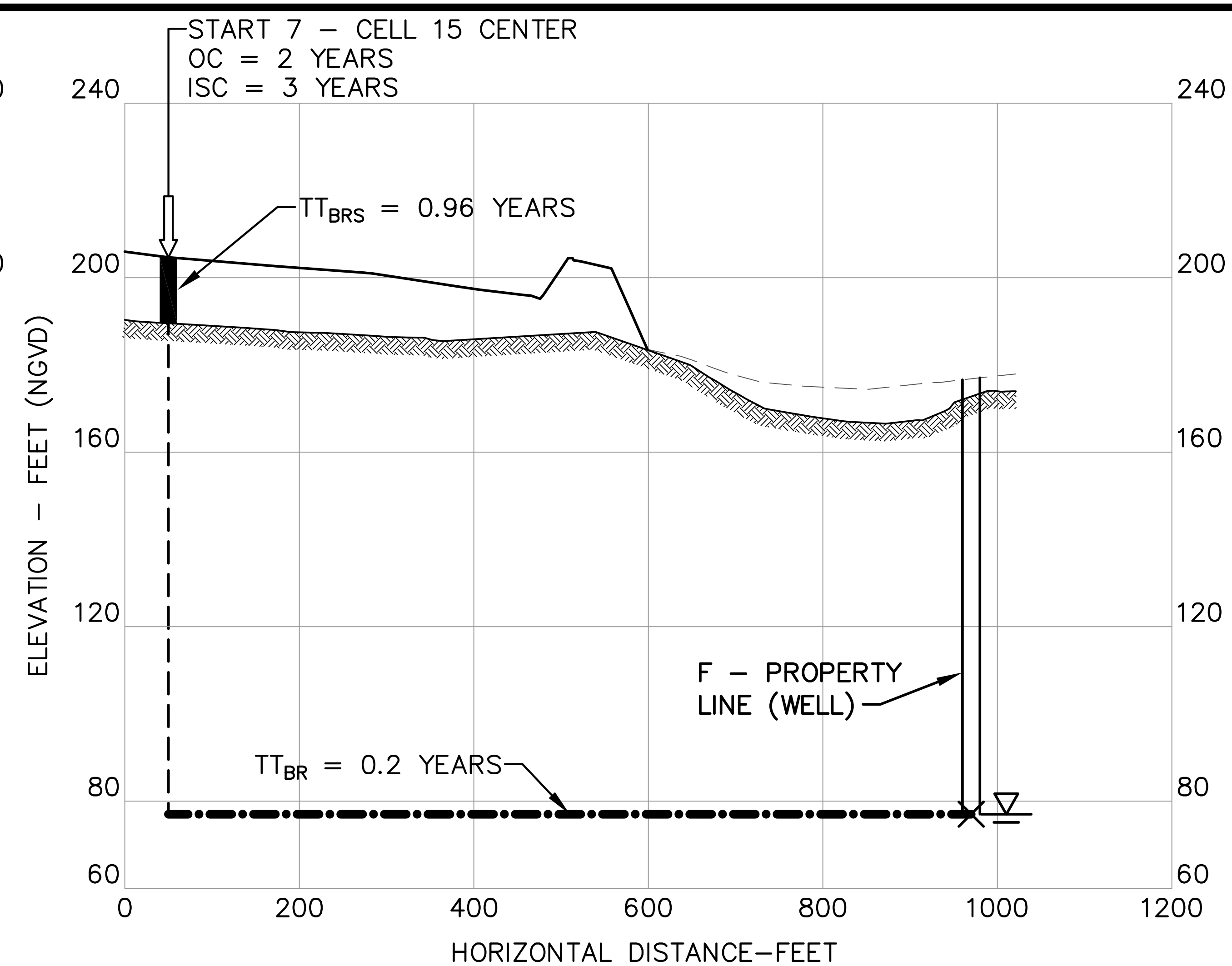
FIGURE 3.4–3
SCHEMATIC TRAVEL TIME ANALYSIS CROSS–SECTIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: CROSS–SECTIONS LMN: SECTIONS CTB: SME–STD REV: 2/25/2016



CROSS–SECTION 6–E



CROSS–SECTION 7–F

LEGEND

EXISTING GRADE

BASE GRADE

BEDROCK (DASHED IS INTERPRETED)

ASSUMED PHREATIC SURFACE IN WELL

TIME OF TRAVEL STARTS HERE AND INCLUDES:

OC — OFFSET CREDITS

ISC — IMPORTED SOIL CREDITS

TT_{BRS} — TIME OF TRAVEL TO BEDROCK SURFACE TT_{BRS}

TIME OF TRAVEL VERTICALLY IN BEDROCK (CONSERVATIVELY ASSUMED TO BE 0.0 YEARS)

TT_{BR} — TIME OF TRAVEL THROUGH BEDROCK TT_{BR} (HORIZONTALLY)

TIME OF TRAVEL ENDS AT SENSITIVE RECEPTOR LOCATION

NOTE:

1. TOTAL TRAVEL TIME INCLUDES FLOW THROUGH TWO OR THREE MATERIALS. TOTAL TRAVEL TIME IS THE SUM THROUGH ALL MATERIALS.

100 0 200 400 FEET (HORZ)

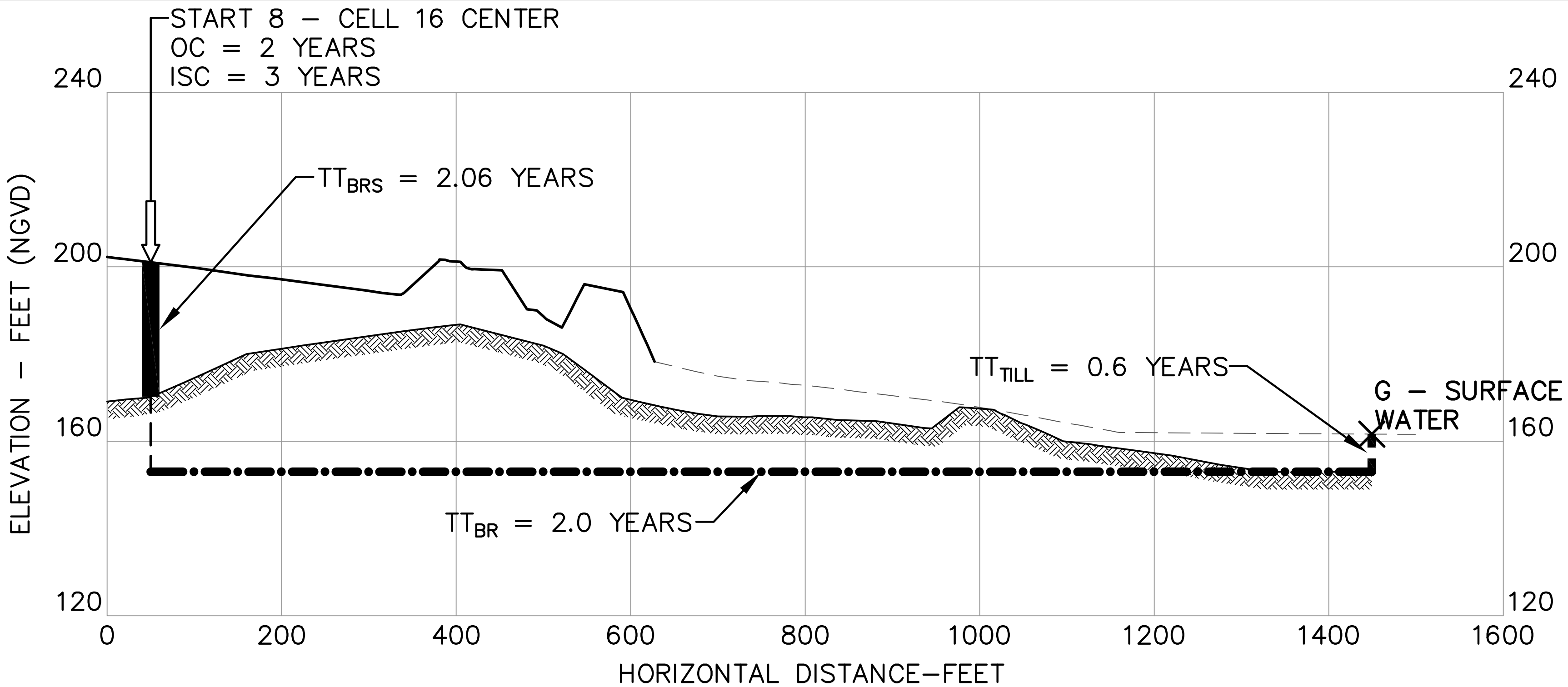
20 0 40 80 FEET (VERT)

FIGURE 3.4–4
SCHEMATIC TRAVEL TIME ANALYSIS CROSS–SECTIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

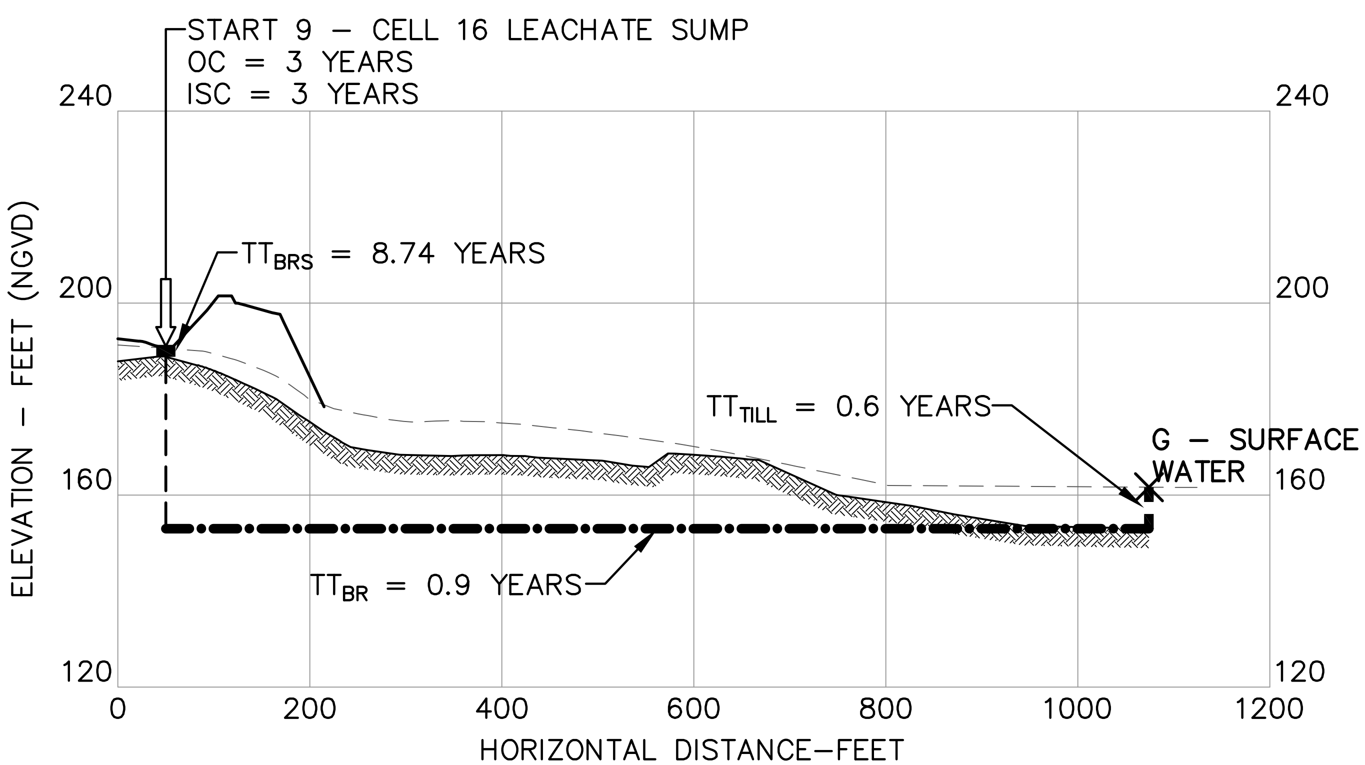
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL · CIVIL · GEOTECHNICAL · WATER · COMPLIANCE

DWG: CROSS–SECTIONS LMN: SECTIONS CTB: SME–STD REV: 2/25/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\SECTIONS.dwg, 2/25/2016 10:01:33 AM, jrl



CROSS–SECTION 8–C



CROSS–SECTION 9–G

LEGEND

EXISTING GRADE

BASE GRADE

BEDROCK (DASHED IS INTERPRETED)

TIME OF TRAVEL STARTS HERE AND INCLUDES:

OC — OFFSET CREDITS

ISC — IMPORTED SOIL CREDITS

TT_{BRS} — TIME OF TRAVEL TO BEDROCK SURFACE TT_{BRS}

TIME OF TRAVEL VERTICALLY IN BEDROCK (CONSERVATIVELY ASSUMED TO BE 0.0 YEARS)

TT_{BR} — TIME OF TRAVEL THROUGH BEDROCK TT_{BR} (HORIZONTALLY)

TT_{TILL} — TIME OF TRAVEL THROUGH TILL TT_{TILL}

TIME OF TRAVEL ENDS AT SENSITIVE RECEPTOR LOCATION

NOTE:

1. TOTAL TRAVEL TIME INCLUDES FLOW THROUGH TWO OR THREE MATERIALS. TOTAL TRAVEL TIME IS THE SUM THROUGH ALL MATERIALS.

100 0 200 400 FEET (HORZ)

20 0 40 80 FEET (VERT)

FIGURE 3.4–5
SCHEMATIC TRAVEL TIME ANALYSIS CROSS–SECTIONS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: CROSS–SECTIONS LMN: SECTIONS CTB: SME–STD REV: 2/25/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\SECTIONS.dwg, 2/25/2016 10:01:48 AM, jrl

APPENDIX X

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

JRL Expansion Application.

Values shown here include Offsets and Credits, Yielding Total Travel Time to Site Sensitive Receptors

					Node:	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
					Site Sensitive Receptor:	A	B	C	C	C	D	E	F	G	G
		TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS													
				Hydraulic Conductivity	Porosity	A	B	C	C	C	D	E	F	G	G
		Till[1]	Low Till[2]	1.7 x 10-5	0.18	10.4	6.7	10.3	10.2	19.8	25.5	7.4	5.6	8.1	10.7
Base Evaluation			GeoMean Till[3]	9.4 x 10-6	0.25	16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
			High Till[4]	5.2 x 10-6	0.3	28.4	13.3	27.9	27.2	81.7	108.5	12.9	7.3	12.8	27.2
		Bedrock	Low BR[5]	4.2 x 10-5	0.000059	16.2	8.8	15.0	14.7	40.0	52.9	9.1	6.0	7.8	15.4
Base Evaluation			GeoMean BR[6]	3.5 x 10-5	0.001	16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
			High BR[7]	2.9 x 10-5	0.016	23.6	11.8	40.8	41.1	58.0	70.2	12.4	9.9	47.1	33.5

NOTES:

1. The hydraulic conductivity values used in this analysis are horizontal hydraulic conductivity measurements. As described in Section 5.1.4 of Volume II of the Application, the average KH/KV ratio of the soils on-site was calculated to be 63, so using the horizontal hydraulic conductivity provides a conservative estimate of travel time, since the horizontal hydraulic conductivity is higher than the measured vertical hydraulic conductivities and the travel time calculations assume vertical flow through the till soils.

2. Travel time (Low Till), assumes a combination of: the Upper Confidence Limit for the mean horizontal hydraulic conductivity of the Till, as defined in Section 7.5 of Volume II of the Application; the low porosity of the Till, as defined in Section 7.5 of Volume II of the Application; and the GeoMean BR values.

3. Travel time (GeoMean Till), assumes a combination of: the Geometric Mean of Till (GeoMean Till), determined from site-specific data using the more permeable horizontal hydraulic conductivity values of the Till, as described in Section 7.4 of Volume II of the Application; the Till porosity values, as defined in Section 3.3.6 of Volume II of the Application; and the GeoMean BR values.

4. Travel time (High Till), assumes a combination of: the lower confidence limit for the mean horizontal hydraulic conductivity of the Till, as defined in Section 7.5 of Volume II of the Application; the high porosity of the Till, as defined in Section 7.5 of Volume II of the Application; and the GeoMean BR values.

5. Travel time (Low BR), assumes a combination of: the upper confidence limit for the mean horizontal hydraulic conductivity of the Bedrock, as defined in Section 7.5 of Volume II of the Application; the low porosity of the Bedrock, as defined in Section 7.5 of Volume II of the Application; and the GeoMean Till values.

6. Travel Time (GeoMean BR), assumes a combination of: the Geometric Mean of Bedrock (GeoMean BR), determined from site-specific data using the geometric mean of the horizontal hydraulic conductivity values for Bedrock as described in Section 7.4 of Volume II of the Application; the Bedrock porsity value, as defined in Section 3.3.6 of Volume II of the Application; and the GeoMean Till Values.

7. Travel time (High BR), assumes a combination of: the lower confidence limit for the mean horizontal hydraulic conductivity of the Bedrock, as defined in Section 7.5 of Volume II of the Application; and the high porosity of the Bedrock, as defined in Section 7.5 of Volume II of the Application; and the GeoMean Till values.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: Low Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)		Layer Thickness		Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.18	k_{TILL} =	1.7E-05	cm/sec	1.8E+01	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	4.4	1.7	5.3	5.2	14.8	19.5	1.4	0.6	3.1	4.7
Total Travel Time to Site Sensitive Receptor	**Years**		**10.4**	**6.7**	**10.3**	**10.2**	**19.8**	**25.5**	**7.4**	**5.6**	**8.1**	**10.7**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatric Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) — Proj #: 14101.00 — Calc by: BBJ
Client: NEWSME — Date: March 4, 2016 — VARYING TWO PARAMETERS: Low Till — Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB})	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	4.04	1.50	1.36	1.22	11.30	1.23	1.23	0.38	0.82	3.48

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 VARYING TWO PARAMETERS: Low Till Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

		Hypothetical Leak Location "Node" (See Figure 7-1)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			2.6	2.6	2.6	17.4			0.2	0.2
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**4.4**	**1.7**	**5.3**	**5.2**	**14.8**	**19.5**	**1.4**	**0.6**	**3.1**	**4.7**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 VARYING TWO PARAMETERS: GeoMean Till = GeoMean BR Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity				Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01 ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00 ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01 ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	$E_{EX\text{-}GS}$	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	$E_{DRY\text{-}PS}$	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	$E_{WET\text{-}PS}$	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY\text{-}PS\text{-}SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET\text{-}PS\text{-}SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.5	3.9	11.3	11.0	35.8	47.7	3.3	1.2	4.7	10.3
Total Travel Time to Site Sensitive Receptor	**Years**		**16.5**	**8.9**	**16.3**	**16.0**	**40.8**	**53.7**	**9.3**	**6.2**	**9.7**	**16.3**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatric Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) — Proj #: 14101.00 — Calc by: BBJ
Client: NEWSME — Date: March 4, 2016 — VARYING TWO PARAMETERS: GeoMean Till = GeoMean BR — Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB})	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: GeoMean Till = GeoMean BR | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Bedrock (horizontal lengths through which a hypothetical leak travels):												
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.5**	**3.9**	**11.3**	**11.0**	**35.8**	**47.7**	**3.3**	**1.2**	**4.7**	**10.3**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: High Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.3	k_{TILL} =	5.2E-06	cm/sec	5.4E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	22.4	8.3	22.9	22.2	76.7	102.5	6.9	2.3	7.8	21.2
Total Travel Time to Site Sensitive Receptor	**Years**		**28.4**	**13.3**	**27.9**	**27.2**	**81.7**	**108.5**	**12.9**	**7.3**	**12.8**	**27.2**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: High Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	22.00	8.17	7.40	6.67	61.60	6.69	6.69	2.08	4.46	18.96

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: High Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			14.1	14.1	14.1	94.9			1.3	1.3
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**22.4**	**8.3**	**22.9**	**22.2**	**76.7**	**102.5**	**6.9**	**2.3**	**7.8**	**21.2**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: High Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.000059	k_{BR} =	4.2E-05	cm/sec	4.3E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.2	3.8	10.0	9.7	35.0	46.9	3.1	1.0	2.8	9.4
Total Travel Time to Site Sensitive Receptor	**Years**		**16.2**	**8.8**	**15.0**	**14.7**	**40.0**	**52.9**	**9.1**	**6.0**	**7.8**	**15.4**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 VARYING TWO PARAMETERS: High Till Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 VARYING TWO PARAMETERS: High Till Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	E_{EX-GS}	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS-BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	E_{HRS-BR} (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS-MM} = E_{HRS-BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.0	0.1	0.0
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX-GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS-BR} - E_{EX-GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.2**	**3.8**	**10.0**	**9.7**	**35.0**	**46.9**	**3.1**	**1.0**	**2.8**	**9.4**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: GeoMean BR = GeoMean Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Parameter	Units	Symbol or Equation	1	2	3	4	5	6	6	7	8	9
Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.5	3.9	11.3	11.0	35.8	47.7	3.3	1.2	4.7	10.3
Total Travel Time to Site Sensitive Receptor	**Years**		**16.5**	**8.9**	**16.3**	**16.0**	**40.8**	**53.7**	**9.3**	**6.2**	**9.7**	**16.3**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatric Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: GeoMean BR = GeoMean Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB})	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L = T_{FILL} + T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H / \Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: GeoMean BR = GeoMean Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.5**	**3.9**	**11.3**	**11.0**	**35.8**	**47.7**	**3.3**	**1.2**	**4.7**	**10.3**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: Low Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)		Layer Thickness		Effective Porosity		Hydraulic Conductivity				Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.016000	k_{BR} =	2.9E-05	cm/sec	3.0E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	17.6	6.8	35.8	36.1	53.0	64.2	6.4	4.9	42.1	27.5
Total Travel Time to Site Sensitive Receptor	**Years**		**23.6**	**11.8**	**40.8**	**41.1**	**58.0**	**70.2**	**12.4**	**9.9**	**47.1**	**33.5**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 VARYING TWO PARAMETERS: Low Till Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB})	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L = T_{FILL} + T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

TIME OF TRAVEL EVALUATION OF VARYING TWO PARAMETERS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | VARYING TWO PARAMETERS: Low Till | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Bedrock (horizontal lengths through which a hypothetical leak travels):		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	E_{EX-GS}	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS-BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	E_{HRS-BR} (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS-MM} = E_{HRS-BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	7.5	3.0	25.9	26.5	18.0	17.4	3.3	3.9	39.4	18.2
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX-GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS-BR} - E_{EX-GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**17.6**	**6.8**	**35.8**	**36.1**	**53.0**	**64.2**	**6.4**	**4.9**	**42.1**	**27.5**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

APPENDIX I

FIELD DATA FORMS

MONITORING WELL SAMPLE PURGING FORM

SITE: Loselle W Old Town PROJECT NO: 03076.04 DATE: 1.26.05

SAMPLE LOCATION: P-0406A WEATHER: 5° Snow

SAMPLE ID: GW X06 A 127 START TIME: 0830 END: 1040

(DUPS) — TRIP BLANK ID: NA

WELL DEPTH: 46.70 FT
(x) TOP OF WELL ( ) TOP OF CASING
( ) MEASURED (x) HISTORICAL

WATER DEPTH: 20.43 FT
(x) TOP OF WELL ( ) TOP OF CASING
(x) MEASURED ( ) HISTORICAL

CONDITION OF WELL:
SURFACE SEAL: ( )GOOD ( )CRACKED
( )OTHER:
PROTECTIVE CASING: ( )LOCKED
( )NO LOCK
( )SECURE
( )NEEDS REPAIR(ABLE TO MOVE)

TUBING INLET (TPVC) 46.0
TUBING DIAMETER .17 (ID)
SCREENED INTERVAL (TPVC) 45.70 TO 46.70

WELL: ( )CAP ( )NO CAP
WELL MATL: ( )PVC ( )SS ( )OTHER:

PUMPING START TIME: 1010 PUMPING END TIME: 1038

EQUIPMENT DECONTAMINATION

PURGING	SAMPLING	
( )	( )	PERISTALTIC PUMP ISCO
( )	( )	PERISTALTIC PUMP GEOTECH
( )	( )	SUBMERSIBLE PUMP
(x)	(x)	BLADDER PUMP
( )	( )	AIR LIFT PUMP
( )	( )	BAILER I.D.
( )	( )	LDPE/SILICON TUBING
( )	( )	TEFLON/SILICON TUBING
( )	( )	IN-LINE FILTER
(x)	(x)	DEDICATED SIL. TUBING
(x)	(x)	DEDICATED POLY. TUBING

DECONTAMINATION FLUIDS USED

(x) DISTILLED/DEIONIZED WATER
( ) TAP WATER
( ) NON-PHOSPHATE DETERGENT
( ) 10% NITRIC ACID
( ) HIGH-PRESSURE STEAM CLEAN
( )

AMOUNT OF WATER CONTAINED IN DEDICATED SYSTEM: 137 mls
AMOUNT OF WATER PURGED PRIOR TO GRAB SAMPLE COLLECTION: 200 mls

NOTES:

SAMPLED BY: T. Cahoon

MONITORING WELL SAMPLE PURGING FORM - PART II

SITE: Cesello W. Old Town

DATE: 1.26.05

SAMPLE LOCATION: P.6406A

ORP OFFSET: Sub1 mV

Elapsed Time (min)	Liters Pumped	Flow Rate (ml/min)	WL TPVC (ft)	WL Top of Casing (ft)	Turb (1)	pH (2)	Spec Cond (3)	Temp °C (4)	DO (5)	ORP (6)	Comments
Unit ID Number:			300'		C	IV	IV	"	Clem	V	
Model ID :			51		Refer to Col log						
0			20.43								
2		100	21.61		GRAB SAMPLE						
					stop pump for cooper tubes, well recharges						
10		100	20.55		begin 1 liter						
12			21.57								
14			21.80								
16		100	21.90								
18		80	21.75								
21		80	21.38								
27			21.38		5.2	6.8	266	5.6	0.6	-9	

Alkalinity(CaCO3) Cole Palmer 05542-01 Field Test

"P" Alkalinity liquid level reading________*______= NA mg/L CaCO3

"T" Alkalinity liquid level reading________*______= NA mg/L CaCO3

NOTES:

(1) TURBIDITY (NTU)
(2) pH (STD UNITS)
(3) SPECIFIC CONDUCTANCE (umhos/cm @25C)
(4) TEMPERATURE (C)
(5) DISSOLVED OXYGEN (ppm)
(6) UNADJUSTED OXIDATION REDUCTION POTENTIAL (+- mV)

MONITORING WELL SAMPLE PURGING FORM

SITE: Cecile Old Town   PROJECT NO: 03076.04   DATE: 1-25-05

SAMPLE LOCATION: PO407B   WEATHER: Clear 18°F

SAMPLE ID: GW X07 B 124   START TIME: 1400   END: 1600

(DUPS) ___ ___ ___ ___   TRIP BLANK ID: NA

WELL DEPTH: 28.53 FT
(✓) TOP OF WELL   ( ) TOP OF CASING
(-) MEASURED   (x) HISTORICAL

CONDITION OF WELL:
SURFACE SEAL: (x) GOOD ( ) CRACKED
( ) OTHER: ________
PROTECTIVE CASING: (x) LOCKED
( ) NO LOCK
( ) SECURE
( ) NEEDS REPAIR (ABLE TO MOVE)

WATER DEPTH: 0.00 Flowing FT
(✓) TOP OF WELL   ( ) TOP OF CASING
(✓) MEASURED   ( ) HISTORICAL

TUBING INLET (TPVC) 27.83
TUBING DIAMETER .17 (ID)
SCREENED INTERVAL (TPVC) 27.53 TO 28.53

WELL: (x) CAP ( ) NO CAP
WELL MATL: (x) PVC ( ) SS ( ) OTHER: ________

PUMPING START TIME: 1450   PUMPING END TIME: 1510

EQUIPMENT DECONTAMINATION

PURGING	SAMPLING	
( )	( )	PERISTALTIC PUMP ISCO
( )	( )	PERISTALTIC PUMP GEOTECH
( )	( )	SUBMERSIBLE PUMP
( )	( )	BLADDER PUMP
( )	( )	AIR LIFT PUMP
( )	( )	BAILER I.D. ________
( )	( )	LDPE/SILICON TUBING
( )	( )	TEFLON/SILICON TUBING
( )	( )	IN-LINE FILTER
( )	( )	DEDICATED SIL. TUBING
( )	( )	DEDICATED POLY. TUBING

DECONTAMINATION FLUIDS USED

( ) DISTILLED/DEIONIZED WATER
( ) TAP WATER
( ) NON-PHOSPHATE DETERGENT
( ) 10% NITRIC ACID
( ) HIGH-PRESSURE STEAM CLEAN
( ) ________

AMOUNT OF WATER CONTAINED IN DEDICATED SYSTEM: ________
AMOUNT OF WATER PURGED PRIOR TO GRAB SAMPLE COLLECTION: ________

NOTES: ________

SAMPLED BY: T. Wilcoxson

MONITORING WELL SAMPLE PURGING FORM

SITE: Cessla Old Town PROJECT NO: 03076.04 DATE: 1-25-05

SAMPLE LOCATION: P0407B WEATHER: Clear 18°F

SAMPLE ID: GW X07 B 124 START TIME: 1400 END: 1600

(DUPS) ___ ___ ___ ___ TRIP BLANK ID: ___ NA ___

___ ___ ___ ___

WELL DEPTH: 28.53 FT
(✓) TOP OF WELL ( ) TOP OF CASING
( ) MEASURED (x) HISTORICAL

WATER DEPTH: 0.00 flowing FT
(✓) TOP OF WELL ( ) TOP OF CASING
(✓) MEASURED ( ) HISTORICAL

CONDITION OF WELL:
SURFACE SEAL: (x) GOOD ( ) CRACKED
( ) OTHER: ______
PROTECTIVE CASING: (x) LOCKED
( ) NO LOCK
( ) SECURE
( ) NEEDS REPAIR(ABLE TO MOVE)

TUBING INLET (TPVC) 27.83
TUBING DIAMETER .17 (ID)
SCREENED INTERVAL (TPVC) 27.53 TO 28.53

WELL: (x) CAP ( ) NO CAP
WELL MATL: (x) PVC ( ) SS ( ) OTHER: ______

PUMPING START TIME: 1450 PUMPING END TIME: 1510

EQUIPMENT DECONTAMINATION

PURGING	SAMPLING	
( )	( )	PERISTALTIC PUMP ISCO
( )	( )	PERISTALTIC PUMP GEOTECH
( )	( )	SUBMERSIBLE PUMP
( )	( )	BLADDER PUMP
( )	( )	AIR LIFT PUMP
( )	( )	BAILER I.D. ______
( )	( )	LDPE/SILICON TUBING
( )	( )	TEFLON/SILICON TUBING
( )	( )	IN-LINE FILTER
( )	( )	DEDICATED SIL. TUBING
( )	( )	DEDICATED POLY. TUBING

DECONTAMINATION FLUIDS USED

( ) DISTILLED/DEIONIZED WATER
( ) TAP WATER
( ) NON-PHOSPHATE DETERGENT
( ) 10% NITRIC ACID
( ) HIGH-PRESSURE STEAM CLEAN
( ) ______

AMOUNT OF WATER CONTAINED IN DEDICATED SYSTEM: ______
AMOUNT OF WATER PURGED PRIOR TO GRAB SAMPLE COLLECTION: ______

NOTES: ______

SAMPLED BY: T.Wilcoxson

MONITORING WELL SAMPLE PURGING FORM - PART II

SITE: Lsodla

DATE: 1·25·05

SAMPLE LOCATION: P0407B

ORP OFFSET: 5431 mV

Elapsed Time (min)	Liters Pumped	Flow Rate (ml/min)	WL TPVC (ft)	WL Top of Casing (ft)	Turb (1)	pH (2)	Spec Cond (3)	Temp °C (4)	DO (5)	ORP (6)	Comments
Unit ID Number:					C	IV	IV	"	Clam	V	
Model ID :			visual		Refer to Log						
0		250	0.00		1.7	7.6	156	6.0	4.0	+160	well flowing collect from
					GRAB SAMPLE						flowing tubing
15		250	0.00		0.5	7.2	177	6.9	4.0	+186	prior to sampling

Alkalinity(CaCO3) Cole Palmer 05542-01 Field Test

"P" Alkalinity liquid level reading_____*_____= NA mg/L CaCO3

"T" Alkalinity liquid level reading_____*_____= NA mg/L CaCO3

NOTES:

(1) TURBIDITY (NTU)
(2) pH (STD UNITS)
(3) SPECIFIC CONDUCTANCE (umhos/cm @25C)
(4) TEMPERATURE (C)
(5) DISSOLVED OXYGEN (ppm)
(6) UNADJUSTED OXIDATION REDUCTION POTENTIAL (+- mV)

APPENDIX U

TABULATED DATA USED TO PRODUCE FIGURE U-16

Tabulated Data Used to Produce Figure U-16
(Part 1 of 5)

PW-08-01 7-Day Pump Test		
Well ID	Azimuth from Pumping Well (degrees)	Log of Normalized Drawdown (log-feet)
MW-05-04	274.7	0.0000
MW-06-01	134.3	0.0591
MW-06-02	285.4	0.0014
MW-207	173.6	0.0000
MW-223A	220.2	0.0248
MW-223B	219.8	0.0162
MW-227	213.9	0.0000
MW-302R	217.6	0.0000
MW-304A	161.1	0.0000
OSW	199.3	0.0819
OW-06-05	286.9	0.0020
OW-06-06	286.6	0.0032
OW-06-07	286.3	0.0190
OW-06-08	285.2	0.0000
OW-06-09	284.7	0.0000
OW-06-10	283.6	0.0000
P-04-05A	182.7	0.0822
P-04-05B	182.7	0.0000
P-04-06A	228.9	0.0619
P-04-06B	228.9	0.0000
P-04-07A	234.7	0.0204
P-04-07B	234.7	0.0602
P-04-07C	234.9	0.0000
P-04-08A	289.8	0.0161
P-04-08B	289.8	0.0000
P-04-09A	317.1	0.0000
P-04-10A	328.8	0.0618
P-04-11A	293.8	0.0290
P-04-12A	289.3	0.0020
P-04-12B	289.3	0.0000
P-04-12C	289.4	0.0000
P-04-13A	79.7	0.0019
P-04-13B	79.7	0.0009
P-04-13C	80.0	0.0000
P-04-14A	359.0	0.0000
P-04-14B	359.0	0.0000
P-06-04B	293.8	0.0000
P-08-03A	330.0	0.0000
P-08-03B	330.0	0.0000
P-08-04	258.7	0.0000
P-08-06	184.3	0.0143
P-08-07	199.9	0.0000
P-08-09a	151.2	0.0000
P-08-09b	151.2	0.0000
P-08-10a	153.6	0.0000
P-08-10b	153.6	0.0000
P-213A	145.7	0.0000
P-213B	145.7	0.0000
PW-08-02	240.5	0.0960
PW-08-03	312.3	0.0346
PW-08-04	302.7	0.0155
SHSW	228.1	0.0945

Tabulated Data Used to Produce Figure U-16
(Part 2 of 5)

PW-08-01 24-Hour Pump Test		
Well ID	Azimuth from Pumping Well (degrees)	Log of Normalized Drawdown (log-feet)
MW-04-111	274.2	0.0000
MW-05-01	274.2	0.0000
MW-05-02	274.5	0.0000
MW-05-03	274.6	0.0000
MW-05-04	274.7	0.0000
MW-05-05	274.0	0.0000
MW-06-01	134.3	0.0727
MW-06-02	285.4	0.0063
MW-207	173.6	0.0017
MW-223A	220.2	0.0167
MW-223B	219.8	0.0098
MW-227	213.9	0.0000
MW-302R	217.6	0.0014
MW-304A	161.1	0.0000
OW-06-05	286.9	0.0074
OW-06-06	286.6	0.0031
OW-06-07	286.3	0.0148
OW-06-08	285.2	0.0000
OW-06-09	284.7	0.0037
OW-06-10	283.6	0.0044
P-04-05A	182.7	0.0696
P-04-05B	182.7	0.0000
P-04-06A	228.9	0.0539
P-04-06B	228.9	0.0000
P-04-07A	234.7	0.0259
P-04-07B	234.7	0.0808
P-04-08A	289.8	0.0104
P-04-08B	289.8	0.0000
P-04-09A	317.1	0.0027
P-04-10A	328.8	0.0783
P-04-10B	328.8	0.0020
P-04-11A	293.8	0.0319
P-04-11B	293.8	0.0000
P-04-12A	289.3	0.0067
P-04-13B	79.7	0.0041
P-04-13C	80.0	0.0000
P-04-14A	359.0	0.0023
P-04-14B	359.0	0.0000
P-06-04A	293.8	0.0000
P-06-04B	293.8	0.0000
P-08-03A	330.0	0.0000
P-08-03B	330.0	0.0000
P-08-04	258.7	0.0000
P-08-06	184.3	0.0102
P-08-07	199.9	0.0021
PW-08-02	240.5	0.1052
PW-08-03	312.3	0.0290
PW-08-04	302.7	0.0107

Tabulated Data Used to Produce Figure U-16
(Part 3 of 5)

PW-08-02 50-Hour Pump Test		
Well ID	Azimuth from Pumping Well (degrees)	Log of Normalized Drawdown (log-feet)
MW-06-01	101.6	0.0135
MW-06-02	343.7	0.0135
MW-207	123.5	0.0000
MW-223A	198.4	0.1248
MW-223B	197.8	0.0766
MW-227	193.1	0.0000
MW-302R	176.0	0.0000
OSW	126.9	0.2745
OW-06-05	343.1	0.0123
OW-06-06	342.6	0.0044
OW-06-07	342.2	0.0258
OW-06-08	341.1	0.0000
OW-06-09	340.7	0.0069
OW-06-10	341.0	0.0086
P-04-05A	91.1	0.1364
P-04-05B	91.1	0.0000
P-04-06A	111.7	0.2814
P-04-06B	111.7	0.0000
P-04-07A	225.3	0.1096
P-04-07B	225.3	0.3311
P-04-08A	5.2	0.0370
P-04-08B	5.2	0.0226
P-04-09A	356.3	0.0000
P-04-09B	356.3	0.0000
P-04-10A	17.2	0.0246
P-04-11A	39.8	0.0502
P-04-12A	324.7	0.0000
P-04-12B	324.7	0.0000
P-04-12C	324.8	0.0000
P-04-13A	68.6	0.0000
P-04-14A	21.7	0.0000
P-04-14B	21.7	0.0000
P-06-04B	39.8	0.0057
P-08-06	66.4	0.0024
P-08-07	116.2	0.0000
PW-08-01	60.5	0.1128
PW-08-03	349.8	0.0128
PW-08-04	334.5	0.0276
SHSW	106.6	0.4444

Tabulated Data Used to Produce Figure U-16
(Part 4 of 5)

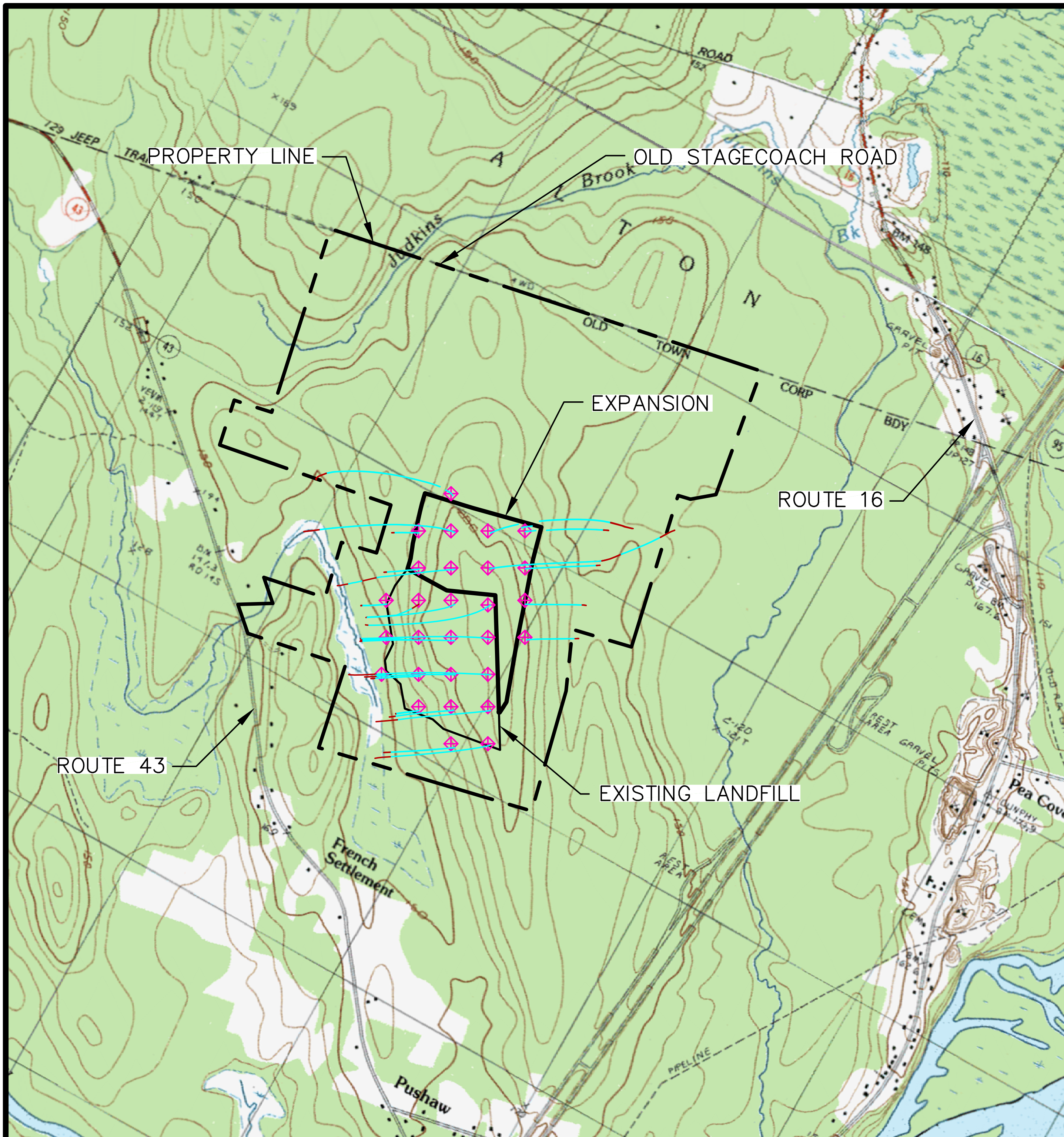
PW-08-03 26.5-Hour Pump Test		
Well ID	Azimuth from Pumping Well (degrees)	Log of Normalized Drawdown (log-feet)
MW-04-111	177.6	0.0000
MW-05-01	178.1	0.0000
MW-05-02	178.0	0.0000
MW-05-03	177.8	0.0000
MW-05-04	177.5	0.0000
MW-05-05	178.0	0.0000
MW-06-01	133.2	0.0000
MW-06-02	176.8	0.0000
MW-207	149.8	0.0007
MW-223A	180.4	0.0000
MW-223B	180.3	0.0000
MW-227	180.2	0.0000
MW-302R	171.5	0.0006
MW-304A	146.4	0.0000
OW-06-05	178.3	0.0000
OW-06-06	178.8	0.0000
OW-06-07	179.2	0.0000
OW-06-08	180.0	0.0000
OW-06-09	180.2	0.0000
OW-06-10	179.2	0.0000
P-04-05A	144.1	0.0006
P-04-05B	144.1	0.0000
P-04-06A	163.3	0.0000
P-04-06B	163.3	0.0000
P-04-07A	184.8	0.0000
P-04-07B	184.8	0.0000
P-04-07C	184.9	0.0000
P-04-08A	155.3	0.0000
P-04-08B	155.3	0.0000
P-04-09A	88.6	0.0014
P-04-09B	88.6	0.0000
P-04-10A	110.9	0.0056
P-04-11A	137.8	0.0000
P-04-11B	137.8	0.0000
P-04-12A	225.3	0.0000
P-04-13A	116.0	0.0009
P-04-13B	116.0	0.0000
P-04-13C	116.1	0.0000
P-04-14A	62.4	0.0000
P-04-14B	62.4	0.0000
P-06-04A	137.8	0.0006
P-06-04B	137.8	0.0000
P-08-03A	117.3	0.0000
P-08-03B	117.3	0.0015
P-08-04	179.8	0.0000
P-08-06	135.5	0.0000
P-08-07	154.7	0.0000
PW-08-01	132.3	0.0000
PW-08-02	169.8	0.0000
PW-08-04	267.6	0.0081

Tabulated Data Used to Produce Figure U-16
(Part 5 of 5)

PW-08-04 26.5-Hour Pump Test		
Well ID	Azimuth from Pumping Well (degrees)	Log of Normalized Drawdown (log-feet)
MW-04-111	152.6	0.0000
MW-05-01	153.0	0.0000
MW-05-02	152.8	0.0000
MW-05-03	152.5	0.0000
MW-05-04	152.3	0.0000
MW-05-05	153.0	0.0000
MW-06-01	127.2	0.0000
MW-06-02	145.7	0.0017
MW-207	141.6	0.0000
MW-223A	169.9	0.0000
MW-223B	169.8	0.0000
MW-227	171.0	0.0000
MW-302R	160.0	0.0000
MW-304A	139.5	0.0000
OW-06-05	145.4	0.0019
OW-06-06	146.0	0.0000
OW-06-07	146.4	0.0036
OW-06-08	147.8	0.0000
OW-06-09	148.3	0.0000
OW-06-10	148.5	0.0013
P-04-05A	134.3	0.0000
P-04-05B	134.3	0.0000
P-04-06A	149.8	0.0000
P-04-06B	149.8	0.0000
P-04-07A	171.7	0.0000
P-04-07B	171.7	0.0000
P-04-07C	171.8	0.0000
P-04-08A	132.8	0.0000
P-04-08B	132.8	0.0000
P-04-09A	87.9	0.0058
P-04-09B	87.9	0.0000
P-04-10A	102.0	0.0254
P-04-10B	102.0	0.0000
P-04-11A	124.8	0.0000
P-04-11B	124.8	0.0000
P-04-12A	183.8	0.0618
P-04-12C	183.6	0.0014
P-04-13A	111.1	0.0013
P-04-13B	111.1	0.0000
P-04-13C	111.2	0.0000
P-04-14A	68.9	0.0000
P-04-14B	68.9	0.0000
P-06-04A	124.8	0.0000
P-06-04B	124.8	0.0000
P-08-03A	107.3	0.0000
P-08-03B	107.3	0.0000
P-08-04	160.2	0.0000
P-08-06	125.7	0.0000
P-08-07	144.0	0.0000
PW-08-01	122.7	0.0032
PW-08-02	154.5	0.0017
PW-08-03	87.6	0.0913

APPENDIX V

FIGURE V-5S
GROUNDWATER PARTICLE PATHWAYS FOR MODEL CALIBRATION



N

1000 0 2000 FEET

LEGEND

GROUNDWATER PARTICLE ORIGIN

PROJECTED GROUNDWATER PARTICLE PATHWAY (DOWNWARD)

PROJECTED GROUNDWATER PARTICLE PATHWAY (UPWARD)

NOTE: MODEL RECHARGE BASED ON CALIBRATION CONDITIONS (i.e. 2009)

FIGURE V–5S
GROUNDWATER PARTICLE PATHWAYS
FOR MODEL CALIBRATION
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

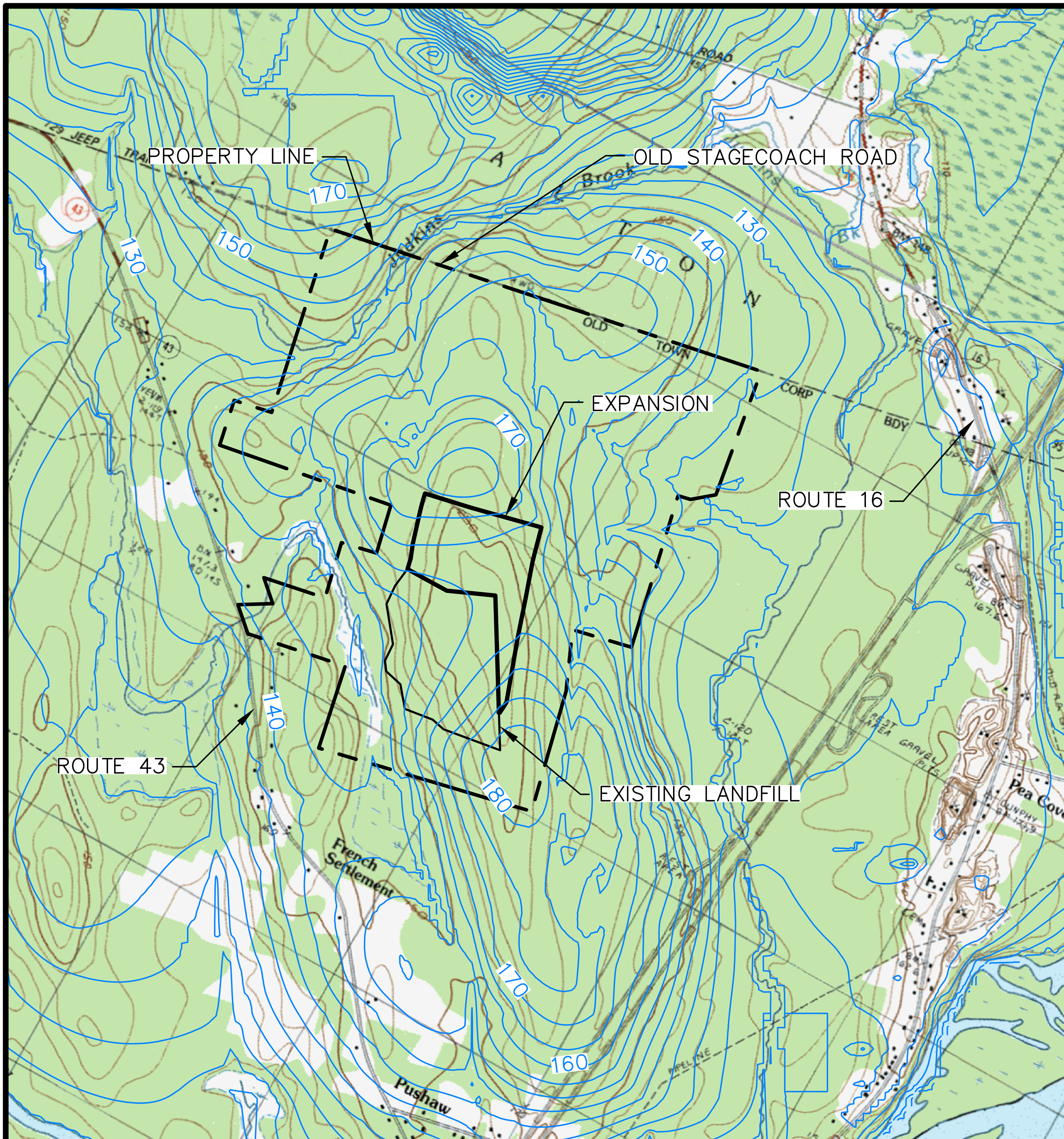
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: V–2 discretization LMN: V–5S CTB: Particle–Path REV: 2/17/16

\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\GWModel\figures\V-2 discretization.dwg, 2/17/2016 3:48:20 PM, paf

APPENDIX V

FIGURE V-6S
GROUNDWATER TABLE WITH RECHARGE CUTOFF



LEGEND

170 GROUNDWATER HEAD EQUIPOTENTIAL CONTOURS (5 FEET INTERVALS)

NOTE: RECHARGE EQUALS ZERO INCHES PER YEAR OVER EXISTING LANDFILL AND EXPANSION AREA.

1000 0 2000 FEET

FIGURE V-6S
GROUNDWATER TABLE WITH RECHARGE CUTOFF
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: V-2 discretization LMN: V-6S CTB: Particle-Path REV: 2/17/16

\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\GWModel\figures\V-2 discretization.dwg, 2/17/2016 2:45:13 PM, paf

SME-4

VOLUME II, VOLUME III AND VOLUME V
UPDATED TABLES AND APPENDICES

- **VOLUME II, TABLE 6-2 ANALYTICAL PROGRAM**
- **VOLUME II, TABLE 7-3 CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS- EXISTING CONDITIONS**
- **VOLUME II, TABLE 7-4 CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS-FUTURE CONDITIONS**
- **APPENDIX X, UPDATED PRINTOUTS FOR THE TRAVEL TIME ANALYSIS**
- **APPENDIX X, ADDITIONAL PRINTOUTS AS REQUESTED BY DEP COMMENT ON PAGE 7-12, 7.5 SENSITIVITY ANALYSIS**
- **VOLUME III, TABLE 4-3 SUMMARY OF CONTAMINANT TRANSPORT ANALYSIS**
- **VOLUME IV, APPENDIX I TABLE 3-2 SURFACE WATER, PORE-WATER, LEACHATE, UNDERDRAIN, AND LEAK DETECTION MONITORING LOCATIONS**
- **VOLUME IV, APPENDIX I TABLE 4-1 ANALYTICAL PROGRAM**

VOLUME II

TABLE 6-2
ANALYTICAL PROGRAM

TABLE 6-2

ANALYTICAL PROGRAM

Water Quality Parameter	Method	PQL[1] (mg/l)
Total Dissolved Solids (TDS)	STM 2540C	10
Total Suspended Solids (TSS)	STM 2540D	4
Ammonia (NH3-N)	STM 4500 NH3 E	0.5
Arsenic (As)	SW846/6010B/3010A	0.005
Calcium (Ca)	SW846/6010B/3010A	0.3
Iron (Fe)	SW846/6010B/3010A	0.05
Magnesium (Mg)	SW846/6010B/3010A	0.3
Manganese (Mn)	SW846/6010B/3010A	0.05
Potassium (K)	SW846/6010B/3010A	0.3
Sodium (Na)	SW846/6010B/3010A	0.3
Total Organic Carbon (TOC)	SW846/9060A	2.0
Chloride (Cl^-)	SW846/E300/9056	1.0
Sulfate (SO_4)	SW846/E300/9056	2.0
Volatile Organic Compounds (VOCs)[3]	U.S.EPA 8260B	0.001 – 0.01
Sulfide	SW846/9030B	2.5
Total Kjeldahl Nitrogen (TKN)[4]	STM 4500 NH_3E	0.3
Total Phosphorous[5]	U.S.EPA 365.3	0.04
Biological Oxygen Demand (BOD)[6]	STM 5210B	5
Copper (Cu)	SW846/6010B/3010A	0.003
Bromide	SW9056	0.1
Nitrate and Nitrite	EPA 353.2	0.05
Total Alkalinity	STM 2320B	1.5
Boron	EPA-200.8	0.05
Methane	EPA 8015B (MOD RSK-175)	0.02
<u>Field Parameters</u>		
Groundwater Elevation	Field Measurement	NA
Specific Conductance	Field Measurement	NA
Dissolved Oxygen	Field Measurement	NA
pH	Field Measurement	NA
Temperature	Field Measurement	NA
Turbidity	Field Measurement (APHA 2130)	NA
Eh	Field Measurement	NA
Monitoring Well Pumping Rate	Field Measurement	NA
Surface Water Flow Rate	Field Measurement	NA
Field Observations	Field Observations	NA

TABLE 6-2 (cont'd)

Notes:
1. Practical Quantitation Limits (PQLs) have been defined by U.S.EPA as up to 10 times the method or instrument detection limit and therefore may vary between laboratories.
2. NA = Not Applicable.
3. VOCs are the 47 organic constituents listed in Appendix I of 40 CFR Part 258. PQLs for VOCs are reported as μg/L. Only included in the Site Characterization Monitoring
4. Monitoring wells and leachate only.
5. Surface waters and underdrain only.
6. Surface waters only

Method Reference: The analytical methods selected are presented in Test Methods for Evaluating Solid Waste, OSWER, SW-846, Third Edition, as revised; Methods for Chemical Analysis of Water and Wastes, EMSL, EPA-600/4-79-020, revised March 1983; and Standard Methods for the Examination of Water and Wastewater, APHA, 19th Edition, 1995. Equivalent and appropriate analytical methods may be substituted with Juniper Ridge Landfill approval, e.g. manual for automated and vice versa.

VOLUME II

TABLE 7-3
CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS - EXISTING CONDITIONS

TABLE 7-3

CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS – EXISTING CONDITIONS

Landfill Node	Site Sensitive Receptors	Offset Credits (Yrs)	Imported Soils (Yrs)	Calculated Travel Time In Soil And Bedrock (Yrs)	Total Travel Time (Yrs)
Cell 11 Southern End	Point A	3	3	10.5	15.5
Center of Cell 11	Point B	2	3	3.9	8.9
Center of Cell 12	Point C	2	3	11.3	16.3
Center of Cell 13	Point C	2	3	11.0	16.0
Cell 13 Leachate Sump	Point C	2	3	35.8	41.8
Center of Cell 14	Point D	3	3	47.7	53.7
Center of Cell 14	Point E	3	3	3.3	9.3
Center of Cell 15	Point F	2	3	1.2	6.2
Center of Cell 16	Point G	2	3	4.7	9.7
Cell 16 Leachate Sump.	Point G	3	3	10.3	16.3

VOLUME II

TABLE 7-4
CALCULATED TRAVEL TIME TO SITE SENSITIVE RECEPTORS –
FUTURE CONDITIONS

TABLE 7-4

CALCULATED TRAVEL TIMES TO SITE SENSITIVE RECEPTORS – FUTURE CONDITIONS

Landfill Location Of Origin	Site Sensitive Receptors	Offset Credits (Yrs)	Imported Soils (Yrs)	Calculated Travel Time In Soil And Bedrock (Yrs)	Total Travel Time (Yrs)
Cell 11 Southern End	Point A	3	3	10.5	15.5
Center of Cell 11	Point B	2	3	3.9	8.9
Center of Cell 12	Point C	2	3	11.4	16.4
Center of Cell 13	Point C	2	3	11.2	16.2
Cell 13 Leachate Sump	Point C	2	3	36.1	42.1
Center of Cell 14	Point D	3	3	62.2	68.2
Center of Cell 14	Point E	3	3	17.7	23.7
Center of Cell 15	Point F	2	3	1.4	6.4
Center of Cell 16	Point G	2	3	5.3	10.3
Cell 16 Leachate Sump.	Point G	3	3	10.3	16.3

APPENDIX X

UPDATED PRINTOUTS FOR THE TRAVEL TIME ANALYSIS

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness		Effective Porosity	Hydraulic Conductivity			Conversions	
Imported Soil Layer	t_{ISL} =	1 ft	n_{ISL} = 0.39	k_{ISL} = 1.0E-07	cm/sec	1.0E-01 ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below	n_{TILL} = 0.25	k_{TILL} = 9.4E-06	cm/sec	9.7E+00 ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below	n_{BR} = 0.001000	k_{BR} = 3.5E-05	cm/sec	3.6E+01 ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.5	3.9	11.3	11.0	35.8	47.7	3.3	1.2	4.7	10.3
Total Travel Time to Site Sensitive Receptor	**Years**		**16.5**	**8.9**	**16.3**	**16.0**	**40.8**	**53.7**	**9.3**	**6.2**	**9.7**	**16.3**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

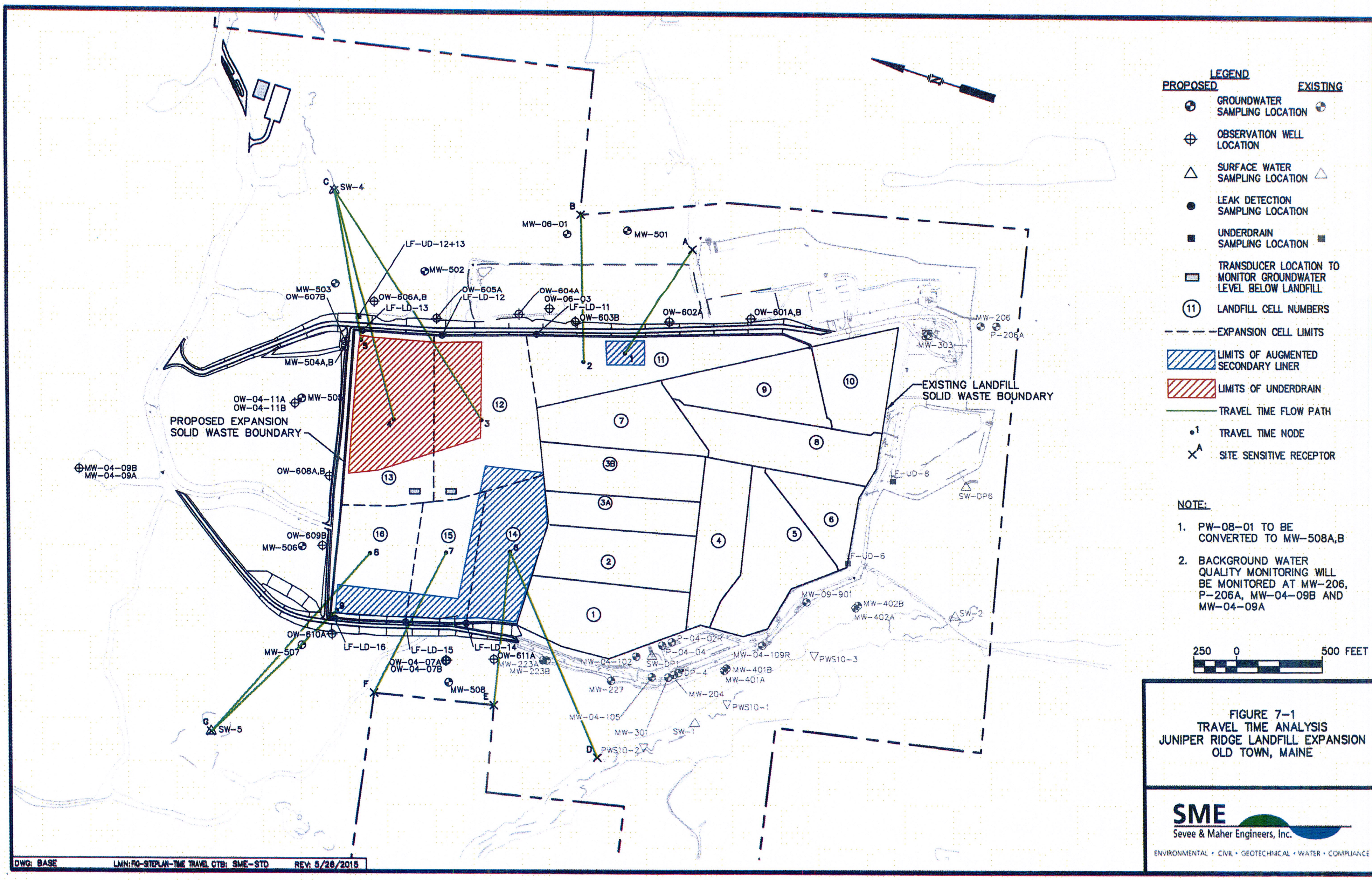
TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.5**	**3.9**	**11.3**	**11.0**	**35.8**	**47.7**	**3.3**	**1.2**	**4.7**	**10.3**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.


LEGEND
PROPOSED
EXISTING
GROUNDWATER SAMPLING LOCATION
OBSERVATION WELL LOCATION
SURFACE WATER SAMPLING LOCATION
LEAK DETECTION SAMPLING LOCATION
UNDERDRAIN SAMPLING LOCATION
TRANSDUCER LOCATION TO MONITOR GROUNDWATER LEVEL BELOW LANDFILL
LANDFILL CELL NUMBERS
EXPANSION CELL LIMITS
LIMITS OF AUGMENTED SECONDARY LINER
LIMITS OF UNDERDRAIN
TRAVEL TIME FLOW PATH
TRAVEL TIME NODE
SITE SENSITIVE RECEPTOR
NOTE:
1. PW-08-01 TO BE CONVERTED TO MW-508A,B
2. BACKGROUND WATER QUALITY MONITORING WILL BE MONITORED AT MW-206, P-206A, MW-04-09B AND MW-04-09A
250 0 500 FEET
EXISTING LANDFILL SOLID WASTE BOUNDARY
PROPOSED EXPANSION SOLID WASTE BOUNDARY
SW-4
MW-06-01
MW-501
LF-UD-12+13
MW-502
MW-503
OW-607B
OW-606A,B
LF-LD-13
OW-605A
LF-LD-12
OW-604A
OW-06-03
LF-LD-11
OW-603B
OW-602A
OW-601A,B
MW-206
P-206A
MW-303
MW-504A,B
OW-04-11A
OW-04-11B
MW-04-09B
MW-04-09A
OW-608A,B
OW-609B
MW-506
OW-610A
MW-507
LF-LD-16
LF-LD-15
LF-LD-14
OW-04-07A
OW-04-07B
OW-611A
MW-223A
MW-223B
MW-508
MW-227
MW-04-102
SW-DP1
P-04-02R
P-04-04
MW-04-109R
MW-401B
MW-401A
MW-204
MW-04-105
MW-301
SW-1
PWS10-1
PWS10-2
PWS10-3
MW-09-901
MW-402B
MW-402A
SW-2
LF-UD-6
LF-UD-8
SW-DP6
SW-5
FIGURE 7-1
TRAVEL TIME ANALYSIS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
DWG: BASE
LMN: FIG-SITEPLAN-TIME TRAVEL CTB: SME-STD
REV: 5/28/2015

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

Hydraulic Gradient Calculation for Dry Till

SITE: Juniper Ridge Landfill
Date: March 4, 2016
Calc. By: BBJ

PROJECT: Expansion Application
PROJECT No.: 14101.00
Ckd. By: MSB

LOCATIONS: Only applys when till is dry.

COMMENTS:

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: Determine the hydraulic gradient through till and fill soils, below the Imported Soil Layer; where the flow rate vertically through the Till is controlled by the leakage through the Imported Soil Layer.

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity						Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr			
Till (Native and recompacted as Fill)				n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr		3.2E+07	sec/yr
Bedrock (horizontal)				n_{BR} =	0.001	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr		30.48	cm/ft

Determine Seepage Rate through the Imported Soil Layer; the time of travel through that layer; and the hydraulic gradient through the Till (Fill and Native)

Nodes (Refer to Figure 7-1 in Volume II of the Application)

Parameter	Units	Symbol or Equation	Value
Imported Soil Layer Thickness	ft	$t_{ISL} = \Delta L_{ISL}$	1.0
Change in Head across Imported Soil Layer (**Delta H)** (For a free-draining Imported Soil Layer $\Delta H = t_{EBS}$)	ft	ΔH_{ISL}	1.0
Hydraulic Gradient (Imported Soil Layer)	ft/ft	$i_{ISL} = \Delta H_{ISL}/\Delta L_{ISL}$	1.0

Travel Time through Imported Soil Layer:

Travel Time through Imported Soil Layer (unit gradient)	Years	$TT_{ISL\_UG} = (\Delta L_{SISL} \times n_{ISL}) / (K_{ISL} \times i_{ISL})$	**3.77**	**<-- Travel Time through the Imported Soil Layer (unit gradient assumption).**
Flow Rate through Imported Soil Layer	ft/year	$q_{ISL} = (k_{ISL} \times i_{ISL})$	0.10	

The Flow Rate in the Imported Soil Layer sets the flow rate in the underlying Till

Flow Rate through Till	ft/year	$q_{TILL} = q_{ISL}$	0.10	
Gradient in Till	ft/ft	$i_{TILL} = q_{TILL}/k_{TILL}$	0.0106	**<--Applied in Travel Time Calculation for "Dry Till".**

This value is used in Travel Time Calculations as i_{BRS} in Post Closure Analysis.

Determine Seepage Rate through the Imported Soil Layer; the time of travel through that layer; Assuming that the leakage is due to the Design Leakage Rate

Parameter	Units	Symbol or Equation	Value				
Imported Soil Layer Thickness	ft	$t_{ISL} = \Delta L_{ISL}$	1.0				
Design Leakage Rate	gal/acre/day	DLR =	4.60	224.62	ft^3/yr	7.48	gal/ft^3
Hydraulic Gradient (Imported Soil Layer)	ft/ft	i_{ISL} = DLR / k_{ISL} / 43560	0.0498				

Travel Time through Imported Soil Layer:

Travel Time through Imported Soil Layer (Design Leakage Rate)	Years	$TT_{ISL\_DLR} = (\Delta L_{ISL} \times n_{ISL}) / (K_{ISL} \times i_{ISL})$	**75.63**	**<-- Travel Time through the Imported Soil Layer (Design Leakage Rate).**

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

POST-CLOSURE CONDITIONS - TIME OF TRAVEL (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy)	Proj #: 14101.00	Calc by: BBJ
Client: NEWSME	Date: March 4, 2016	Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to Sensitive Receptors shown on Figure 7-1. Under the condition where the leak is driven by some leakage rate out from the bottom of the Imported Soil Layer

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity						
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.00100	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

	Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
				1	2	3	4	5	6	6	7	8	9
	Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
	Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
	Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
	Underdrain, Thickness	ft	T_{UD}	0.00	0.00	0.00	1.00	1.00	0.00	0.00	0.00	0.00	0.00
	Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
	Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
5 ft drop	Post-Closure Phreatic Surface, Elevation	ft, Elev	E_{WET-PS} - (Drop due to Liner Installation)	192.41	195.60	196.95	193.21	166.50	196.34	196.34	195.44	194.25	179.62
	Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 1)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
5 ft drop	Post-Closure Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ - (Drop due to Liner Installation) (Note 1)	192.41	191.25	192.67	180.00	165.00	195.00	195.00	187.94	181.80	179.62
				See Note 3					See Note 3	See Note 3			See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) Imported Soil.	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.5	3.9	11.4	11.2	36.1	62.2	17.7	1.4	5.3	10.3
Total Travel Time to Site Sensitive Receptor	**Years**		**16.5**	**8.9**	**16.4**	**16.2**	**41.1**	**68.2**	**23.7**	**6.4**	**10.3**	**16.3**

NOTES:

1. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map was used.

2. Drop due to Liner Installation, based on Groundwater Modeling presented in Appendix V of Volume II of the Application.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

POST-CLOSURE CONDITIONS - TIME OF TRAVEL (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Nodes (Refer to Figure 7-1 in Volume II of the Application)										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	No	No	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	195.60	196.95	193.21	166.50	Till is Dry	Till is Dry	195.44	194.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	191.25	192.67	180.00	165.00	Till is Dry	Till is Dry	187.94	181.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	212.14	191.25	192.67	180.00	165.00	200.90	200.90	189.55	181.80	188.78
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	Till is Dry	Till is Dry	5.89	12.45	Till is Dry
Future Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.0106	0.0106	0.35	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under **Wet Season** hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	17.55	17.55	1.22	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of iBRS = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

POST-CLOSURE CONDITIONS - TIME OF TRAVEL (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	191.25	192.67	180.00	165.00	200.90	200.90	189.55	181.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	NA	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		47.67	35.00	20.00	51.90			16.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		131.25					128.90	112.55		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.02	0.02	0.04	0.14	0.12	0.01	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.5	1.6	1.2	0.9	0.2	0.2	2.6	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.17	31.17	31.17	66.41			8.78	8.78
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.83	3.83	3.83	2.59			3.22	3.22
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.12	0.12	0.12	0.04			0.37	0.37
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.5**	**3.9**	**11.4**	**11.2**	**36.1**	**62.2**	**17.7**	**1.4**	**5.3**	**10.3**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

Summary of Sensitivity Analysis, JRL Expansion Application.
Values shown here include Offsets and Credits, Yielding Total Travel Time in Years to Site Sensitive Receptors

			Node:	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Site Sensitive Receptor:		A	B	C	C	C	D	E	F	G	G
		Sensitivity Parameters											
		Hydraulic Conductivity	Porosity										
Till[1]	LCL Till [2]	5.2 x 10-6	0.25	24.7	12.0	24.3	23.7	69.0	91.5	11.8	6.9	11.9	23.9
	Base Evaluation[3]	9.4 x 10-6		16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	UCL Till[4]	1.7 x 10-5		12.0	7.2	11.8	11.7	25.2	32.8	7.9	5.7	8.5	12.1
Bedrock	LCL BR[5]	2.9 x 10-5	0.001	16.6	9.0	16.5	16.2	41.0	53.9	9.3	6.2	10.1	16.5
	Base Evaluation[3]	3.5 x 10-5		16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	UCL BR[6]	4.2 x 10-5		16.5	8.9	16.0	15.7	40.7	53.6	9.2	6.1	9.4	16.1
Till[1]	High Till n[7]	9.4 x 10-6	0.3	18.6	9.7	18.3	17.9	47.8	63.1	9.9	6.4	10.2	18.2
	Base Evaluation[3]		0.25	16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	Low Till n[8]		0.18	13.7	7.9	13.5	13.3	31.1	40.6	8.4	5.9	9.0	13.7
Bedrock	High BR n[9]	3.5 x 10-5	0.016	22.3	11.3	36.4	36.5	54.9	67.2	11.9	9.2	40.3	30.4
	Base Evaluation[3]		0.001	16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	Low BR n[10]		0.000059	16.2	8.8	15.0	14.7	40.0	52.9	9.1	6.0	7.8	15.4

NOTES:

1. The hydraulic conductivity values used in this analysis are horizontal hydraulic conductivity measurements. As described in Section 5.1.4 of Volume II of the Application, the average K_H/K_V ratio of the soils on-site was calculated to be 63, so using the horizontal hydraulic conductivity provides a conservative estimate of travel time, since the horizontal hydraulic conductivity is higher than the measured vertical hydraulic conductivities and the travel time calculations assume vertical flow through the till soils.

2. Travel Time (LCL Till), assumes a combination of: the Upper Confidence Limit (UCL) for the mean horizontal hydraulic conductivity of the Till, as defined in Section 7.5 of Volume II of the Application; the porosity of the Till, as defined in Section 3.3.6 of Volume II of the Application; the GeoMean BR hydraulic conductivity value; and the bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

3. Travel Time for the Base Evaluation, assumes a combination of: the Geometric Mean of Till (GeoMean Till), determined from site-specific data using the more permeable horizontal hydraulic conductivity values of the Till, as described in Section 7.4 of Volume II of the Application; the porosity of the Till, as defined in Section 3.3.6 of Volume II of the Application; the GeoMean BR hydraulic conductivity value; and the bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

4. Travel Time (UCL Till), assumes a combination of: the Upper Confidence Limit (UCL) for the mean horizontal hydraulic conductivity of the Till, as defined in Section 7.5 of Volume II of the Application; the porosity of the Till, as defined in Section 3.3.6 of Volume II of the Application; the GeoMean BR hydraulic conductivity value; and the bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

5. Travel Time (LCL BR), assumes a combination of: the Geometric Lower Confidence Limit (LCL) for the geometric mean of the horizontal hydraulic conductivity of the Bedrock, as described in Section 7.5 of Volume II of the Application; the porosity of the Bedrock as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Till horizontal hydraulic conductivity value; and the Till porosity value, as defined in Section 3.3.6 of Volume II of the Application..

6. Travel Time (UCL BR), assumes a combination of: the Geometric Upper Confidence Limit (UCL) for the geometric mean of the horizontal hydraulic conductivity of the Bedrock, as described in Section 7.5 of Volume II of the Application; the porosity of the Bedrock as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Till horizontal hydraulic conductivity value; and the Till porosity value, as defined in Section 3.3.6 of Volume II of the Application.

7. Travel Time (High Till porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the porosity of the Till as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the Bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

8. Travel Time (Low Till porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the low porosity of the Till as defined in Section 7.5 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the Bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

9. Travel Time (High Bedrock porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the porosity of the Till as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the high Bedrock porosity value, as defined in Section 7.5 of Volume II of the Application.

10. Travel Time (Low Bedrock porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the porosity of the Till as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the low Bedrock porosity value, as defined in Section 7.5 of Volume II of the Application.

APPENDIX X

ADDITIONAL PRINTOUTS AS REQUESTED BY DEP COMMENT ON PAGE 7-12, 7.5 SENSITIVITY ANALYSIS

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

Summary of Sensitivity Analysis, JRL Expansion Application.
Values shown here include Offsets and Credits, Yielding Total Travel Time in Years to Site Sensitive Receptors

			Node:	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Site Sensitive Receptor:		A	B	C	C	C	D	E	F	G	G
		Sensitivity Parameters											
		Hydraulic Conductivity	Porosity										
Till[1]	LCL Till [2]	5.2 x 10-6	0.25	24.7	12.0	24.3	23.7	69.0	91.5	11.8	6.9	11.9	23.9
	Base Evaluation[3]	9.4 x 10-6		16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	UCL Till[4]	1.7 x 10-5		12.0	7.2	11.8	11.7	25.2	32.8	7.9	5.7	8.5	12.1
Bedrock	LCL BR[5]	2.9 x 10-5	0.001	16.6	9.0	16.5	16.2	41.0	53.9	9.3	6.2	10.1	16.5
	Base Evaluation[3]	3.5 x 10-5		16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	UCL BR[6]	4.2 x 10-5		16.5	8.9	16.0	15.7	40.7	53.6	9.2	6.1	9.4	16.1
Till[1]	High Till n[7]	9.4 x 10-6	0.3	18.6	9.7	18.3	17.9	47.8	63.1	9.9	6.4	10.2	18.2
	Base Evaluation[3]		0.25	16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	Low Till n[8]		0.18	13.7	7.9	13.5	13.3	31.1	40.6	8.4	5.9	9.0	13.7
Bedrock	High BR n[9]	3.5 x 10-5	0.016	22.3	11.3	36.4	36.5	54.9	67.2	11.9	9.2	40.3	30.4
	Base Evaluation[3]		0.001	16.5	8.9	16.3	16.0	40.8	53.7	9.3	6.2	9.7	16.3
	Low BR n[10]		0.000059	16.2	8.8	15.0	14.7	40.0	52.9	9.1	6.0	7.8	15.4

NOTES:

1. The hydraulic conductivity values used in this analysis are horizontal hydraulic conductivity measurements. As described in Section 5.1.4 of Volume II of the Application, the average K_H/K_V ratio of the soils on-site was calculated to be 63, so using the horizontal hydraulic conductivity provides a conservative estimate of travel time, since the horizontal hydraulic conductivity is higher than the measured vertical hydraulic conductivities and the travel time calculations assume vertical flow through the till soils.

2. Travel Time (LCL Till), assumes a combination of: the Upper Confidence Limit (UCL) for the mean horizontal hydraulic conductivity of the Till, as defined in Section 7.5 of Volume II of the Application; the porosity of the Till, as defined in Section 3.3.6 of Volume II of the Application; the GeoMean BR hydraulic conductivity value; and the bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

3. Travel Time for the Base Evaluation, assumes a combination of: the Geometric Mean of Till (GeoMean Till), determined from site-specific data using the more permeable horizontal hydraulic conductivity values of the Till, as described in Section 7.4 of Volume II of the Application; the porosity of the Till, as defined in Section 3.3.6 of Volume II of the Application; the GeoMean BR hydraulic conductivity value; and the bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

4. Travel Time (UCL Till), assumes a combination of: the Upper Confidence Limit (UCL) for the mean horizontal hydraulic conductivity of the Till, as defined in Section 7.5 of Volume II of the Application; the porosity of the Till, as defined in Section 3.3.6 of Volume II of the Application; the GeoMean BR hydraulic conductivity value; and the bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

5. Travel Time (LCL BR), assumes a combination of: the Geometric Lower Confidence Limit (LCL) for the geometric mean of the horizontal hydraulic conductivity of the Bedrock, as described in Section 7.5 of Volume II of the Application; the porosity of the Bedrock as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Till horizontal hydraulic conductivity value; and the Till porosity value, as defined in Section 3.3.6 of Volume II of the Application..

6. Travel Time (UCL BR), assumes a combination of: the Geometric Upper Confidence Limit (UCL) for the geometric mean of the horizontal hydraulic conductivity of the Bedrock, as described in Section 7.5 of Volume II of the Application; the porosity of the Bedrock as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Till horizontal hydraulic conductivity value; and the Till porosity value, as defined in Section 3.3.6 of Volume II of the Application.

7. Travel Time (High Till porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the porosity of the Till as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the Bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

8. Travel Time (Low Till porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the low porosity of the Till as defined in Section 7.5 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the Bedrock porosity value, as defined in Section 3.3.6 of Volume II of the Application.

9. Travel Time (High Bedrock porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the porosity of the Till as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the high Bedrock porosity value, as defined in Section 7.5 of Volume II of the Application.

10. Travel Time (Low Bedrock porosity (n)), assumes a combination of: the mean horizontal hydraulic conductivity of the Till as defined in Section 7.4 of Volume II of the Application; the porosity of the Till as defined in Section 3.3.6 of Volume II of the Application; the GeoMean Bedrock hydraulic conductivity value; and the low Bedrock porosity value, as defined in Section 7.5 of Volume II of the Application.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: LCL Till | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity				Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07 cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	5.2E-06 cm/sec	5.4E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05 cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	18.7	7.0	19.3	18.7	64.0	85.5	5.8	1.9	6.9	17.9
Total Travel Time to Site Sensitive Receptor	**Years**		**24.7**	**12.0**	**24.3**	**23.7**	**69.0**	**91.5**	**11.8**	**6.9**	**11.9**	**23.9**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 SENSITIVITY ANALYSIS: LCL Till Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		**Nodes (Refer to Figure 7-1 in Volume II of the Application)**	**Cell 11 Southern End**	**Cell 11 Center**	**Cell 12 Center**	**Cell 13 Center**	**Cell 13 Leachate Sump**	**Cell 14 Center**	**Cell 14 Center**	**Cell 15 Center**	**Cell 16 Center**	**Cell 16 Leachate Sump**
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	18.33	6.81	6.17	5.56	51.33	5.58	5.58	1.73	3.72	15.80

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: LCL Till | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	E_{EX-GS}	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS-BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	E_{HRS-BR} (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS-MM} = E_{HRS-BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX-GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS-BR} - E_{EX-GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			11.8	11.8	11.8	79.1			1.1	1.1
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**18.7**	**7.0**	**19.3**	**18.7**	**64.0**	**85.5**	**5.8**	**1.9**	**6.9**	**17.9**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 SENSITIVITY ANALYSIS: Base Evaluation Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB
NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness		Effective Porosity		Hydraulic Conductivity			Conversions
Imported Soil Layer	t_{ISL} =	1 ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07 cm/sec	1.0E-01 ft/yr	
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below	n_{TILL} =	0.25	k_{TILL} =	9.4E-06 cm/sec	9.7E+00 ft/yr	3.2E+07 sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below	n_{BR} =	0.001000	k_{BR} =	3.5E-05 cm/sec	3.6E+01 ft/yr	30.48 cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.5	3.9	11.3	11.0	35.8	47.7	3.3	1.2	4.7	10.3
Total Travel Time to Site Sensitive Receptor	**Years**		**16.5**	**8.9**	**16.3**	**16.0**	**40.8**	**53.7**	**9.3**	**6.2**	**9.7**	**16.3**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.
2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 SENSITIVITY ANALYSIS: Base Evaluation Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		**Nodes (Refer to Figure 7-1 in Volume II of the Application)**	**Cell 11 Southern End**	**Cell 11 Center**	**Cell 12 Center**	**Cell 13 Center**	**Cell 13 Leachate Sump**	**Cell 14 Center**	**Cell 14 Center**	**Cell 15 Center**	**Cell 16 Center**	**Cell 16 Leachate Sump**
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 SENSITIVITY ANALYSIS: Base Evaluation Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.5**	**3.9**	**11.3**	**11.0**	**35.8**	**47.7**	**3.3**	**1.2**	**4.7**	**10.3**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: UCL Till | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	1.7E-05	cm/sec	1.8E+01	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	6.0	2.2	6.8	6.7	20.2	26.8	1.9	0.7	3.5	6.1
Total Travel Time to Site Sensitive Receptor	**Years**		**12.0**	**7.2**	**11.8**	**11.7**	**25.2**	**32.8**	**7.9**	**5.7**	**8.5**	**12.1**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: UCL Till | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	5.61	2.08	1.89	1.70	15.70	1.71	1.71	0.53	1.14	4.83

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: UCL Till | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			3.6	3.6	3.6	24.2			0.3	0.3
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**6.0**	**2.2**	**6.8**	**6.7**	**20.2**	**26.8**	**1.9**	**0.7**	**3.5**	**6.1**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: LCL BR | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity			Conversions
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07 cm/sec	1.0E-01 ft/yr	
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06 cm/sec	9.7E+00 ft/yr	3.2E+07 sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	2.9E-05 cm/sec	3.0E+01 ft/yr	30.48 cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.6	4.0	11.5	11.2	36.0	47.9	3.3	1.2	5.1	10.5
Total Travel Time to Site Sensitive Receptor	**Years**		**16.6**	**9.0**	**16.5**	**16.2**	**41.0**	**53.9**	**9.3**	**6.2**	**10.1**	**16.5**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: LCL BR | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: LCL BR | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.5	0.2	1.6	1.7	1.1	1.1	0.2	0.2	2.5	1.1
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.6**	**4.0**	**11.5**	**11.2**	**36.0**	**47.9**	**3.3**	**1.2**	**5.1**	**10.5**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: UCL BR | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity				Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07 cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06 cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	4.2E-05 cm/sec	4.3E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.5	3.9	11.0	10.7	35.7	47.6	3.2	1.1	4.4	10.1
Total Travel Time to Site Sensitive Receptor	**Years**		**16.5**	**8.9**	**16.0**	**15.7**	**40.7**	**53.6**	**9.2**	**6.1**	**9.4**	**16.1**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatric Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: UCL BR | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: UCL BR | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Bedrock (horizontal lengths through which a hypothetical leak travels):

Parameter	Units	Hypothetical Leak Location "Node" (See Figure 7-1)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.3	0.1	1.1	1.1	0.8	0.7	0.1	0.2	1.7	0.8
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.5**	**3.9**	**11.0**	**10.7**	**35.7**	**47.6**	**3.2**	**1.1**	**4.4**	**10.1**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: High Till n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.3	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Parameter	Units	Symbol or Equation	1	2	3	4	5	6	6	7	8	9
Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	12.6	4.7	13.3	12.9	42.8	57.1	3.9	1.4	5.2	12.2
Total Travel Time to Site Sensitive Receptor	**Years**		**18.6**	**9.7**	**18.3**	**17.9**	**47.8**	**63.1**	**9.9**	**6.4**	**10.2**	**18.2**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: High Till n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB})	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	12.17	4.52	4.09	3.69	34.07	3.70	3.70	1.15	2.47	10.49

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: High Till n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Bedrock (horizontal lengths through which a hypothetical leak travels):		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	E_{EX-GS}	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS-BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	E_{HRS-BR} (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS-MM} = E_{HRS-BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS-BR} - E_{HRS-MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX-GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS-BR} - E_{EX-GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			7.8	7.8	7.8	52.5			0.7	0.7
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**12.6**	**4.7**	**13.3**	**12.9**	**42.8**	**57.1**	**3.9**	**1.4**	**5.2**	**12.2**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) Proj #: 14101.00 SENSITIVITY ANALYSIS: Low Till n Calc by: BBJ
Client: NEWSME Date: March 4, 2016 Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)		Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft		n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below			n_{TILL} =	0.18	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below			n_{BR} =	0.001000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	7.7	2.9	8.5	8.3	26.1	34.6	2.4	0.9	4.0	7.7
Total Travel Time to Site Sensitive Receptor	**Years**		**13.7**	**7.9**	**13.5**	**13.3**	**31.1**	**40.6**	**8.4**	**5.9**	**9.0**	**13.7**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: Low Till n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS-PS-SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS-PS-SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	7.30	2.71	2.46	2.21	20.44	2.22	2.22	0.69	1.48	6.29

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: Low Till n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Bedrock (horizontal lengths through which a hypothetical leak travels):		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.4	0.2	1.3	1.4	0.9	0.9	0.2	0.2	2.0	0.9
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			4.7	4.7	4.7	31.5			0.4	0.4
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**7.7**	**2.9**	**8.5**	**8.3**	**26.1**	**34.6**	**2.4**	**0.9**	**4.0**	**7.7**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: High BR n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.016000	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**	1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Parameter	Units	Symbol or Equation	1	2	3	4	5	6	6	7	8	9
Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	16.3	6.3	31.4	31.5	49.9	61.2	5.9	4.2	35.3	24.4
Total Travel Time to Site Sensitive Receptor	**Years**		**22.3**	**11.3**	**36.4**	**36.5**	**54.9**	**67.2**	**11.9**	**9.2**	**40.3**	**30.4**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: High BR n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB})	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) — Proj #: 14101.00 — SENSITIVITY ANALYSIS: High BR n — Calc by: BBJ
Client: NEWSME — Date: March 4, 2016 — Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Bedrock (horizontal lengths through which a hypothetical leak travels):		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	6.2	2.5	21.5	21.9	14.9	14.4	2.8	3.2	32.7	15.0
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**16.3**	**6.3**	**31.4**	**31.5**	**49.9**	**61.2**	**5.9**	**4.2**	**35.3**	**24.4**

NOTES:
1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: Low BR n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

PURPOSE: To calculate the time of travel for a hypothetical drop of liquid to travel from the base of the Imported Soil Layer to the Sensitive Receptors shown on Figure 7-1.

INPUT PARAMETERS:

Soil Layer Name (Top Down)	Layer Thickness			Effective Porosity		Hydraulic Conductivity					Conversions	
Imported Soil Layer	t_{ISL} =	1	ft	n_{ISL} =	0.39	k_{ISL} =	1.0E-07	cm/sec	1.0E-01	ft/yr		
Till (Native and recompacted as Fill)	T_{TILL} =	Varies, based on Geology, see below		n_{TILL} =	0.25	k_{TILL} =	9.4E-06	cm/sec	9.7E+00	ft/yr	3.2E+07	sec/yr
Bedrock (horizontal)	L_{BR} =	Varies, based on Geology, see below		n_{BR} =	0.000059	k_{BR} =	3.5E-05	cm/sec	3.6E+01	ft/yr	30.48	cm/ft

Parameter	Units	Nodes (Refer to Figure 7-1 in Volume II of the Application) / Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
			1	2	3	4	5	6	6	7	8	9
Existing Ground Surface	ft, Elev	E_{EX-GS}	212.66	213.62	210.00	200.00	176.39	207.00	207.00	204.00	201.29	190.61
Base of Grubbing, Elevation	ft, Elev	E_{GRUB}	211.66	212.63	209.66	199.00	175.38	206.67	206.67	203.01	200.29	189.60
Base Grade of Secondary Liner System (or Base of Imported Soil Layer), Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain, Thickness	ft	T_{UD}	0	0	0	1	1	0	0	0	0	0
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Existing (Dry Season) Phreatic Surface, Elevation	ft, Elev	E_{DRY-PS}	192.94	193.25	201.00	192.28	166.11	198.88	198.88	196.36	193.16	181.88
Existing (Wet Season) Phreatic Surface, Elevation	ft, Elev	E_{WET-PS}	197.41	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	184.62
Existing (Dry Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{DRY-PS-SBR}$ (Note 1)	192.94	190.00	192.14	181.03	162.96	198.88	198.88	188.62	184.09	181.88
Existing (Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WET-PS-SBR}$ (Note 2)	197.41	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	184.62
			See Note 3									See Note 3

SUMMARY OF TRAVEL TIMES (see the following pages for details):

Parameter	Units	Symbol or Equation	1	2	3	4	5	6	6	7	8	9
Site Sensitive Receptors		Figure 7-1 in Volume II of the Application	A	B	C	C	C	D	E	F	G	G
Offset Credits	Years	MEDEP 401.2.D(2) Secondary liner with leak detection.	3	2	2	2	2	3	3	2	2	3
Imported Soil Credits	Years	MEDEP 401.2.D(2) imported soil	3	3	3	3	3	3	3	3	3	3
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	Determined in the following pages (Value from bottom of page 3)	10.2	3.8	10.0	9.7	35.0	46.9	3.1	1.0	2.8	9.4
Total Travel Time to Site Sensitive Receptor	**Years**		**16.2**	**8.8**	**15.0**	**14.7**	**40.0**	**52.9**	**9.1**	**6.0**	**7.8**	**15.4**

NOTES:

1. Dry Season Potentiometric Surface in Shallow Bedrock is equal to the Dry Season Phreatic Surface when it is below the bedrock surface (Cell 14 Center and Cell 16 Sump), otherwise the Potentiometric surface map (Figure 5-8 in Volume II of the Application) was used.

2. Wet Season Potentiometric Surface in Shallow Bedrock is equal to the Wet Season Phreatic Surface when it is below the bedrock surface (Cell 16 Sump), otherwse the Potentiometric Surface map (Figure 5-8 in Volume II of the Application) was used.

3. The wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", unit gradient assumption applied to the Imported Soil Layer.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: Low BR n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO BEDROCK SURFACE

Soil Profile (vertical thickness through which a hypothetical leak travels, top down):		Nodes (Refer to Figure 7-1 in Volume II of the Application)	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Parameter	**Units**	**Symbol or Equation**										
Is Fill Soil Required (in addition to Underdrain)?		CUT or FILL	FILL	CUT	CUT	CUT	CUT	FILL	FILL	FILL	FILL	FILL
Fill Thickness is:	ft	If(FILL, $T_{FILL}=(E_{BASE}-T_{UD})-E_{GRUB}$)	2.34					0.75	0.75	1.64	0.73	1.40
Native Till Thickness	ft	$T_{TILL}=(E_{BASE}-T_{UD}-E_{BR})$	1.86	25.25	23.84	39.77	40.72	6.52	6.52	15.10	30.84	2.22
Delta L, (for hydraulic gradient calculation)	ft	$\Delta L=T_{FILL}+T_{TILL}$	4.20	25.25	23.84	39.77	40.72	7.27	7.27	16.74	31.57	3.62
Hydraulics:												
Base of Liner System, Elevation	ft, Elev	E_{BASE}	214.00	210.49	206.00	194.15	171.00	207.42	207.42	204.65	201.02	191.00
Underdrain Present		Yes or No	No	No	No	Yes	Yes	No	No	No	No	No
Head Driving Seepage	ft, Elev	E_{HDS} = (Note 1)	Till is Dry	200.60	201.95	198.21	171.50	201.34	201.34	200.44	199.25	Till is Dry
(Wet Season) Potentiometric Surface in Shallow Bedrock, Elevation	ft, Elev	$E_{WS\text{-}PS\text{-}SBR}$	Till is Dry	196.25	197.67	185.00	170.00	200.00	200.00	192.94	186.80	Till is Dry
Bedrock Surface, Elevation	ft, Elev	E_{BR}	212.14	185.24	182.16	153.38	129.28	200.90	200.90	189.55	170.18	188.78
Head Receiving Seepage, Elevation	ft, Elev	E_{HRS} = Max($E_{WS\text{-}PS\text{-}SBR}$ or E_{BR}) (Note 2)	Till is Dry	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	Till is Dry
Delta H, (for hydraulic gradient calculation)	ft/ft	$\Delta H = E_{HDS} - E_{HRS}$	Till is Dry	4.35	4.28	13.21	1.50	0.44	0.44	7.50	12.45	Till is Dry
Existing Hydraulic Gradient (Wet Season)	ft/ft	$i_{BRS} = \Delta H/\Delta L$ (Note 3)	0.0106	0.17	0.18	0.33	0.04	0.06	0.06	0.45	0.39	0.0106
Travel Time to Bedrock Surface:												
Travel Time to Bedrock Surface, under Wet Season hydraulic conditions	Years	$TT_{BRS} = (\Delta L \times n_{TILL}) / (K_{TILL} \times i_{BRS})$	10.14	3.76	3.41	3.08	28.40	3.09	3.09	0.96	2.06	8.74

NOTES:

1. Assumed to be the existing wet season potentiometric surface away from sumps. Where Till is Dry, See Note 3 on Page 1.
2. The Elevation Head Receiving Seepage is the highest of: a) the Bedrock Surface; or b) the Existing (Wet Season) Potentiometric Surface in Shallow Bedrock. When Till is NOT Dry.
3. Calculated as shown, unless the wet season phreatic surface is below the bedrock surface at this Node. So, under these conditions, no natural vertical seepage would occur in the Till. To provide flow through the Till, the seepage from the Imported Soil Layer was assumed to be the only source of vertical flow for this node. See "Hydraulic Gradient Calculation for Dry Till", for determination of i_{BRS} = 0.0106, when the flow rate through the Till equals the flow rate through the overlying and less permeable Imported Soil Layer under a unit gradient condition.

UPDATED PRINTOUTS FOR TRAVEL TIME ANALYSIS

EXISTING CONDITIONS - TRAVEL TIME CALCULATIONS (Base of Imported Soil Layer to Sensitive Receptors)

Project: Juniper Ridge Landfill, Expansion (9.35 Mcy) | Proj #: 14101.00 | SENSITIVITY ANALYSIS: Low BR n | Calc by: BBJ
Client: NEWSME | Date: March 4, 2016 | Ckd by: MSB

NOTE: Yellow shaded cells are input values. Non-shaded cells are calculated using the equation shown.

TRAVEL TIME TO SENSITIVE RECEPTORS (in Bedrock)

Parameter	Units	Symbol or Equation	Cell 11 Southern End	Cell 11 Center	Cell 12 Center	Cell 13 Center	Cell 13 Leachate Sump	Cell 14 Center	Cell 14 Center	Cell 15 Center	Cell 16 Center	Cell 16 Leachate Sump
Bedrock (horizontal lengths through which a hypothetical leak travels):		Hypothetical Leak Location "Node" (See Figure 7-1)										
Sensitive Receptor Location (See Figure 7-1)			A	B	C	C	C	D	E	F	G	G
Sensitive Receptor Type (See Table 7-1)			Southern Sandy Zone	Property Line	Surface Water	Surface Water	Surface Water	Surface Water	Property Line	Property Line	Surface Water	Surface Water
Ground Surface at Sensitive Receptor (OR Surface Water, Elevation)	ft, Elev	$E_{EX\text{-}GS}$	180.00	157.22	141.17	141.17	141.17	146.41	172.13	176.84	161.78	161.78
Bedrock Surface, Elevation	ft, Elev	E_{BR}	115.00	150.00	110.00	110.00	110.00	80.00	150.00	172.82	153.00	153.00
Delta L, Horizontal Length through Bedrock	ft	ΔL_{BR}	740	880	1600	1410	920	1300	900	920	1270	900
Hydraulics:												
Assumed Drawdown in at Property-Line Well	ft	ΔH_{WELL}		100					100	100		
Head Driving Seepage (in Bedrock)	ft, Elev	$E_{HDS\text{-}BR} = E_{HRS}$ (Note 1)	212.14	196.25	197.67	185.00	170.00	200.90	200.90	192.94	186.80	188.78
Head Receiving Seepage (in Bedrock)	ft, Elev	$E_{HRS\text{-}BR}$ (Note 2)	173.00	160.00	145.00	145.00	145.00	149.00	172.00	177.00	165.00	165.00
Man-Made Head	ft, Elev	$E_{HRS\text{-}MM} = E_{HRS\text{-}BR} - \Delta H_{WELL}$ (Note 3)	173.00	60.00	NA	NA	NA	NA	72.00	77.00	NA	NA
Delta H, (for hydraulic gradient calculation)	ft/ft	Natural Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}BR}$	39.14		52.67	40.00	25.00	51.90			21.80	23.78
		Man-Made Head: $\Delta H_{BR} = E_{HDS\text{-}BR} - E_{HRS\text{-}MM}$		136.25					128.90	115.94		
Hydraulic Gradient through Bedrock	ft/ft	$i_{BR} = \Delta H_{BR} / \Delta L_{BR}$	0.05	0.15	0.03	0.03	0.03	0.04	0.14	0.13	0.02	0.03
Travel Time through Bedrock (Horizontally):												
Travel Time Horizontally through Bed Rock, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{BR} = (\Delta L_{BR} \times n_{BR}) / (K_{BR} \times i_{BR})$	0.0	0.0	0.1	0.1	0.1	0.1	0.0	0.0	0.1	0.1
From Bedrock Vertically Upward to Surface Water:												
Native Till Thickness = Flow Length (ΔL_{TILL})	ft	$T_{TILL} = (E_{EX\text{-}GS} - E_{BR}) = \Delta L_{TILL}$			31.2	31.2	31.2	66.4			8.8	8.8
Delta H, (for hydraulic gradient calculation)	ft/ft	Head Through Till: $\Delta H_{TILL} = E_{HRS\text{-}BR} - E_{EX\text{-}GS}$			3.8	3.8	3.8	2.6			3.2	3.2
Existing Hydraulic Gradient through Till	ft/ft	$i_{TILL} = \Delta H_{TILL} / \Delta L_{TILL}$			0.1	0.1	0.1	0.0			0.4	0.4
Travel Time Vertically through Till, under **DRY SEASON** hydraulic conditions (Note 4)	Years	$TT_{TILL} = (\Delta L_{TILL} \times n_{TILL}) / (K_{TILL} \times i_{TILL})$			6.5	6.5	6.5	43.7			0.6	0.6
TOTAL TRAVEL TIME (Value shown on Page 1):												
Calculated Travel Time: Sum of Time to Bedrock Surface; Time through Bedrock; and if appropriate Time to Surfacewater	Years	$TT_{TOTAL} = TT + TT_{BR} + TT_{TILL}$	**10.2**	**3.8**	**10.0**	**9.7**	**35.0**	**46.9**	**3.1**	**1.0**	**2.8**	**9.4**

NOTES:

1. The head driving seepage horizontally through the bedrock is assumed to be equal to the Head Receiving Seepage from the previous page. See Note 3 on pages 1 and 2, for special conditions where till is dry.
2. The head receiving seepage (under Natural Conditions) is the potentiometric surface elevation in shallow bedrock (Wet Season). See Figure 5-8 in Volume II of the Application.
3. A Man-Made Water Level is assumed. Ex. A potential water supply well having a drawdown of 100 feet at sensitive Receptor A: EHRS-MM = 160 - 100 = 60
4. Assume that all flow is horizontal through bedrock to be conservative. Actual flow path would be longer and therefore take longer.

VOLUME III

TABLE 4-3 SUMMARY OF CONTAMINANT TRANSPORT ANALYSIS

TABLE 4-3

SUMMARY OF CONTAMINANT TRANSPORT ANALYSIS

Scenario[1]	Site Sensitive Receptor (Media)[2]	Parameter	Concentration, mg/L At Time of Years[6]		Time to Reach Applicable Criteria, Years	Time to Reach Steady-State, Years	Time in Years to Reach Maximum and (Concentration, mg/L)
			3	6			
Scenario 1 (Horizontal Flow in Till)	A (GW)	Nitrate		<0.00033	>30	>30	30 (0.06)
		Iron		<0.000070	>30	>30	30 (0.014)
		Alkalinity		<0.0020	>30	>30	30 (0.40)
		Chloride		<0.012	>30	>30	30 (2.4)
		Ammonia		<0.00067	>30	>30	30 (0.13)
		Arsenic		<0.0000002	>30	>30	30 (0.00003)
	B (GW)	Nitrate		<0.00033	>30	>30	30 (0.33)
		Iron		<0.000070	>30	>30	30 (0.07)
		Alkalinity		<0.0020	>30	>30	30 (2.0)
		Chloride		<0.012	>30	>30	30 (12)
		Ammonia		<0.00067	>30	>30	30 (0.66)
		Arsenic		<0.0000002	>30	>30	30 (0.00015)
	C (SW)	Alkalinity		<0.0020	>30	>30	30 (1.8)
		Nitrate		<0.00033	>30	>30	30 (0.3)
		Arsenic		<0.0000002	>30	>30	30 (0.00014)
		Chloride		<0.012	>30	>30	30 (11)
		Ammonia		<0.00067	>30	>30	30 (0.6)
		Iron		<0.00007	>30	>30	30 (0.063)
	D (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
	E (GW)	Nitrate		<0.00033	>30	>30	30 (0.66)
		Iron		<0.000070	>30	>30	30 (0.14)
		Alkalinity		<0.0020	>30	>30	30 (4)
		Chloride		<0.012	>30	>30	30 (24)
		Ammonia		<0.00067	>30	>30	30 (1.3)
		Arsenic		<0.0000002	>30	>30	30 (0.00030)
	F (GW)	Nitrate		<0.00033	>30	>30	30 (0.002)
		Iron		<0.000070	>30	>30	30 (0.00042)
		Alkalinity		<0.0020	>30	>30	30 (0.012)
		Chloride		<0.012	>30	>30	30 (0.071)
		Ammonia		<0.00067	>30	>30	30 (0.004)
		Arsenic		<0.0000002	>30	>30	30 (0.0000009)
	G (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
Scenario 1 (Horizontal Flow in Bedrock)	A (GW)	Nitrate		<0.00033	>30	>30	30 (5.9)
		Iron		<0.000070	>30	>30	30 (0.013)
		Alkalinity		<0.0020	>30	>30	30 (0.36)
		Chloride		<0.012	>30	>30	30 (2.1)
		Ammonia		<0.00067	>30	>30	30 (0.12)
		Arsenic		<0.0000002	>30	>30	30 (0.0027)
	B (GW)	Nitrate		<0.00033	Never	20	20 (9.8)
		Iron		<0.000070	Never	20	20 (2.1)
		Alkalinity		<0.0020	Never	20	20 (60)

TABLE 4-3 (cont'd)

Scenario[1]	Site Sensitive Receptor (Media)[2]	Parameter	Concentration, mg/L At Time of Years[6]		Time to Reach Applicable Criteria, Years	Time to Reach Steady-State, Years	Time in Years to Reach Maximum and (Concentration, mg/L)
			3	6			
		Chloride		<0.012	Never	20	20 (350)
		Ammonia		<0.00067	Never	20	20 (20)
		Arsenic		<0.0000002	Never	20	20 (0.0045)
	C (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
	D (SW)	Alkalinity		<0.0020	Never	23	23 (2.2)
		Nitrate		<0.00033	Never	23	23 (0.36)
		Arsenic		<0.0000002	Never	23	23 (0.00017)
		Chloride		<0.012	Never	23	23 (13)
		Ammonia		<0.00067	Never	23	23 (0.73)
		Iron		<0.00007	Never	23	23 (0.077)
	E (GW)	Nitrate		<0.00033	Never	22	22 (9.8)
		Iron		<0.000070	Never	22	22 (2.1)
		Alkalinity		<0.0020	Never	22	22 (60)
		Chloride		<0.012	Never	22	22 (350)
		Ammonia		<0.00067	Never	22	22 20)
		Arsenic		<0.0000002	Never	22	22 (0.0045)
	F (GW)	Nitrate		<0.00033	Never	23	23 (1.3)
		Iron		<0.000070	Never	23	23 (0.28)
		Alkalinity		<0.0020	Never	23	23 (8)
		Chloride		<0.012	Never	23	23 (47)
		Ammonia		<0.00067	Never	23	23 (2.6)
		Arsenic		<0.0000002	Never	23	23 (0.00061)
	G (SW)	Alkalinity		<0.0020	Never	24	24 (3.0)
		Nitrate		<0.00033	Never	24	24 (0.49)
		Arsenic		<0.00000023	Never	24	24 (0.00023)
		Chloride		<0.012	Never	24	24 (18)
		Ammonia		<0.00067	Never	24	24 (0.99)
		Iron		<0.00007	Never	24	24 (0.10)
Scenario 2 (Horizontal Flow in Till)[5]	A (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	B (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	C (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
	D (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)

TABLE 4-3 (cont'd)

Scenario[1]	Site Sensitive Receptor (Media)[2]	Parameter	Concentration, mg/L At Time of Years[6]		Time to Reach Applicable Criteria, Years	Time to Reach Steady-State, Years	Time in Years to Reach Maximum and (Concentration, mg/L)
			3	6			
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
	E (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	F (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	G (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
Scenario 2 (Horizontal Flow in Bedrock)[5]	A (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	B (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	C (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
	D (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
	E (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	F (GW)	Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Iron		<0.000070	>30	>30	30 (<0.000070)
		Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)

TABLE 4-3 (cont'd)

Scenario[1]	Site Sensitive Receptor (Media)[2]	Parameter	Concentration, mg/L At Time of Years[6]		Time to Reach Applicable Criteria, Years	Time to Reach Steady-State, Years	Time in Years to Reach Maximum and (Concentration, mg/L)
			3	6			
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
	G (SW)	Alkalinity		<0.0020	>30	>30	30 (<0.0020)
		Nitrate		<0.00033	>30	>30	30 (<0.00033)
		Arsenic		<0.0000002	>30	>30	30 (<0.0000002)
		Chloride		<0.012	>30	>30	30 (<0.012)
		Ammonia		<0.00067	>30	>30	30 (<0.00067)
		Iron		<0.00007	>30	>30	30 (<0.00007)
Scenario 3 (Horizontal Flow in Till)	A (GW)	Nitrate	0.00033		Never	Never	7 (0.0033)
		Iron	0.000070		Never	Never	7 (0.000696)
		Alkalinity	0.0020		Never	Never	7 (0.0201)
		Chloride	0.012		Never	Never	7 (0.11788)
		Ammonia	0.00066		Never	Never	7 (0.0066)
		Arsenic	0.00000015		Never	Never	7 (0.0000015)
	B (GW)	Nitrate	0.00036		Never	Never	7 (0.0033)
		Iron	0.000077		Never	Never	7 (0.00070)
		Alkalinity	0.0022		Never	Never	7 (0.020)
		Chloride	0.013		Never	Never	7 (0.12)
		Ammonia	0.00073		Never	Never	7 (0.0066)
		Arsenic	0.00000017		Never	Never	7 (0.0000015)
	C (SW)	Alkalinity	<0.0020		Never	Never	12 (0.0020)
		Nitrate	<0.00033		Never	Never	12 (0.00033)
		Arsenic	<0.00000015		Never	Never	12 (0.00000015)
		Chloride	<0.012		Never	Never	12 (0.012)
		Ammonia	<0.00067		Never	Never	12 (0.00066)
		Iron	<0.000070		Never	Never	12 (0.000070)
	D (SW)	Alkalinity	<0.0020		Never	Never	11 (0.056)
		Nitrate	<0.00033		Never	Never	11 (0.0092)
		Arsenic	<0.00000015		Never	Never	11 (0.0000042)
		Chloride	<0.012		Never	Never	11 (0.33)
		Ammonia	<0.00067		Never	Never	11 (0.019)
		Iron	<0.000070		Never	Never	11 (0.0019)
	E (GW)	Nitrate	0.00066		Never	Never	6 (0.0029)
		Iron	0.00014		Never	Never	6 (0.00063)
		Alkalinity	0.0040		Never	Never	6 (0.018)
		Chloride	0.024		Never	Never	6 (0.11)
		Ammonia	0.0013		Never	Never	66 (0.0060)
		Arsenic	0.00000030		Never	Never	6 (0.0000014)
	F (GW)	Nitrate	<0.00000015		Never	Never	6 (<0.00098)
		Iron	<0.000070		Never	Never	6 (0.00021)
		Alkalinity	<0.0020		Never	Never	6 (0.0060)
		Chloride	<0.012		Never	Never	6 (0.035)
		Ammonia	<0.00067		Never	Never	6 (0.0020)
		Arsenic	<0.00033		Never	Never	6 (<0.0000005)
	G (SW)	Alkalinity	<0.0020		Never	Never	11 (0.0012)
		Nitrate	<0.00033		Never	Never	11 (0.0020)
		Arsenic	<0.00000015		Never	Never	11 (0.00000091)
		Chloride	<0.012		Never	Never	11 (0.071)
		Ammonia	<0.00067		Never	Never	11 (0.0040)
		Iron	<0.000070		Never	Never	11 (0.000042)
Scenario 3 (Horizontal Flow in Bedrock)	A (GW)	Nitrate	<0.00033		Never	Never	0.3 (0.023)
		Iron	<0.000070		Never	Never	0.03 (0.0049)
		Alkalinity	<0.0020		Never	Never	0.3 (0.14)
		Chloride	<0.012		Never	Never	0.3 (0.83)
		Ammonia	<0.00067		Never	Never	0.3 (0.046)

TABLE 4-3 (cont'd)

Scenario[1]	Site Sensitive Receptor (Media)[2]	Parameter	Concentration, mg/L At Time of Years[6]		Time to Reach Applicable Criteria, Years	Time to Reach Steady-State, Years	Time in Years to Reach Maximum and (Concentration, mg/L)
			3	6			
		Arsenic	<0.00000015		Never	Never	0.3 (000011)
	B (GW)	Nitrate	<0.00033		Never	Never	0.14 (0.066)
		Iron	<0.000070		Never	Never	0.14 (0.014)
		Alkalinity	<0.0020		Never	Never	0.14 (0.40)
		Chloride	<0.012		Never	Never	0.14 (2.4)
		Ammonia	<0.00067		Never	Never	0.14 (0.13)
		Arsenic	<0.00000015		Never	Never	0.14 (0.000030)
	C (SW)	Alkalinity	<0.0020		Never	Never	0.5 (0.012)
		Nitrate	<0.00033		Never	Never	0.5 (0.0020)
		Arsenic	<0.00000015		Never	Never	0.5 (0.00000091)
		Chloride	<0.012		Never	Never	0.5 (0.071)
		Ammonia	<0.00067		Never	Never	0.5 (0.0040)
		Iron	<0.000070		Never	Never	0.5 (0.00042)
	D (SW)	Alkalinity	<0.0020		Never	Never	0.45 (0.030)
		Nitrate	<0.00033		Never	Never	0.45 (0.0049)
		Arsenic	<0.00000015		Never	Never	0.45 (0.0000023)
		Chloride	<0.012		Never	Never	0.45 (0.18)
		Ammonia	<0.00067		Never	Never	0.45 (0.0099)
		Iron	<0.000070		Never	Never	0.45 (0.0010)
	E (GW)	Nitrate	<0.00033		Never	Never	0.5 (0.0029)
		Iron	<0.000070		Never	Never	0.5 (0.00063)
		Alkalinity	<0.0020		Never	Never	0.5 (0.018)
		Chloride	<0.012		Never	Never	0.5 (0.11)
		Ammonia	<0.00067		Never	Never	0.5 (0.0060)
		Arsenic	<0.00000015		Never	Never	0.5 (0.0000014)
	F (GW)	Nitrate	<0.00033		Never	Never	0.45 (0.033)
		Iron	<0.000070		Never	Never	0.45 (0.0070)
		Alkalinity	<0.0020		Never	Never	0.45 (0.20)
		Chloride	<0.012		Never	Never	0.45 (1.18)
		Ammonia	<0.00067		Never	Never	0.45 (0.066)
		Arsenic	<0.00000015		Never	Never	0.45 (0.000015)
	G (SW)	Alkalinity	<0.0020		Never	Never	0.35 (0.16)
		Nitrate	<0.00033		Never	Never	0.35 (0.026)
		Arsenic	<0.00000015		Never	Never	0.35 (0.000012)
		Chloride	<0.012		Never	Never	0.35 (0.94)
		Ammonia	<0.00067		Never	Never	0.35 (0.053)
		Iron	<0.000070		Never	Never	0.35 (0.0056)

VOLUME IV

APPENDIX I TABLE 3-2
SURFACE WATER, PORE-WATER, LEACHATE, UNDERDRAIN, AND LEAK DETECTION MONITORING LOCATIONS

TABLE 3-2

SURFACE WATER, PORE WATER, LEACHATE, UNDERDRAIN, AND LEAK DETECTION MONITORING LOCATIONS

Location Designation	Water Body Description	Collectio n Method	Position Relative To Landfill	First Recorded	Current Status
SW-4	Surface water feature which drains to unnamed tributary to Judkins Brook East	Grab	Downgradient	NA	To Be Established
SW-5	Surface water feature which drains to unnamed tributary of Pushaw Stream West	Grab	Downgradient	NA	To Be Established
LF-UD-12+13	Cell 12 +13 Underdrain	Grab	Underdrain discharge on Eastern side of Cell 13	NA	To Be Established
LF-LD-11	Cell 11 Leak Detection System	Grab	Eastern Perimeter Dike	NA	To Be Established
LF-LD-12	Cell 12 Leak Detection System	Grab	Eastern Perimeter Dike Leak Detection	NA	To Be Established
LF-LD-13	Cell 13 Leak Detection System	Grab	Eastern Perimeter Dike Leak Detection	NA	To Be Established
LF-LD-14	Cell 14 Leak Detection System	Grab	Western Perimeter Dike Leak Detection	NA	To Be Established
LF-LD-15	Cell 15 Leak Detection System	Grab	Western Perimeter Dike Leak Detection	NA	To Be Established
LF-LD-16	Cell 16 Leak Detection System	Grab	Western Perimeter Dike Leak Detection	NA	To Be Established
PWS-4	Pore water sample collected at Surface water feature which drains to unnamed tributary to Judkins Brook East	Grab	Northeast of Landfill	NA	To Be Established
PWS-5	Pore water sample collected at Surface water feature which drains to unnamed tributary of Pushaw Stream West	Grab	Northwest of Landfill	NA	To Be Established

Acronyms:
PWS – Pore Water Sample Location
SW-5 – Surface Water Sample Location
LF-UD-12 + 13– Landfill Underdrain Sample Location
LF-LD-11 – Landfill Leak Detection System Sample Location
NA – Not analyzed

VOLUME IV

APPENDIX I TABLE 4-1
ANALYTICAL PROGRAM

TABLE 4-1

ANALYTICAL PROGRAM

Water Quality Parameter	Method	PQL[1] (mg/l)
TDS	STM 2540C	10
TSS	STM 2540D	4
Ammonia (NH3-N)	STM 4500 NH3 E	0.5
Arsenic (As)	SW846/6010B/3010A	0.005
Calcium (Ca)	SW846/6010B/3010A	0.3
Iron (Fe)	SW846/6010B/3010A	0.05
Magnesium (Mg)	SW846/6010B/3010A	0.3
Manganese (Mn)	SW846/6010B/3010A	0.05
Potassium (K)	SW846/6010B/3010A	0.3
Sodium (Na)	SW846/6010B/3010A	0.3
Total Organic Carbon (TOC)	SW846/9060A	2.0
Chloride (Cl^-)	SW846/E300/9056	1.0
Sulfate (SO_4)	SW846/E300/9056	2.0
Volatile Organic Compounds (VOCs)[3]	U.S.EPA 8260B	0.001 – 0.01
Sulfide	SW846/9030B	2.5
Total Kjeldahl Nitrogen (TKN)[4]	STM 4500 NH_3E	0.3
Total Phosphorous[5]	U.S.EPA 365.3	0.04
BOD[6]	STM 5210B	5
Copper (Cu)	SW846/6010B/3010A	0.003
Bromide	SW9056	0.1
Nitrate & Nitrite	EPA 353.2	0.05
Total Alkalinity	STM 2320B	1.5
Boron	EPA-200.8	.05
Methane	EPA 8015B(MOD RSK-175)	.02.
<u>Field Parameters</u>		
Groundwater Elevation	Field Measurement	NA
Specific Conductance	Field Measurement	NA
Dissolved Oxygen	Field Measurement	NA
pH	Field Measurement	NA
Temperature	Field Measurement	NA
Turbidity	Field Measurement (APHA 2130)	NA
Eh	Field Measurement	NA
Monitoring Well Pumping Rate	Field Measurement	NA
Surface Water Flow Rate	Field Measurement	NA
Field Observations	Field Observations	NA

<u>Notes</u>:
1. Practical Quantitation Limits (PQLs) have been defined by U.S.EPA as up to 10 times the method or instrument detection limit and therefore may vary between laboratories.
2. NA = Not Applicable.
3. VOCs are the 47 organic constituents listed in Appendix I of 40 CFR Part 258. PQLs for VOCs are reported as μg/L. After two round of Characterization monitoring these compounds will only be sampled in the landfill leachate on a routine basis.
4. Monitoring wells and leachate only.
5. Surface waters and underdrain only.
6. Surface waters only.

<u>Method Reference</u>: The analytical methods selected are presented in <u>Test Methods for Evaluating Solid Waste</u>, OSWER, SW-846, Third Edition, as revised; Methods <u>for Chemical Analysis of Water and Wastes</u>, EMSL, EPA-600/4-79-020, revised March 1983; and <u>Standard Methods for the Examination of Water and Wastewater</u>, APHA, 19th Edition, 1995. Equivalent and appropriate analytical methods may be substituted with Juniper Ridge Landfill approval, e.g. manual for automated and vice versa.

EXHIBIT C

BGS AND NEWSME'S RESPONSE TO DEP'S JANUARY 20, 2016 TECHNICAL MEMORANDUM

Below BGS and NEWSME set forth each of Staff's comments in the January 20 letter and follow each comment with our response.

I. VOLUME I - MAINE SOLID WASTE MANAGEMENT RULES[1]

A. Section 9.0, DEP Reg. Chapter 400.11 - Financial Assurance.

For completeness, the closure and post-closure care cost estimates should be included in the Application. This section specifies that the cost estimates are included with the facility Annual Report. Additionally, due to the degree of uncertainty involved in both closure and post-closure care cost estimating, we recommend that a general cost contingency be included. This is typically 10 or 15 percent of the total costs.

> Response: The closure and post-closure cost estimates included in the application (Volume I Table 3-1) were based on 2015 capital closure and post closure opinions of costs prepared by Sevee and Maher Engineers Inc. (SME) for the Juniper Ridge Landfill, (JRL), and were included in the facility's 2014 Annual Report submitted to the DEP on April 24, 2015. A copy of this opinion of cost is included in Attachment SME-1. In preparing these costs SME does not include a separate line item contingency for several reasons. First, actual construction cost information is available for the site. These costs are used in developing capital closure costs. Second, the post closure costs reflect actual costs for items such as leachate treatment and disposal. Third, NEWSME has extensive experience with operating and closing landfills and post-closure monitoring and maintenance of landfills. Fourth, NEWSME accrues costs for closure and post closure care on a yearly basis, in addition to maintaining the surety bonds. These costs are audited by an independent auditing firm. Fifth, the 30 year post-closure costs have contingencies included in the overall cost since most of the individual line items are assumed to be constant over the entire closure period when in reality they will likely decrease over the post-closure period. Sixth, the closure and post-closure costs are consistent with actual closure and post-closure costs SME experience in the State of Maine. Finally as discussed during the January 29, 2016 meeting, the closure and post-closure costs included in the expansion application have a de-facto contingency built into the costs because they include landfill gas infrastructure costs. The landfill gas infrastructure is installed at JRL as part of the facility's ongoing operations, and thus will already have been installed by the time of closure and post-closure.

[1] *Juniper Ridge Landfill Expansion Application - Volume I - Maine Solid Waste Management Rules,* Sevee & Maher Engineers, Inc., July 2015.

<u>B. Section 11.0, DEP Reg. Chapter 400.13 – Variances</u>.
An alternative design assessment needs to be completed in general conformance with *Landfill Siting, Design and Operation*, 06-096 CMR 401(2)(E) (last amended April 12, 2015) in order to demonstrate that the barrier soil can be placed in a compacted lift thickness of 12 inches in lieu of the required maximum allowable 9-inch lift thickness. A variance request is not necessary as long as the technical equivalency of the proposed thickness can be successfully demonstrated. The completion of a test pad is proposed during each cell's construction in order to demonstrate that a homogeneous barrier layer meeting the design standards can be produced using the specified compaction techniques.

> <u>Response</u>: In preparing the application we chose to request a variance as allowed by Chapter 400 (13)(A) of the Rules rather than preparing an alternate design assessment per Chapter 401(2)(E). As described in the application, Volume I Section 11 the use of a 12-inch compacted lift is supported by past and current site practices, and the inclusion of a test pad program in the project specifications for both the compacted clay and till borrow. Nevertheless, as requested at the January 29, 2016 meeting included in Attachment SME-1 is an alternate design assessment addressing the items identified in Chapter 401(2)(E) to support using 12-inch soil compacted lift thicknesses for this project. If this demonstration is acceptable to the Department, then the variance request will not be required.

<u>C. Appendix J - Stormwater Management Plan</u>

<u>The time of concentration table on Sheet D-101</u> has many flow lengths that do not match the HydroCAD calculation lengths. This needs to be clarified.

> <u>Response</u>: The difference between the subcatchments flow lengths included on sheet D-101, and those used in the Hydro CAD modeling is a result of rounding the flow lengths in the table on sheet D-101. The difference in lengths is generally less than five feet. The one exception is the length of D-4 which is shown correctly on D-101 at 679 feet versus 824 feet used in the Hydro CAD model. These differences do not change the post development flows at any of the analysis points by more than 0.1cfs.

<u>On Sheet C-306, Outlet Control Structure table</u>, the orifice Inv. El. "E" for DP-10 shows 178.3 feet while the HydroCAD calculations show this as 178.0 feet. This should be clarified.

> <u>Response</u>: The HydroCAD elevation is correct. Drawing C-306 has been corrected (See Attachment SME-2).

<u>D. Appendix L - Leachate Disposal Contracts</u>
Section 1 of the Industrial Wastewater Discharge Permit between NEWSME and the City of Brewer Water Pollution Control Authority (Brewer) specifies that Brewer is the secondary discharge location and that authorization is required prior to discharge. It is our understanding that Brewer will now be the primary discharge location. The

Discharge Permit and Sections 2.4.5 and 3.3 of the Design Report (Volume III) will need to be updated accordingly.

> Response: Your understanding is not correct. The wastewater treatment facility at the Old Town mill continues to operate as the primary disposal location with Brewer being the secondary discharge location.

II. VOLUME III - DESIGN REPORT[2]

A. Section 2.1.1 Liner System

This Section specifies that the secondary liner will be augmented "in areas where the soil depth between the bedrock and landfill base grades is less than 10 feet…." We assume that this is a typographical error and the sentence should read 5 feet in lieu of 10 feet. This section and Section 2.2 should be updated as necessary.

> Response: Ten feet is correct and was the design criteria used to define the areas where the secondary liner will be augmented as described in section 2.1.1 of volume III of the application. The 10-foot soil depth criteria is based on the time of travel analysis completed to comply with the performance standard in Chapter 401(2)(C)(2) of the Rules as described in section 7.0 of volume II of the application.

B. Section 2.3 Base Preparation Below Liner Systems

The acceptability of placing the barrier soil in a 12-inch lift will need to be determined based on the results of each cell's test pad construction.

> Response: Comment noted.

C. Section 3.1 Geotechnical Evaluation.

This section notes that calculated tensile strains are "less than the maximum allowable strains recommended by geosynthetic manufacturers". Potential adverse impacts of calculated maximum tensile strains on the soil components of liner and cover systems should also be addressed.

> Response: Volume III, Appendix F-9, page 4 shows the calculated strains in the liner and cover systems that are expected due to settlement. Allowable tensile strains are shown as 5% for the liner system, which includes soil and geosynthetics. The maximum tensile strain in the cover and liner systems was calculated at 0.003% and 0.023%, respectively. These strains correspond to approximately 0.012 and 0.072 inches of total movement in the cover and liner systems, respectively. These very low levels of tensile strain and total movement are expected to have no measurable effect on the performance or physical properties of the soils in the liner or cover systems.

---

[2] *Juniper Ridge Landfill Expansion Application - Volume III - Design Report,* Sevee & Maher Engineers, Inc., July 2015.

<u>D. Table 3-4, Leachate Transport System</u>
The Design Selection column lists the existing leachate pond as available leachate storage capacity. The pond is a former leachate pond that is now part of the stormwater management system and is not available for leachate storage.

> <u>Response</u>: Comment noted. The expansion design has been completed without relying on the use of the former leachate pond for onsite leachate storage. In an unforeseen event, which would require temporary leachate storage in the pond, NEWSME would review the use of the pond with DEP prior to using it.

<u>E. Table 3-5, Gas Management Systems</u>
For clarity, the proposed gas header piping should be noted as 24-inch not 30-inch HDPE.

> <u>Response</u>: We agree. The updated table is include Attachment SME-3

<u>F. Table 3-6, Cover Systems</u>
The Table lists 20-mil geomembrane as an option for intermediate cover. Section 5.2 of the Operations Manual[3] specifies a 40-mil material while Section 7.8.2 notes a minimum 30-mil material when geomembrane is used as intermediate cover. A clarification should be made.

> <u>Response</u>: The reference to the 20 mil geomembrane option is specified as a "minimum" value. Typically the site utilizes a 40 mil geomembrane.

<u>G. Table 3-7, Potential Failure Modes and Significance of Failures in Engineered Systems</u>
The Table generally does an adequate job identifying Potential Modes of Failure but not Failure Significance. The Failure Significance column mostly addresses how the significance of failure is limited/ minimized through the design instead of what the significance of failure would be if it were to occur. The Table should be revised accordingly.

> <u>Response</u>: We've augmented this table to state the specific impact of the engineered system failures (see Attachment SME-3). We've retained the discussion of how significance failures are minimized through the redundancy of the design because it is an important design concept for the expansion.

<u>H. Section 3.5.1 Cell Development</u>
This Section notes that final closure "will likely occur over a several-year period" following filling in Cell 16. The closure sequencing should be scheduled such that final cover installation will be completed within one year of final waste acceptance.

[3] *Juniper Ridge Landfill - Operations Manual*, August 2005, last updated April 2015.

Response: This section of the application identified that "At least one year before the final phase cover placement begins, an application for final closure of the landfill containing the information required in Section 401.5.D of the Rules will be submitted to the DEP." Upon approval of that application the final cover will be constructed." This section of the application should have stated that this application will be submitted at least one year before *final waste placement* instead of final cover placement. As part of the closure application, it's anticipated that an alternative closure schedule will be requested to apply final cover over a several year period based on the amount of area requiring final cover at that time and limitations on the ability to construct the final cover for this area during a single construction season. This alternative schedule will not be undertaken without DEP's approval.

I. Figure 3-7 in Section 3.5.1 depicts stormwater flowing from intermediate cover onto final cover. A detail of how this transition will be accomplished should be developed and included with the Typical Operational Development Details in Appendix E of the Operations Manual.

Response: Comment noted. The stormwater will be directed at the edge of the cell in an operational ditch constructed directly adjacent to the final cover. A detail of this ditch has been added to Figure E-3 of the Operations Manual (Volume IV of the Application) as included in Attachment SME-4.

J. Appendix A - Construction Specifications[4]

1. Section 02200 - Earthwork

a. Page 02200-7
Part 2.01 D. 1.b). The clay layer should achieve an in-place, not remolded, hydraulic conductivity less than or equal to 1 x 10^{-7} cm/sec. This paragraph and Part 3.11 B. 1.b) of this section should be revised accordingly.

Response: Part 2.01 D.1.b of the specification is describing the properties of the material used for landfill construction including the hydraulic conductivity of the clay borrow, therefore remolded versus inplace hydraulic conductivity is the correct referenced property. We agree that the reference to the hydraulic conductivity of the clay found in Part 3.11.B.1.b should reference inplace hydraulic conductivity and this change has been made to this section and the updated specification is included in Attachment SME-5.

b. Page 02200-13
i. Part 3.09 C. Clay test pads for liner systems should encompass the transition from base liner to perimeter berm. Also, the clay test pads for the secondary liner

[4]*Bid Documents and Technical Specifications - Landfill Expansion - Juniper Ridge Landfill - Old Town, Maine,* Sevee & Maher Engineers, Inc., July 2015.

systems should encompass the transition from standard liner (one foot of clay) to augmented liner (two feet of clay) where applicable.

Response: This section of the specification has been updated to include these areas. The updated specification is included in Attachment SME-5.

ii. Part 3.09 C. Clay test pads for cover systems should encompass the transition from top slope to sideslope.

Response: This section of the specification has been updated to include this area. The updated specification is included in Attachment SME-5.

iii. Part 3.09 C. Hydraulic conductivity samples for clay test pads should also be taken across the interface between the two lifts of the augmented secondary liner and the two lifts of the final cover systems.

Response: This section of the specification has been updated to include this requirement. The updated specification is included in Attachment SME-5.

b. **Page 02200-17**
Part 3.14 B. The common borrow moisture content should be tested in general accordance with ASTM D 6938. This standard replaced ASTM D 3017.

Response: This section of the specification has been updated to include this requirement. The updated specification is included in Attachment SME-5.

2. Section 02272 - Geotextiles and Drainage Geocomposite

a. **Page 02272-5**
i. Part 2.01 A. 5. This section should reference the Mirafi® 600X woven geotextile that is proposed to be used within the plunge pool associated with the perimeter berm downspout. This detail is illustrated on Sheet C-306 of the Cell 11 Drawings.

Response: We have changed the fabric used in the plunge pools to Mirafi® FW700 or equal in this application which is reflected in the specification. The updated specification is included in Attachment SME-5.

ii. Part 2.01 B. Minimum property values with corresponding test methods should be established for the 10 oz/yd² non-woven geotextile that is proposed to be utilized within the gas header pipe trenches.

Response: Properties of the 10 oz/yd² fabric has been added to the specification. The updated specification is included in Attachment SME-5.

iii. Part 2.01 B. 5. a) Reference to ASTM D 3786, "Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength

Tester Method" should be removed from this section. ASTM Committee D35 on Geosynthetics does not currently recognize D 3786 as being applicable to geotextiles. This section and Parts 1.05 C. 10. d. and 1.06 1. should be updated accordingly.

Response: The reference to ASTM D 3786 has been removed from the specification. The updated specification is included in Attachment SME-5.

b. **Page 02272-6**
Part 2.01 C. 6. ASTM D 7179, "Standard Test Method for Determining Geonet Breaking Force" should be specified in lieu of D 5034. The former method is the most appropriate method for geonet testing.

Response: This section of the specification has been changed to reference ASTM D 7179. The updated specification is included in Attachment SME-5.

3. Section 02275 - Geosynthetic Clay Liner

a. **Page 02275-5**
Parts 3.02 B. 1. & 6. Installation provisions for the GCL in contact with the 60-mil geomembrane within the proposed secondary liner systems will need to be established. This section only references the GCL in contact with the 80-mil geomembrane within the proposed primary liner system.

Response: Since the installation criteria for the GCL used in the primary and augmented secondary liner system are the same the reference to only the 80 mil geomembrane has been removed to make this requirement generic to both the primary and secondary geomembranes. The updated specification is included in Attachment SME-5.

4. Section 02771 - Geomembrane Liner High Density Polyethylene (HDPE)

a. **Page 02771-2**
Part 1.05 C. Geomembrane asperity testing should be conducted in general conformance with ASTM D 7466 in lieu of the specified GM 12. GM 12 has been discontinued by the Geosynthetic Institute. Part 1.06 of this Section does note the correct test method.

Response: This change has been made to the specification and the updated specification is included in Attachment SME-5.

5. Section 02772 - Leak Location Survey

a. **This section should note that the leak location survey will be conducted in general conformance with ASTM D 7007, "Standard Practices for Electrical Methods for Locating Leaks in Geomembranes Covered with Water or Earth Materials". This**

test method is referenced in Technical Specification Section 02771 and Section 3.1 of the Quality Assurance/Quality Control Plan.

Response: This change has been made to the specification and the updated specification and Quality Assurance/Quality Control Plan is included in Attachment SME-5.

b. **We assume that one leak location survey will be conducted upon completion of the installation of both the secondary and primary liner systems including the primary leachate collection system. A clarification should be made as necessary.**

Response: The leak location survey will be done on the primary liner because this liner contains the landfill leachate and will be in contact with the leachate. The secondary liner will still be subject to the same high level of conformance testing, quality control oversight and testing during installation as the primary liner in accordance with the requirements of project specifications and QA/QC manual.

6. Section 02780 - Interfacial Friction Angle Conformance Testing

a. **This section will need to be clarified to state that an interface shear strength test will be conducted on the drainage geocomposite to 80-mil geomembrane and 80-mil geomembrane to GCL interfaces within the proposed primary liner system and the drainage geocomposite to 60-mil geomembrane and 60-mil geomembrane to GCL interfaces within the proposed secondary liner systems. Also, the 60-mil geomembrane to compacted clay will need to be tested for the non-augmented secondary liner.**

Response: The specification has been updated to clarify that the testing will be done for both the primary and secondary liners. The requirement to test the textured geomembrane against the clay has been added. The updated specification is included in Attachment SME-5.

b. **It should be specified that the geomembranes will be tested against the non-woven side of the GCL and that the compacted clay will be tested against the woven side of the GCL.**

Response: The specification has been updated to indicate that these interfaces will be tested as described. The updated specification is included in Attachment SME-5.

c. **Part 3.03 allows re-testing of failed interface tests. It should require the successful completion of a minimum of two re-tests for each failure.**

Response: The testing frequency for any retesting will be determined by the CQA project manager based on the results of the initial tests.

<u>7. Section 15100 - HDPE Pipe and Fittings</u>

a. **This section should specify pipe size and corresponding SDR rating in accordance with the pipe strength design calculations presented in Appendix D-1. Alternately, the SDR rating for all piping could be specified within pertinent details of the drawings.**

<u>Response</u>: The SDR ratings are included on the drawings prepared for construction, such as the Cell 11 drawings, as identified in Section B.11 of this specification. In addition as stipulated in section B 8 of the specification, "HDPE fittings, molded or fabricated, shall have the same pressure rating and strength as the pipe to which joining is intended." Several of the Cell 11 construction drawings have been updated and are included in Attachment SME-2.

b. **Part 2.11 A. inadvertently specifies that pressure transducers will be installed within Cell 6 in lieu of Cells 11 through 16. A clarification should be made.**

<u>Response</u>: We understand this reference is to Section 15110. The change has been made to Part 2.11 A. of Section 15110. The updated specification is included in Attachment SME-5.

<u>K. Appendix B - Construction Quality Assurance Manual</u>

<u>1. Section 5.5.1 Seam Layout</u>
This section should be consistent with Technical Specification Section 02771-9 Part 3.05 A. which notes that "no horizontal seams shall be allowed on the sideslopes of the cell."

<u>Response</u>: This change has been made to this section of the QA/QC manual. A copy of this section of the manual is included in Attachment SME-5.

<u>2. Section 5.10.1 Preparation</u>
This section specifies the installation of electrodes "if required" under the GCL prior to deployment. We assume that electrode installation under the GCL is necessary in order to appropriately perform the leak location survey. A clarification should be made.

<u>Response</u>: Since the main heading of this section of the manual is titled Leak Location Survey, the reference to the electrode installation is related to the leak location survey.

EXHIBIT C

L. Appendix D-3, Geocomposite Drainage Net Design

1. Supporting justification should be provided for the selected reduction factors (RFs) used to determine the drainage geocomposite allowable flow rate. As an example, for the intermediate condition, SME selected a RF_{IN} of 1.0 for intrusion, a RF_{CR} of 1.2 for creep, a RF_{BC} of 1.6 for biological clogging and a RF_{CC} of 1.6 for chemical clogging. Dr. Robert Koerner[5] recommends reduction factors ranging from 1.5 to 2.0 for intrusion, biological clogging and chemical clogging and 1.4 to 2.0 for creep for both primary and secondary leachate collection systems. The selected reduction factors used within the leak detection system design in Appendix D-5 of the Design Report should also be justified. Technical Specification Section 02272-6 may need to be updated if the reduction factors are modified.

> Response: SME selected reduction factors (RF) used for the expansion consistent with those used for the recent cells constructed at JRL. For the intrusion RF the specified conformance testing conditions of normal load and boundary conditions impart intrusive conditions in the geocomposite during testing, therefore the RF for intrusion is set at 1.0. We used creep reduction factor of 1.2 based on Stepped Isothermal Method (SIM) testing by TRI on Skaps geonet that concluded under normal loads of 15,000 psf a RF as low as 1.1 is justified. These reduction factors have been accepted by the DEP since the first phase of closure at the Pine Tree Landfill, which occurred in 2008. Justification for biological clogging and chemical clogging RFs is found in GRI Standard GC8 – Determination of Allowable Flow Rate of a Drainage Geocomposite 2001, rev 2013, pg 9 by the Geosynthetic Institute.

2. The calculations associated with the Cell 11 Leachate Collection Design Intermediate Condition should note that the specified drainage geocomposite transmissivity is 3.17 x 10^{-4} m^2/sec rather than 3.17 x 10^{-3} m^2/sec.

> Response: We agree. This is a typographical error in the reference. The design value is 3.17 x 10^{-4} m^2/sec.

M. Appendix D-5, Leak Detection System

A RF_{CC} of 1.3 and RF_{BC} of 1.5 were utilized within the leak detection system design calculations while a RF_{CC} of 1.5 and RF_{BC} of 1.3 were used for the same design calculations presented in Appendix D-3. We assume that this was a typographical error. A clarification should be made.

> Response: We agree, this was a typographical error.

[5]Designing with Geosynthetics, 6th Edition, Vol. 2, Robert M. Koerner, 2012, page 873.

EXHIBIT C

N. Appendix E - Design Drawings

1. Sheet C-102, Site Base Grading Plan
Installation details should be provided for the groundwater level monitoring transducers that are proposed to be installed under Cells 12 and 13.

Response: Installation details for the groundwater level monitoring transducers vary based on the model and manufacturer of the transducer. The transducers will be installed in accordance with the manufacturer's recommendations in order not to void the manufacturer's warranty. The installation details will be part of the construction drawings for Cells 12 and 13.

2. Sheet C-104, Leak Detection Piping Plan

a. The location of the Cell 11 leak detection sample sump should be shown.

Response: Drawing C-104 has been revised to show the leak detection cleanout and leak detection sample sump as shown on the Cell 11 Construction Drawings in Volume III Appendix K. This drawing is included in Attachment SME-2

b. The locations and dimensions of the temporary cell division berms separating Cells 12 and 13, Cells 14 and 15, and Cells 15 and 16 should be shown.

Response: The division berm separating Cells 12 and 13 will be graded into the base grades similar to how the intermediate cell berm between Cells 11 and 12 is shown. The grades associated with this berm will be established during the final detail design of Cell 12. Between Cells 14 and 15, and Cells 15 and 16 a temporary division berm will be used similar to berms that have been previously used at the site. Conceptual details of the temporary cell berms between Cells 14 and 15 and Cells 15 and 16 have been added to Drawing C-301 included in Attachment SME-2.

3. Sheet C-105, Leachate Collection Piping Plan
Installation details and specifications for the leachate level transducers should be included.

Response: Similar to the groundwater level transducers, the leachate level transducers will be installed in accordance with the manufacturer's recommendations in order not to void the manufacturer's warranty. Typically the transducers have been laid within the leachate collection sand and the cables placed in conduits back to the pump stations. The information on the transducer installation will be handled during cell construction as part of the contractor submittal process.

<u>4. Sheet C-106, Gas Collection System Plan</u>
A note specifies connection of a new 12-inch header pipe along the west side of the landfill to an existing 12-inch conveyance pipe. The existing conveyance pipe is 24 inches.

Response: The new 12-inch gas header pipe will be connected to an existing 12-inch gas conveyance pipe that is connected to the 24-inch gas conveyance pipe.

<u>5. Sheet C-107, Final Site Drainage Plan</u>
Details of drainage relief at the toe of the final cover system should be included. This includes both the riprap downspouts and the drainage sand layer.

Response: Details of the cover terminations at the landfill toe and downspout are shown on Figures 1 & 2 in Attachment SME-4.

<u>6. Sheet C-108, Final Development Plan</u>

a. The following additional typical details should be developed and included:

i. Final cover system to liner system connections;

Response: Details of the final cover to liner system connections are shown on Figures 1 and 2 included in Attachment SME-4.

ii. Final cover penetration boots for the active landfill gas extraction system wells and wellheads and the leachate collection and leak detection cleanouts; and

Response: A details of the pipe penetration boots for the leachate and leak detection cleanouts is shown on Figure 3 included in Attachment SME-4. Details of the pipe boots used for the gas collection system are shown on Sheet 13 of 14 in Attachment SHA-3.

iii. Final cover system to pump station connections.

Response: A detail of the final cover to pump station connection is shown on Figure 4 included in Attachment SME-4.

<u>7. Sheet C-201, Transverse Cross Sections Sta 14+00 to Sta 24+00</u>
The cross section for station 16+00 should be provided. It appears that it was inadvertently omitted from the drawings.

Response: The cross section for station 16+00 was inadvertently omitted from the drawings and has been added to Drawing C-201. The updated cross section Drawings C-201 and C-202 are included in Attachment SME-2.

8. Sheet C-300, Sections and Details

a. The transition between the native till and imported soil (12-inch compacted clay) should be depicted on the Liner System with Augmented Secondary Liner Detail. It is unclear if the intent is to "box" cut into the till.

Response: The detail shows the components of the liner system in the areas of the augmented secondary liner. The imported soils will be placed on the till soil one foot below the base grades shown on Drawing C-102. This is described in Note 5 on Drawing C-102. The reference to a "box" cut is a typical construction technique used when the grade change is abrupt such as the location where the secondary liner system changes to the augmented secondary liner system.

b. The dimensions of the drainage stone envelope around the leak detection pipe as depicted on the Piping at Perimeter Berm Detail should be specified.

Response: The dimensions of the leak detection piping stone envelope have been added to the Leak Detection Pipe detail on Drawing C-300 which is included in Attachment SME-2.

c. The north/south extent of the 6-inch deep by 6-foot wide base grade undercut leak detection sump, as indicated on the Leachate Collection & Leak Detection Cleanouts Detail, should be specified.

Response: The perpendicular dimension is 3 feet. This detail has been updated on Drawing C-300 which is included in Attachment SME-2.

d. Liner Termination. We assume that the impervious borrow specified within the anchor trench will achieve the specification for "clay borrow". If so, the terminology should be consistent. If not, a specification for impervious borrow should be established.

Response: The impervious borrow will meet the grain size requirements of the clay borrow. A material specification has been added to specification Section 02200 which is included in Attachment SME-5.

9. Sheet C-302, Sections and Details

A description and details to describe how the temporary leachate collection sumps will be abandoned or removed and how connections to subsequent cells will be made should be provided.

Response: The temporary leachate sumps will remain in place as subsequent cells are constructed. The 8-inch leachate collection header pipe along the outer perimeter berm of the cell will be connected into allowing the leachate flow to the subsequent cells. The connection will be detailed in the construction drawings for the subsequent cell, such as

Cell 12 for the Cell 11 connection. The stub end of the header pipe has been identified on Drawing C-302.

10. Sheet C-306, Sections and Details

a. **Culvert Schedule. Based on the information provided in Table 7-1 of the Stormwater Management Plan (Appendix J, Volume III), the inlet invert elevation for C-2BA should be 203.2 feet not 202.9 feet and the slope should be 0.008 % instead of 0.02 %.**

Response: The table on Drawing C-306 has been updated to reflect the correct culvert inverts and slopes. The updated drawing is included in Attachment SME-2.

b. **Catch Basins 4K & 4L. For clarity, the depth below the pipe stub invert should be specified as 2 feet.**

Response: The detail of Catch Basins 4K and 4L has been updated to show a 2-foot sump (see Attachment SME-2).

11. Sheet C-307, Sections and Details

a. **The Final Cover System Detail indicates that the 24-inch soil barrier layer is to be constructed over a surface prepared with "select waste". The term select waste should be defined. It should capable of acting as a gas transmission layer with a minimum hydraulic conductivity of $1x10^{-3}$ cm/sec.**

Response: The final cover system detail has been amended to identify that the minimum depth of the selected waste is 6 inches and the minimum hydraulic conductivity of this material will be$1x10^{-3}$ cm/sec and maximum particle size of four inches. This updated drawing is included in Attachment SME-2.

b. **Rodent guards should be specified on the drainage pipe discharges to the riprap down spouts depicted on the Riprap Downspout Detail.**

Response: A note indicating the requirement for rodent screens has been added to the detail. The updated drawing is included in Attachment SME-2.

c. **The Terrace Drainage Swale Detail notes a swale depth of 1.5 feet while the sizing information provided in Appendix K, Erosion and Sedimentation Control Plan of Volume III specifies a 2-foot depth. A clarification should be made.**

Response: The correct depth of the terrace ditches is 1.5 feet. Table 7-1 in the Erosion and Sediment Control Plan has been updated. This updated table is included in Attachment SME-3

d. **Depending on the results of the analysis recommended in Comment O. 1. of Appendix F below, it may be appropriate to add a drainage geocomposite between the geomembrane and topsoil at the Terrace Drainage Swale.**

Response: The drainage geocomposite will not be required for the drainage swale. This is also supported by a similar design used in the Phase VIII-C Stages 1 & 2 at the Pine Tree Landfill closure completed in 2010.

12. Sheet C-308, Sections and Details

a. **Anti-Seep Collar. This detail specifies a pipe length of 70 feet and saturated pipe length of 32 feet for DP-10 while the Detention Pond OCS Table (Table) in Appendix C-2 of Attachment J of Volume III notes a pipe length of 52 feet and saturated pipe length of 42 feet. Additionally, the detail specifies a saturated pipe length of 30 feet for DP-11 while the Table notes a saturated pipe length of 37 feet. Clarifications should be made as necessary.**

Response: The table on Drawing C-308 has been updated with the correct values and is included in Attachment SME-2.

b. **Level Spreader. The type of geotextile proposed to be placed underneath the stone within the level spreader should be clearly specified.**

Response: The note on the level spreader detail has been updated to specify the geotextile as Mirafi FW700, or approved equal. The updated drawing is included in Attachment SME-2.

O. Appendix F - Geotechnical Data

1. The geotechnical report should evaluate veneer stability of the final cover system. The evaluation should include sand saturation conditions due to failure of a 4-inch perforated Hancor drainage pipe at a terrace drainage swale.

Response: The veneer stability of the final cover system is an item that will be evaluated during the detailed design and preparation of the construction plans for the cover system. As the Department is aware SME has evaluated, designed and overseen the construction of cover systems in the State of Maine with cover slopes ranging from five percent to as steep as 2.5H:1V. These projects have been reviewed and approved by DEP, have been constructed and have performed satisfactorily.

The expansion includes final cover slopes up to 3H:1V, which is within the range of slopes that are considered coverable with commercially available geosynthetic materials and commonly available earthen materials using common construction techniques. The cover is proposed and will be designed with physical characteristics that comply with Chapter 401(5)(G) and during the detailed design of the cover, a stability assessment will be completed in accordance with Chapter 401 (1)(5)(I)(a). Although not specifically addressed by the Rules as part of the cover design, SME evaluates a saturated cover condition, which could occur with the complete failure of the 4-inch perforated drainage pipe. For this evaluation a factor of safety of at least 1.0 will be used as the design criteria consistent with past cover designs completed by SME and approved by the DEP.

3. Figures E-2 and E-3 in the Operations Manual indicate a slope of 1:1 over a length of about 4.5 feet at the waste toe. The geotechnical report should evaluate stability through these segments.

Response: A stability evaluation has been completed for the detail shown on Figure E-2 and E-3, using a modified version of slope stability Cross-Section A-A' that was included in the Application. The calculated factors of safety for the waste slopes including the 1:1 toe detail exceed the DEP required minimum values. See Attachment SME-6 for the results in the form of a summary table and SLOPE/W outputs.

3. Peak and large displacement liner strength envelopes have been taken from a 2005 geotechnical stability analysis. Substantial additional interface strength data from subsequent construction projects at the JRL facility is available and comparison of that data with the 2005 values should be made.

Response: Peak and large displacement liner strength envelopes from Cells 7, 8, and 9, the last three cells constructed at JRL, and the specified values contained in the application (Specified Shear Strength) are provided in Attachment SME-6 Test results are and will continue to be reviewed during the cell and cover construction projects at JRL as they relate to landfill and cover stability namely meeting the factors of safety specified in Chapter 401(2)(F)(1) of the Rules. Occasionally, construction test results are below the specified shear strength values for one or more of the strengths which define the strength envelopes. If this is the case a, geotechnical evaluation, using reported results, is completed and reviewed with the DEP before deciding if the material is acceptable for use on the project. The acceptance criteria in these cases are compliance with the previously mentioned factors of safety. From this process we have defined the 2005 shear strength envelops as the appropriate envelopes to use for the expansion.

4. The Sensitivity Assessment in Appendix F-7 should include an evaluation of the impact of leachate head build-up on the primary liner system if it were to occur.

Response: The slope stability evaluations presented in the application assume that the potentiometric surface of the leachate is at the top of the four-foot-thick liner system, (i.e., one foot of head in the leachate drainage sand). Stability cross-section A-A' is representative of the four cross-sections evaluated in the application and has been used to assess the sensitivity of landfill stability to leachate levels in the landfill. For this sensitivity analysis, the potentiometric surface of the leachate is placed at 10 feet above the top of the four-foot-thick liner system, (i.e., 11 feet of head in the leachate drainage sand). The results of this analysis are summarized, along with the results presented in the application for the post-closure period in Attachment SME-6. These results demonstrate required factors of safety are achieved for this condition. A similar analysis for the construction and operations phase would be expected to yield similar proportional decreases in the factor of safety as determined for the post closure period.

Also included in Attachment SME-6 is an analysis done using large displacement (LD) strengths. This analysis, consistent with the analysis contained in the application, was completed as part of a sensitivity analysis. The primary purpose of a sensitivity analysis is to assess the potential relative change in the calculated factors of safety from the base condition (i.e., that presented in the application) to any assumption applied in the analysis. This allows the identification of the assumption that may most affect the factor of safety. The minimum factors of safety values contained in the Rules do not apply to sensitivity analyses because the analysis is done assuming failed conditions; instead it is the relative change that is important. Since the relative change in the factor of safety is at most 5% for the 10-foot increase in head, it is not considered significant. The sensitivity analysis presented in the application was performed to assess the potential impact that LD strengths in the liner system may have, which was calculated to be approximately 26 percent (from 1.72 to 1.27). Therefore, the sensitivity analysis presented in the application represents a higher potential effect and that is why it was included.

5. The Settlement Evaluation Points on Figure 1 of Appendix F-8, Settlement Coefficients, should be labeled.

<u>Response</u>: This figure has been updated and included in Attachment SME-4.

<u>P. Appendix I - Landfill Gas Design Report</u>[6]

<u>1. Section 3.0 - Facility Description</u>

This section discusses the capacity of the site flares and a future landfill gas-to-energy facility in terms of a landfill gas (LFG) methane (CH_4) concentration of 50 percent by volume. Historic data suggests that the CH_4 concentration at JRL is on the order of 35 to 40 percent by volume. System capacities should also be compared to the likely lower concentrations of CH_4.

<u>Response</u>: Section 3.0 of the LFG System Expansion Design Report (LFG Design Report) provides a description of the existing gas collection and control system (GCCS) at JRL. The capacities of the existing equipment are related by the industry standard of normalizing the methane concentration of the gas to 50 percent by volume. The statement that the future LFGTE facility is anticipated to handle 2,170 scfm at 50 percent methane is consistent with the industry standard for having a basis for comparison. Section 4.0 of the LFG Design Report acknowledges that JRL LFG methane content is less than 50 percent, and states that the design basis for the conveyance infrastructure is based on LFG flow rate adjusted for the lower methane content.

[6] *LFG System Expansion Design Report - Juniper Ridge Landfill - Old Town, Maine,* Sanborn, Head & Associates, Inc., June 2015.

2. Section 4.0 - Landfill Gas Generation Estimates

This section discusses LFG generation estimates as predicted by the LandGEM[7] Model. The Model projection has been initiated from the first year of operation at JRL. A comparison of the projected generation rates with actual data from JRL to date should be made to calibrate the LandGEM Model input.

> Response: Figure 2 of the LFG Design Report was prepared based on a methane concentration of 40 percent. As requested, Figure 2 was revised to include annual flow data. The revised Figure 2 is provided as Attachment SH-1. Also, the LandGEM Model was calibrated based on site-specific data. As shown on the revised figure, the LandGEM Model results over predict the actual measured flow rates, demonstrating that the GCCS infrastructure design is conservative.

3. Section 5.2 - Gas Collection Trenches

It is noted that intermediate cover will be placed over the extraction trenches. Intermediate cover over the trenches would interfere with LFG extraction above the trenches and the reasons including it should be discussed.

> Response: The intermediate cover layer reference was removed from Section 5.2 of the LFG Design Report and Detail 2 on Sheet 13 of the Landfill Gas System Expansion Drawings (LFG Expansion Drawings). The revised Section 5.2 of the LFG Design Report is provided as Attachment SH-2, and the revised LFG Expansion Drawings are provided as Attachment SH-3.

4. Section 5.3 - Conveyance Pipe

Reference is made to "industry experience" when sizing smaller diameter pipe. A citation or citation(s) for "industry experience" should be included.

> Response: The reference to "industry experience" refers to Sanborn Head's own experience at JRL and other municipal solid waste landfills in Maine and throughout New England where LFG systems have been designed, installed, and operated. The 4-inch and 6-inch diameter lateral LFG conveyance pipes at these sites have performed adequately, and under conditions similar to those proposed for the expansion of the LFG system at JRL.

5. Appendix A - Calculations

a. **The LandGEM Model includes estimates for the methane generation rate of k = 0.1 year^{-1} and potential methane generation capacity of L_o = 110 m^3/Mg. As noted above, actual data from JRL should be used to calibrate the Model input.**

> Response: The k and Lo values used in the LandGem Model were developed based on site-specific information and considered only the degradable fraction of the waste mass.

[7] *Landfill Gas Emissions Model,* United States Environmental Protection Agency, Version 3.02, May 2005.

b. **The calculated LFG flow velocity in the new 24-inch header on the east side of the landfill is checked against criteria for concurrent LFG and condensate flow. Flow in the northern half of the header is countercurrent and that condition should be checked as well.**

Response: The Header Sizing Calculations were revised as requested, and are provided in Attachment SH-4.

c. **The calculated LFG flow velocity in two of the internal header pipes (5 and 6) exceeded identified criteria. The calculated exceedances are under a worst case scenario where LFG is being pulled from one side of the header only. Both headers are designed to be pulled from two directions, therefore velocity "is not expected" to exceed criteria under normal operations. Calculations under normal operations should be included to verify the expectation.**

Response: The Header Sizing Calculations were revised as requested, and are provided in Attachment SH-4.

6. Appendix B - Engineering Drawings[8]

a. **Sheet 2 of 14 - Landfill Gas Extraction System Plan**

i. An existing conditions plan should be prepared to depict the LFG infrastructure expected to exist at the time of the development of the first cell of the expansion. For clarity the horizontal collectors can be left off the plan.

Response: The LFG Expansion Drawings were revised to include a new sheet titled "Cells 1 through 10 Projected Development Plan." See Sheet 1 of 14 in Attachment SH-3. Sheet 1 depicts the projected condition of the GCCS prior to the expansion for Cells 11 through 16.

ii. Consideration should be given to providing a redundant header connection for extraction laterals that collect LFG from several, as an example more than three, extraction wells. Under this example, six relatively short sections of header pipe would be required and the need for future repairs may be mitigated.

Response: The lateral conveyance pipes shown on the LFG Expansion Drawings will be located just below the final cover system. Existing deeply-buried laterals will be replaced prior to installation of the final cover system. A slope of at least 5 percent was provided on the lateral pipes to accommodate waste settlement. For these reasons, redundancy is not required for the final conveyance pipe system.

[8] *Landfill Gas System Expansion Drawings - Juniper Ridge Landfill - Old Town, Maine,* Sanborn, Head & Associates, Inc., June 2015.

b. **Sheet 3 of 14 - Perimeter LFG Header Pipe Profile**

i. A note should be added to field verify the leachate force main locations prior to installing the LFG header pipe.

Response: The anticipated construction sequence for the perimeter berm includes installing the leachate force main and LFG header pipes concurrent with the berm construction. Critical pipe crossings will be identified on the construction-level drawings.

ii. Procedures for completing the LFG header pipe crossings under culverts should be developed.

Response: The anticipated construction sequence for the perimeter berm includes installing the LFG header pipes concurrent with the berm construction, which would include the stormwater culverts. Critical pipe crossings will be identified on the construction-level drawings.

c. **Sheet 4 of 14 – Cell 11 LFG Infrastructure Development Plan**

i. This plan depicts extraction wells labeled GW-24, 25, and 16 while the Cell 11 construction drawings[9] label the same wells GW-24R, 25R, and 16R. We assume that the R indicates that these are replacement wells and that the construction drawings depict the intended designations, however, a clarification should be provided. Abandonment procedures for the old wells should be developed if wells are to be replaced.

Response: GW-24R, 25R, and 16R are replacements for GW-24, 25, and 16, respectively. Sheet 4 of the LFG Expansion Drawings (Attachment SH-3) was revised to include a note about replacing or extending these wells. In addition, a new detail (Detail 6) depicting the decommissioning (abandonment) procedure for old wells was added to Sheet 13 of the LFG Expansion Drawings. Similar clarifications were made to the Cell 11 Landfill Gas System Expansion Drawings (Cell 11 LFG Drawings), which are provided as Attachment SH-5

ii. LFG collection headers and laterals for extraction wells on the north sideslope at this stage of development should be depicted on the plan.

Response: Sheet 4 of the LFG Expansion Drawings (Attachment SH-3) was revised to show the LFG collection headers and laterals on the north sideslope, as requested.

d. **Sheet 7 of 14 – Cell 14 LFG Infrastructure Development Plan**

[9] *Cell 11 Landfill Gas System Expansion Drawings - Juniper Ridge Landfill - Old Town, Maine,* Sanborn, Head & Associates, Inc., June 2015.

In general, it is not clear whether many of the extraction wells within the existing landfill footprint are to be extended or abandoned and replaced as the expansion cells fill over them. As an example GW-12, which currently exists within the Cell 3A footprint, will be 64 feet beneath the waste surface at the stage of development depicted on this plan. GW-12 is, however, shown as an existing well connected to a new collection lateral located near the waste surface.

Response: Existing LFG extraction wells may need to be extended or replaced as filling occurs in areas surrounding the wells. The decision to either extend or replace a well will be based on the length and depth of the existing well screen, performance of the well, etc. The LFG Expansion Drawings were revised to include a note about replacing or extending wells. In addition, a new detail (Detail 6) depicting the decommissioning (abandonment) procedure for old wells was added to Sheet 13 of the LFG Expansion Drawings (see Attachment SH-3).

e. Sheet 10 of 14 - Landfill Gas Extraction System Details

i. A polyvinyl chloride (PVC) well bore seal with a collapsible boot is proposed around the final cover system LFG pipe penetrations. While not indicated on the details, the well bore seal skirt should be sealed to the cover system geomembrane.

Response: The PVC Well Bore Seal and the HDPE final cover geomembrane are dissimilar materials, and therefore cannot be physically sealed (i.e., welded together). Rather, air intrusion will be limited because the PVC Well Bore Seal will be weighed down by the mass of the final cover soil placed over the PVC that is in direct contact with the final cover geomembrane.

ii. Detail 2, Typical Gas Extraction System Wellhead Detail, depicts the collapsible portion of the well bore seal partially below final cover system grade. The well bore seal should be installed such that the collapsible portion is located fully above grade.

Response: Detail 2 on Sheet 10 of the LFG Expansion was revised as requested. Please see Attachment SH-3.

iii. Detail 2, depicts the 90° elbow connection to the LFG header a minimum of 4 feet below the bottom of the final cover. This conflicts with the Cell 11 construction plans (Detail 3, Sheet 8 of 11) which specifies the elbow 2 feet below the final waste grade.

Response: Detail 2 on Sheet 10 of the LFG Expansion Drawings was revised and is consistent with Detail 3 on Sheet 8 of the Cell 11 LFG Drawings (see Attachments SH-3 and SH-5, respectively). The depth to the top of the pipe is a minimum 2 feet.

iv. Detail 4, Typical Cover System, depicts the final cover system barrier soil placed directly on refuse. The barrier soil should be placed on a LFG transmission sand layer or equivalent.

Response: As outlined in the response to Question 11, Sheet C-307, Sections and Details, the final cover system will include a minimum 6-inch-thick layer of select waste that has a minimum hydraulic conductivity of 1x10-3 cm/sec. Detail 4 on Sheet 10 of the LFG Expansion Drawings was revised to include the layer of select waste.

v. Gas extraction system boots associated with the geomembrane intermediate cover should also be detailed.

Response: Detail 3 on Sheet 13 of the LFG Expansion Drawings was revised as requested, see Attachment SH-3.

f. Sheet 12 of 14 - Landfill Gas Extraction System Details

i. The condensate knockout (KO-4) appears to be inappropriately designed for its location. There is no LFG flow from the downstream side of the structure so it will function only as a low point collector.

Response: This design of KO-4 was developed to provide JRL with the flexibility to extend the LFG header in the future.

ii. Note 9 states "Pump in condensate knockout shall be able to pump more than 1 gpm." An actual pump should be selected and specified.

Response: Note 5 on Sheet 12 of the LFG Expansion Drawings provides the requested information on the specified pump. Note 9 was removed from Sheet 12.

g. Sheet 13 of 14 - Landfill Gas Extraction System Details

i. Detail 2, Typical Gas Collection Trench Section, specifies "12-inch intermediate cover" placed directly over the trench. As noted earlier, intermediate cover over the trenches would interfere with LFG collection above them.

Response: As noted in an earlier response, the intermediate cover layer was removed from the Detail 2 on Sheet 13 of the LFG Expansion Drawings.

ii. Perforation size and pattern should be specified for the horizontal LFG collection pipe.

Response: Detail 2 on Sheet 13 of the LFG Expansion Drawings was revised to include a pipe perforation pattern.

h. <u>Sheet 14 of 14 - Landfill Gas Extraction System Details</u>

Note 4 indicates that the intent is for LFG extraction wells and wellheads to stick up three feet above the final waste fill grade. Details and procedures to extend them at the time of final closure should be established.

Response: The purpose of Note 4 is to provide installation information associated with the LFG extraction well schedule. Detail 2 on Sheet 10 of the LFG Expansion Drawings depicts the construction condition of the wellhead. As previously noted, additional information relative to specific well extensions/replacements will be included in construction-level drawings.

7. Appendix C - Technical Specifications

Section 02560 - Landfill Gas Extraction Wells

Part 1.3 B. requires removal of any soil placed to level drilling locations following well completion. Proper disposal requirements for any soil mixed with waste should be specified.

Response: Specification Section 02560 was revised as requested. Specifically, Paragraph 1.3B was revised to state that "if soil placed for leveling contacts refuse, it shall be disposed of in the active area of the landfill." The revised specification is provided as Attachment SH-6.

8. Appendix D - Construction Quality Assurance Plan[10]

Section 3.1 - Pre-Construction Meeting

The Department should be notified of the time and location of pre-construction meetings.

Response: Section 3.1 of the Construction Quality Assurance (CQA) Plan was revised as requested. Refer to Attachment SH-7.

9. Appendix E - Operation and Maintenance Manual (Manual)[11]

a. **<u>General</u>. The Manual is very generic and should be reviewed and updated to reflect the actual conditions at JRL, which are well known at this point. Some examples include, but are not limited to, the following:**

i. Section 2.1 notes that LFG is comprised of methane "(typically about 55 percent)" and carbon dioxide "(typically about 45 percent)". Methane

[10] *Construction Quality Assurance Plan - Landfill Gas Extraction System Expansion - Juniper Ridge Landfill - Old Town, Maine,* Sanborn, Head & Associates, Inc., June 2015.
[11] *Operation and Maintenance Manual - Landfill Gas Management System - Juniper Ridge Landfill - Old Town, Maine,* Sanborn, Head & Associates, Inc., June 2015.

concentrations at JRL are typically 30 to 40 percent and carbon dioxide concentrations are typically 20 to 35 percent.

Response: The revised Operations and Maintenance (O&M) Manual is provided as Attachment SH-8. Specific to Section 2.1, the narrative was revised to reflect actual conditions at JRL, including the LFG composition and temperature range.

ii. Section 2.1 describes the concentration of nitrogen as "lesser amounts". The concentration of nitrogen (balance gas) at JRL is quite high, typically 20 to 50 percent.

Response: Please see the previous response.

iii. Section 2.2.1 states that "Potentially lethal concentrations of hydrogen sulfide (H_2S) may be present at landfills". H_2S is highly elevated at JRL and the levels should be noted. Additional emphasis on the dangers associated with H_2S should be addressed in the Manual.

Response: Section 2.2.1 of the O&M Manual was revised as suggested.

iv. Section 5.3 states that "conveyance pipe is generally installed at a minimum slope of 5 percent within the landfill". Conveyance pipe within the JRL is required to be installed at a minimum slope of 7 percent.

Response: Both the LFG Expansion Drawings and Cell 11 LFG Drawings were revised to designate a minimum slope of 5 percent on solid LFG conveyance pipes located within the limit of waste containment at JRL. As such, the note requiring a minimum 7 percent slope was removed the Cell 11 LFG Drawings. Maintaining slope on LFG conveyance pipe helps to accommodate settlement and facilitate the flow of condensate. Section 5.3 of the O&M manual was revised to clarify this information.

v. Section 5.4.1 discusses what pipe and fittings "commonly consist of" or have "been successfully used in landfill gas applications". The section should be specific to the pipe and fittings actually used at JRL. Section 5.4.2 treats valves in the same manner.

Response: Sections 5.4.1 and 5.4.2 of the O&M Manual were revised to include additional information as requested.

vi. Section 7.1 covers data assessment and focuses on assessment during the initial start-up and operation of the landfill gas management system. It introduces such terms as "baseline data", "normal conditions", "established parameters for normal operating ranges", "acceptable ranges and conditions", and "target criteria". Since initial start-up and operation of the landfill gas management system happened about ten years ago the aforementioned terms can be, and should be, defined and included in the Manual.

Response: Section 7.1 of the O&M Manual was revised to reflect conditions specific to JRL, as requested.

vii. Section 7.3 refers to migration monitoring probes surrounding the landfill. Currently, there are no migration monitoring probes surrounding the landfill.

Response: Reference to the migration monitoring probes was deleted from Section 7.3 of the O&M Manual.

b. Section 2.1 - Landfill Gas Characteristics. LFG is described as flammable and explosive in the range of 5 to 15 percent in air. It should state that LFG is flammable and explosive when *methane* is present in the range of 5 to 15 percent in air.

Response: Section 2.1 of the O&M Manual was revised to reflect conditions specific to JRL, as requested.

c. Section 2.1 states that gas levels should be monitored "at any location where there is potential for landfill gas to be present and where personnel could be exposed". There are known locations at JRL that meet this description and they should be listed in the Manual. The same paragraph states that the percent hydrogen sulfide should be monitored. Hydrogen sulfide should be monitored in the unit of parts per million.

Response: The narrative in Section 2.1 of the O&M manual was revised as requested.

d. 3.0 - System Components and Monitoring Program

i. Section 3.1 - Introduction. It is stated that LFG is conveyed to a blower/flare station for treatment. The LFG is first conveyed to a sulfur treatment unit.

Response: The narrative in Section 3.1 of the O&M Manual was revised to include the sulfur removal system.

ii. Condensate structures and management systems located outside the landfill footprint should be addressed in this section.

Response: The narrative in Section 3.3 of the O&M Manual was revised as requested to address condensate management outside the landfill footprint.

iii. The Thiopac® and SulfaTreat® sulfur scrubbing units should be addressed in this section or reference made to where they are addressed.

Response: A new Section 3.5 was inserted into the O&M Manual specifically to discuss the basics of the sulfur removal systems. Specific O&M considerations for the sulfur scrubbing units are provided in separate documents.

iv. Section 3.4 - Wellhead Assemblies. It is stated (also in Section 3.7) that wellheads are monitored on a weekly basis. The wellheads are currently monitored less frequently.

Response: Section 3.4 of the O&M Manual was revised to state that wellheads are monitored on a monthly basis.

v. Section 3.4 should also address inspection and adjustment of the well bore seals that will be installed during phased final closure of the landfill.

Response: Section 3.4 of the O&M Manual was revised as requested.

vi. Extension of the temporary collection trench risers should be addressed in this section.

Response: Section 3.2 of the O&M Manual was revised to address extension of collection trench risers.

e. 4.0 - Operation and Adjustment

i. Section 4.3 - Indicator Parameters. This section should identify levels of oxygen and nitrogen that are excessive in LFG and actions to take if they are present.

Response: Section 4.3 of the O&M Manual was revised to include a requirement for maintaining the concentration of oxygen in LFG to five percent or less.

ii. Section 4.5 - Pressures. Reference is made to "normal ranges" of LFG constituent concentrations. Normal ranges of LFG constituents at JRL should be identified.

Response: Section 4.5 was revised with the requested concentration levels for oxygen and methane.

iii. Section 4.6 - Temperature. This section should address excessively high temperatures, identify what they are, and discuss actions to take if excessive temperatures are present.

Response: The narrative in Section 4.6 of the O&M Manual was revised to indicate that a wellhead temperature measurement of 150°F is the maximum allowable, which would require adjusting the wellhead opening to reduce LFG flow.

f. <u>**5.0 - Maintenance and Troubleshooting**</u>

i. <u>Section 5.3 - Conveyance Pipe</u>. This section discusses excavation into the landfill to address problems with blocked header pipe. Process and procedures to be followed if excavation into a section of the landfill with final cover in place is necessary, including cover system restoration, should be discussed.

<u>Response</u>: Section 5.3 of the O&M manual was revised as requested.

ii. <u>Section 5.4.10 - Condensate Handling Systems</u>. This section should address operation and maintenance requirements for the condensate handling system recently installed to collect and recirculate condensate from the sulfur treatment system building and Condensate Knockout KO-3.

<u>Response</u>: Section 5.4.10 of the O&M manual was revised to include inspection and maintenance for LFG treatment process condensate handling systems.

g. <u>Table 1 - Typical Landfill Gas Constituents</u>. The table should be revised to reflect the concentration ranges of landfill gas constituents typically measured at JRL.

<u>Response</u>: Table 1 of the O&M Manual was revised to include ranges of LFG constituent concentrations measured at JRL.

<u>**10. Appendix K – Cell 11 Design Drawings**</u>

a. <u>Sheet C-101, Existing Site Conditions Plan</u>. This sheet should be updated prior to construction to reflect existing conditions at that time. The current schedule calls for 2017 construction to allow use of Cell 11 in 2018.

<u>Response</u>: Comment noted. This drawing will be updated with existing site information prior to construction of Cell 11.

b. <u>Sheet C-102, Base Grading Plan</u>

i. The reference to the section for the intermediate cell berm is to Detail 2 on Sheet C-300. The correct reference is Detail 1 on Sheet C-300. This reference carries through to other sheets.

<u>**Response:**</u> The Intermediate Cell Berm detail on Drawing C-300 was mislabeled; it has been changed to 2. The updated Drawing C-300 is provided in Attachment SME-2

ii. The reference for Grass Ditch DP-10 is to DWG C-304. The correct reference is to DWG C-307.

Response: The reference has been changed to reference Drawing C-307. The updated drawing is provided in Attachment SME-2

iii. The reference for exterior layout grade points is to the 200 Series Table. The correct reference is to the 500 Series Table.

Response: That is correct, the reference has been updated. The updated drawing is provided in Attachment SME-2

c. Sheet C-104, Leachate Collection Piping Plan. The proposed leachate level transducer along with any pertinent installation details should be illustrated on this drawing. Sheet C-105 of the Design Drawings found in Appendix E of the Design Report illustrates the transducer location within Cell 11.

Response: The leachate level transducers will be installed in accordance with the manufacturer's recommendations in order not to void the manufacturer's warranty. The location of the transducer is shown on Drawing C-104.

d. Sheet C-200, Force Main and Landfill Gas Header Plan and Profile. The Landfill Gas Header Plan and Profile should be updated prior to Cell 11 construction. A Thiopaq® sulfur treatment system is now in use and a landfill gas-to-energy plant is planned to be constructed during 2016.

Response: The drawing has been updated to reflect this location of the condensate line. This updated drawing is included in Attachment SME-2

e. Sheet C-300, Sections and Details

i. The transition between the native till and imported soil (12-inch compacted clay) should be depicted on the Liner System with Augmented Secondary Liner Detail. It is unclear if the intent is to "box" cut into the till.

Response: The detail shows the components of the liner system in the areas of the augmented secondary liner. The imported soils will be placed on the till soil one foot below the base grades shown in Drawing C-102. This is described in Note 12 on Drawing C-102. The reference to a "box" cut is a typical construction technique used to transition the liner system in the area of the augmented liner system

ii. The dimensions of the drainage stone envelope around the leak detection pipe as depicted on the Piping at Perimeter Berm Detail should be specified.

Response: The dimensions of the leak detection piping envelope have been added to the leak detection pipe detail on Drawing C-300. This updated drawing is included in Attachment SME-2

iii. The north/south extent of the 6-inch deep by 6-foot wide base grade undercut leak detection sump, as indicated on the Leachate Collection & Leak Detection Cleanouts Detail, should be specified.

Response: The perpendicular dimension will be about 4 feet.

iv. Liner Termination. We assume that the impervious borrow specified within the anchor trench will achieve the specification for "clay borrow". If so, the terminology should be consistent. If not, a specification for impervious borrow should be established.

Response: The impervious borrow will meet the grain size requirements of the clay borrow. A material specification has been added to specification Section 02200. The update specification is included in Attachment SME-5.

f. **Sheet C-305, Sections and Details. Catch Basins 4K & 4L. For clarity, the depth below the pipe stub invert should be specified as 2 feet in accordance with the design presented in Appendix J, Stormwater Management Plan and Appendix K, Erosion and Sedimentation Control Plan of Volume I.**

Response: The detail of Catch Basins 4K and 4L has been updated to show a 2-foot sump. The updated drawing in included in Attachment SME-2.

g. **Sheet C-306, Sections and Details. We could not locate stormwater sizing calculations for the perimeter berm downspout and associated riprap plunge pool that is illustrated on this drawing. The calculations should be provided for our review at this time.**

Response: Calculations of the downspouts and plunge pool are provided in Attachment SME-4

11. Cell 11 Gas System

a. **Sheet 6 of 11 - LFG Infrastructure Development Plan - Stage 5**

i. The Cell 11 Infrastructure Development Plan provided with the LFG plans for the Application[6] (Sheet 4 of 14) indicate the installation of five additional LFG extraction wells not shown here (GW-22R, 32R, 41, 50, and 59) during Cell 11 development. The same plan also indicates that the main 12-inch header in Cell 11 is to connect to a stub at an existing 12-inch header in Cell 4.

Response: Wells GW-22R, 32R, 41, 50, and 59 will be installed as part of Cell 9 and Cell 10 LFG system construction events. Accordingly, the Cell 11 LFG Drawings will need to be updated with the as-built locations of these wells, and the remainder of the Cell 9 and Cell 10 GCCS infrastructure, prior to LFG system construction for Cell 11. To address the need for this update, a note was added to the cover sheet of the Cell 11 LFG Drawings.

Furthermore, the “existing” 12-inch diameter header referenced in the comment has not yet been installed. The header pipes shown on the Sheet 1 of 14 of the LFG Expansion Drawings are the locations of headers to be re-installed prior to final closure. Connection of these headers will be coordinated at a future date.

ii. The same Cell 11 Infrastructure Development Plan indicates that extraction wells GW-72, 71, and 61 are to be developed with Cell 11, while this sheet indicates that they are existing, and that existing headers in Cells 8 and 10 are to be extended and connected within Cell 11, while this sheet does not.

<u>Response</u>: The referenced wells are shown as proposed on Sheets 4 and 5 of the Cell 11 LFG Drawings. The “existing” headers for Cells 8 and 10 referenced in the comment have not yet been installed. The header pipes shown on Sheet 1 of 14 of the LFG Expansion Drawings are the locations of headers to be re-installed prior to final closure. Connection of these headers will be coordinated at a future date.

b. **<u>Sheet 8 of 11 - Details</u>**

i. A detail of penetration boot connections to the geomembrane intermediate cover should be prepared and included.

<u>Response</u>: Response: The Cell 11 LFG Drawings do not contemplate cover system types, and as such, there is no boot detail provided. Boot details for pipe penetrations are provided as Detail 3 on Sheet 13 of the LFG Expansion Drawings (see Attachment SH-3).

ii. A Well Schedule is included on this sheet. It appears that that the Bottom of Waste elevations listed for extraction wells GW-98 through GW-106 will all be four feet higher than what is listed. These wells are all located within the Cell 11 footprint and it is likely that the specified elevations were taken from base grades before liner construction instead of the top of the leachate collection system sand. The Total Well Depth and Bottom of Well Screen elevations for these wells will also need to be raised by four feet.

<u>Response</u>: The well schedule on Sheet 8 of the Cell 11 LFG Drawings was revised. (see Attachment SH-5).

III. VOLUME IV - OPERATIONS MANUAL[12]

Response: As noted previously and agreed to with Staff the updated site Operations Manual will be included with the 2015 annual report submitted by the end of April 2016. If specific review comments below are not addressed in this response they will be addressed in that updated manual.

A. Section 5.1, Cell Construction, notes that information regarding the design and construction of the leachate collection and storage systems can be found in Construction Documentation Reports for the individual cells. We recommend the development of an overall site plan depicting the location of all leachate collection, transport, and storage systems at the facility for inclusion in the Operations Manual (Manual). The plan can subsequently be updated as new construction projects are completed.

Response: This will be addressed in the updated site Operations Manual.

B. Section 5.2, Landfill Cell Intermediate Cover, specifies that intermediate cover may consist of a 40-mil geomembrane while Section 7.8.2 specifies a 30-mil geomembrane. A clarification should be made.

Response: This will be addressed in the updated site Operations Manual.

C. Section 6.1, Permitted Landfill Cell Development, references waste material approved for use as soft layer material. The Manual should list materials approved as soft layer material. Other materials can be considered and added to the list on a case by case basis as appropriate.

Response: This will be addressed in the updated site Operations Manual.

B. Section 7.2, Facility Access/Hours of Operation, refers to the application of calcium chloride to internal gravel access roads. We recommend alternatives to calcium chloride where practical.

Response: Agreed, calcium chloride is not typically utilized. This is clarified in the manual.

C. Section 7.7, Compaction (Waste Placement), sixth operational detail, states that "Waste setbacks, a minimum of 2 feet, shall be maintained at the outer edge of the waste to contain surface water runoff, to allow it to infiltrate into the waste". It is unclear what this statement is specifying and a clarification should be provided.

Response: This will be addressed in the updated site Operations Manual.

[12] *Juniper Ridge Landfill Expansion - Volume IV - Operations Manual,* Sevee & Maher Engineers, Inc., July 2015.

<u>D. Section 7.8.2, Intermediate Cover</u>, specifies either geomembrane or soil based intermediate cover. The Manual should include provisions for removal and stockpiling or disposal as appropriate of soil intermediate cover, if it is used, prior to waste placement above it.

> <u>Response</u>: This will be addressed in the updated site Operations Manual.

<u>E. Section 9.3, Annual Report</u>, includes a summary of items to be included in the annual operations report. The summary of leak detection system monitoring should include a comparison with previous years' monitoring data. Site changes that did not require Department approval and a summary of operator training during the year should also be included.

> <u>Response</u>: This will be addressed in the updated site Operations Manual.

<u>F. Appendix B, Compliance Self Audit Checklist</u>, should include a check to assure that leak detection monitoring is being conducted as specified in the Manual.

> <u>Response</u>: This will be added to the checklist completed as part of the annual report during the operations of the expansion

<u>G. The following apply to Appendix D, Cell Development Plans:</u>

1. Section 2.0, Cell 11 Development, states that chimney drains will be constructed of "tire shreds and piping". The chimney drain detail on Figure E-3 does not indicate that piping will be included. A clarification should be provided.

> <u>Response</u>: No piping will be placed in the chimney drains for the expansion cells.

2. The Cell Development Figures should specify where the perimeter downspouts, as detailed on Figure E-3, are to be located.

> <u>Response</u>: As shown on the Figure E-3 these downspouts will be part of the plunge pool construction so their locations are at the plunge pool locations shown on the cell development plans.

3. A cross-section of landfill access road construction, as discussed in Section 2.0, should be developed and included along with details needed to construct the access roads in areas of intermediate cover.

> <u>Response</u>: A suggested cross-section for an access road in areas where intermediate cover has been placed is shown on Figure 6 included in Attachment SME-4.

4. Details adequate to construct the stormwater discharge internal to the perimeter access road in the northeast corner of Cell 11 should be developed and included.

> Response: The approach and details for the northeast corner of Cell 11 are shown in the Cell 11 drawings (see C-102 and C-301) included in Appendix K of Volume III.

5. It is not clear how stormwater and leachate will be kept separate in the northwest and southeast corners of the Cell 11 stages as they are built out. Additional detail should be developed and included.

> Response: A detail has been prepared for these areas and is shown on Figure 7 included in Attachment SME-4.

6. Figure 5 depicts a triangular wedge of intermediate cover discharging directly into the active area of Stage 4 at about elevation 360 feet. The stormwater should be diverted from the operating area.

> Response: This wedge of intermediate cover will be discharged as indicated such that an adequate slope can be maintained for the temporary up-slope stormwater diversion berm.

H. The following apply to the details in Attachment E, Typical Operational Development details:

1. It is unclear what the function of the drainage geocomposite, fully embedded in clay above the anchor trench, as depicted on the Final Cover Termination at Top of Slopes is. A clarification should be provided.

> Response: The drainage geocomposite is just to keep the soil off the top of geomembrane, at the location were the future geomembrane will be attached.

2. A pipe boot detail should accompany the Leachate Collection Inlet detail depicted on Figure E-2.

> Response: This detail is shown on Figure 3 included in Attachment SME-4.

3. The Temporary Up-Slope Stormwater Diversion Berm detail on Figure E-2 indicates that the berm is to be constructed with "select waste". The term select waste should be defined.

> Response: Select waste consists of waste materials such as ashes, CDD fines, or contaminated soils with a maximum particle size of 4 inches, which would be easy to grade to construct the berm as shown.

4. The Downspout Section depicted on Figure E-3 specifies stripping of topsoil. It is unclear why topsoil would be present in this location.

> Response: The topsoil would have been placed during cell construction. These downspouts would be constructed once the cell is filled and clean water is shed from either the intermediate or final cover.

I. The Gas Monitoring Operation and Maintenance Manual is included in Appendix J. We have discussed that Manual previously in this memorandum.

> Response: See previous response to comments on the gas operations manual.

J. The Geotechnical Monitoring Plan[13] included in Appendix N discusses an annual review of measurements obtained from site transducers "installed below the landfill liner". We assume that the intent is to reference the transducers above the landfill liner within the leachate collection system. The Manual itself should address the procedures and frequency for monitoring of the transducers and response actions to take if elevated leachate levels are measured. We note that transducers installed within leachate collection systems at other landfills have been problematic and spare transducers should be maintained on-site in the event of failure.

> Response: The reference to the annual review of site transducers data is for both the transducers installed above and below the liner (i.e., "a review of pore pressure measurements obtained from site transducers installed below the landfill liner, and in the leachate collection system".) The procedures and frequency for monitoring of the transducers and response actions to take if elevated leachate levels are measured is better addressed in Section 8.0 Facility Inspection and Maintenance of the Site Operations Manual since that section addresses ongoing inspections at the site. Finally, we acknowledge the comment concerning potential long term operating issues with transducers. NEWSME's experience has been similar to the Departments. Typically, however, these transducers do remain operational during the active waste filling period for the cells.

K. The following apply to the Liner Action Plan[14] included in Appendix P:

1. In general we do not recommend implementing the leak detection program proposed. It is based on a formula to derive a Leak Detection System Action Level (LDSAL) considering action leakage rates (ALRs), base flow rates, baseline specific conductance, leachate specific conductance, and base flow rate to the underdrain. The formula was developed for a different landfill in Maine with a different liner and leak detection configuration (the base flow rate to the underdrain is not a consideration at JRL).

[13] *Juniper Ridge Landfill Expansion Application - Geotechnical Monitoring Plan,* Sevee & Maher Engineers, Inc., July 2015.

[14] *Juniper Ridge Landfill Expansion Application - Liner Action Plan*, Sevee & Maher Engineers, Inc., July 2015.

We recommend a simpler approach that establishes a two tiered action leakage rate program, based on gallons per acre per day (gpad), similar to what is done at most landfills with double liner systems. If our recommended approach is followed, the proposed ALRs of 4.6 and 92 gpad should be rounded for ease of monitoring and reporting. SME could consider rounding the ALR-I to 10 gpad and the ALR-II to 100 gpad.

> Response: The Department is correct that this approach was developed for another landfill in the State which has both landfill cells with underdrain monitoring, and dedicated leak detection system that is similar to what is proposed for the JRL expansion. The proposed liner leak action plan has worked at this site for both types of landfill configurations and provides a more robust method to monitor the primary liner performance. It is SME's experience that liner action plans that only rely on flow data have some inherent issues as discussed during our January 29, 2016 meeting, not the least of which is addressing the low frequency of pumping cycles typical of leak detection pumping systems. Incorporating conductivity measurements into the leak detection monitoring affords the ability to more quickly assess if liner leakage is occurring within a cell because of the relatively high conductivity levels of the leachate would affect the conductivity levels in leak detection fluids immediately if a leak occurred irrespective of flow levels. Included in Attachment SME-7 is an example of how this calculation would be applied to the JRL expansion site, using current leachate and underdrain flow data from the JRL site.

2. Section 2.2, Leak Detection Flow Rates, correctly notes that there will be liquids other than leakage water present in the leak detection system. These flows include construction water, consolidation water, impingement water, and condensation. There are calculation methods available to predict these flows and we recommend doing so prior to placing each cell in service in the event of, and to the extent they are, encountered during active monitoring of the leak detection system.

> Response: This is a good suggestion and will be done prior to placement of waste within each cell.

3. Leak detection system flow rates, along with leachate collection system flow rates, should be provided to the Department on a monthly basis in electronic data deliverable (EDD) format.

> Response: Understood, data will continue to be provided to the Department as completed and data is always available upon request.

EXHIBIT C

List of Attachments

SME-1	Volume I Supplemental Information.
SME-2	Volume III Updated Design and Cell 11 Drawings
SME-3	Volumes I and III Updated Application Tables
SME-4	Volume III and IV Updated and Supplemental Application Figures and Calculation
SME-5	Volume III Appendix A Updated Specifications
SME-6	Volume III Appendix F Supplemental Geotechnical Evaluations
SME-7	Volume IV Appendix P Supplemental Example of Liner Leakage Action Plan Calculations
SH-1	Volume III Appendix I Figure 2 – Projected LFG Collection Rates With and Without Proposed Expansion
SH-2	Volume III Appendix I Section 5.2 of the LFG System Expansion Design Report
SH-3	Volume III Appendix I Landfill Gas System Expansion Drawings
SH-4	Volume III Appendix I Header Sizing Calculation
SH-5	Volume III Appendix I Cell 11 Landfill Gas System Expansion Drawings
SH-6	Volume III Appendix I Technical Specification 02560 – Landfill Gas Extraction Wells
SH-7	Volume III Appendix I Construction Quality Assurance Plan
SH-8	Volume III Appendix I Operations & Maintenance Manual – Landfill Gas Management System

SME 1

VOLUME I SUPPLEMENTAL INFORMATION

- **SUPPLEMENT TO VOLUME I SECTION 9 CAPITAL CLOSURE AND POST-CLOSURE COSTS FOR 2015**

- **SUPPLEMENT TO VOLUME I SECTION 11 ALTERNATE DESIGN ASSESSMENT FOR 12 INCH COMPACTED LIFT THICKNESS**

SUPPLEMENT TO VOLUME I SECTION 9

CAPITAL CLOSURE AND POST-CLOSURE COSTS FOR 2015

SME
Sevee & Maher Engineers, Inc.

ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

April 14, 2015

15010
20150415 JL.doc

Mr. Jeremy Labbe
Environmental Compliance Manager
Pine Tree Landfill
358 Emerson Mill Rd
Hampden, ME 04444

Subject: Update of Opinion of Capital Closure and Post-Closure Costs
For Calendar Year 2015
Juniper Ridge Landfill
Old Town, Maine

Dear Jeremy:

As requested by NEWSME Landfill Operations, LLC (NEWSME), Sevee & Maher Engineers, Inc. (SME) has updated our opinions of capital closure and post-closure costs for the Juniper Ridge Landfill (JRL) in Old Town, Maine for calendar year 2015. The capital closure cost is for those cells that, as of the end of the calendar year 2015, have been or will be constructed and operational, but have not received final cover. These include Cells 1, 2, 3A, 3B, 4, 5, 6, 7, 8 and 9. In total, these landfill cells have approximately 64.1 acres of closure area. Our opinion of the capital closure cost to close the 64.1 acres is $13,244,248. This cost is based on a per-acre closure cost presented in Table 1, for a final cover consistent with the final waste grades and cover components requirements of Maine Department of Environmental Protection (MEDEP) Solid Waste Management Rules (SWMRs). The unit costs used to develop the closure cost are from material unit costs obtained for the 2014 Cell 9 construction project at JRL, and similar projects in central Maine adjusted for cover versus cell construction.

The post-closure monitoring and maintenance cost for the site (developed as of December 2015) is $10,280,390 for the items presented in Table 2. The post-closure costs assume a 30-year post-closure period and are based on 2015 dollars.

Our opinion of closure and post-closure costs is based on the following assumptions.

1. The closure of the individual cells will consist of placing final cover over the areas of the developed landfill which have not received final cover. Note that operational costs such as placement and removal of intermediate cover, and operational waste grading are not included in the final cover costs presented herein. The cost for the areas where an active gas system has been installed as part of landfill operations will continue to operate during the post-closure

4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021 • Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

period. In areas that currently do not have active gas collection, a gas extraction system will be installed as part of the final cover construction.

2. The final cover of these cells will consist of the components outlined in the current SWMRs. Our opinion of closure costs are based on unit material prices developed from the construction bids for NEWSME's Cell 9 project and other similar projects in the central Maine area adjusted for closure versus cell construction. These costs are also based on our current understanding of site conditions. Actual closure costs will vary and are dependent upon the actual nature and extent of waste placement, timing of closure, and other factors not evident at this time.

3. The post-closure costs include costs for post-closure activities including landfill inspection, water quality monitoring, leachate management, general site maintenance, gas treatment and maintenance, and engineering for the entire facility. These post-closure costs are based on our current understanding of site conditions, and projections of both leachate and landfill gas quantity and quality, and costs associated with treatment and disposal. Actual post-closure costs will vary and are dependent upon the actual nature of site conditions at the time of closure, long-term management decisions of NEWSME and the Regulators, and other factors not evident at this time.

If there are any questions concerning the cost issues presented in this letter, please feel free to contact us.

Sincerely,

SEVEE & MAHER ENGINEERS, INC.

Michael S. Booth, P.E.
Project Engineer

Attachments
Table 1 - Opinion of Final Cover Costs for the JRL as of December 2015
Table 2 –Opinion of Post-Closure Monitoring and Maintenance Costs for Juniper Ridge Landfill as developed in Calendar Year 2015

cc: Toni King, NEWSME
Wayne Boyd, NEWSME

TABLE 1

OPINION OF FINAL COVER COSTS FOR JUNIPER RIDGE LANDFILL FOR LANDFILL AREA
DEVELOPED AS OF DECEMBER 2015

JUNIPER RIDGE LANDFILL PER-ACRE FINAL COVER COSTS without gas collection (Update 2/2015)				
ITEM	UNIT	QUANT.	UNIT COST(1)	TOTAL
Mobilization	L.S.	1	$20,000	$20,000
Erosion Control	L.S.	1	$3,000	$3,000
Active Gas System Modifications	L.S.	1	$21,700	$21,700
Site Grading	L.S.	1	$3,000	$3,000
Drainage Terraces	L.S.	1	$16,800	$16,800
24" compacted clay	C.Y.	3,230	$16.75	$54,103
Texture Membrane	SQ.FT.	43,600	$0.54	$23,544
12" Drainage Sand	C.Y.	1,620	$18.50	$29,970
12" Vegetative Cover	C.Y.	1,620	$20	$32,400
Seed & Mulch	L.S.	1	$2,500	$2,500
Engineer/Const. Monitoring	L.S.	1	$19,000	$19,000
			Total	$226,017

JUNIPER RIDGE LANDFILL PER-ACRE FINAL COVER COSTS w/existing gas collect (Update 2/2015)				
ITEM	UNIT	QUANT.	UNIT COST(1)	TOTAL
Mobilization	L.S.	1	$20,000	$20,000
Erosion Control	L.S.	1	$3,000	$3,000
Site Grading	L.S.	1	$3,000	$3,000
Drainage Terraces	L.S.	1	$16,800	$16,800
24" compacted clay	C.Y.	3,230	$16.75	$54,103
Texture Membrane	SQ.FT.	43,600	$0.54	$23,544
12" Sand Common Borrow	C.Y.	1,620	$18.50	$29,970
12" Vegetative Cover	C.Y.	1,620	$20	$32,400
Seed & Mulch	L.S.	1	$2,500	$2,500
Engineer/Const. Monitoring	L.S.	1	$19,000	$19,000
			Total	$204,317

(1) Unit Cost based upon Third Party Construction cost (Cell 9 bids October 2014) adjusted to reflect the cover construction on 3H to 1V sideslopes.

	Acres	Closure Cost
Area with Existing Gas Collection	57.3	$11,707,335
Area without Gas Collection (Cell 8)	6.8	$1,536,912
	Total	$13,244,248

TABLE 2

OPINION OF POST-CLOSURE MONITORING AND MAINTENANCE COSTS FOR JUNIPER RIDGE LANDFILL AS DEVELOPED IN CALENDAR YEAR 2015

ITEM	OPINION OF YEARLY COSTS	TOTAL COST FOR 30 YEAR PERIOD	ASSUMPTIONS
Leachate Collection, Transport and Disposal			
A. Electrical Costs to Operate Pump Station	$910	$27,300	Assumes an average of 450 hours per year for a15 Hp pump
B. Disposal Costs for Leachate Years 1-30	$56,653	$1,699,590	decreasing rate over closure beginning with11.4 M at year 1 decreasing to 0.157 M at year 30 , transport costs $0.021/gal
C. Annual Leachate Testing	$4,600	$138,000	Annual cost for pretreatment testing
	Subtotal Total	**$1,864,890**	
Post Closure Water Quality & Methane Gas Monitoring			
A.1 Collect Samples From 20 Wells, 7 underdrains,1 leak detection,2 leachate & 8 Surface Waters for 3 Rounds/Year & Methane Measurements From Wells 3 Times per Year	$36,500	$182,500	Assumes 2 rounds detect monitor para. 1 round extended list for year 1-5
A.2 Collect Samples From 20 Wells,7 underdrains,1 leak detection,2 leachate & 8 Surface Waters for 2 Rounds/Year & Methane Measurements From Wells 2 Times per Year	$24,300	$121,500	Assumes 2 rounds, one detect monitor para. & one round extended list for years 6-10
A.3 Collect Samples From 20 Wells,7 underdrains,1 leak detection,2 leachate & 8 Surface Waters for 1 Round/Year & Methane Measurements From Wells 1 Time per Year	$12,200	$244,000	Assumes one round extended list for years 11-30
B.1 Analyses of 41 samples 3 Times per Year	$52,500	$262,500	Assumes 20 wells,7 underdrains,1 leak detection,2 leachate, 8 surface, & 3 QA/QC
B.2 Analyses of 41 Sample 2 Times per Year	$35,000	$175,000	Assumes 20 wells,7 underdrains,1 leak detection,2 leachate, 8 surface, & 3 QA/QC
B.3 Analyses of 41 Sample 1 Time per Year	$17,500	$350,000	Assumes 20 wells,7 underdrains,1 leak detection,2 leachate, 8 surface, & 3 QA/QC
C. Compile Data and Submit to MDEP	$7,500	$225,000	Assumes Report prepared and submitted to MDEP after each sample round
Subtotal Yearly Cost Years 1-5	$96,500		
Subtotal Yearly Cost Years 6-10	$66,800		
Subtotal Yearly Cost Years 11-30	$37,200		
	Subtotal Total	**$1,560,500**	
Landfill Inspection			
A. Monthly Site Walk Over & Report Generation	$8,200	$246,000	Assumes 8 hr per month @ $85/hr
Subtotal	**$8,200**	**$246,000**	
Active Landfill Gas Extraction System			
A. Gas Collection Equipment Replacement	$10,000	$300,000	General equipment replacement including well heads, condensate pumps etc.
B. Flare Maintenance	$5,500	$165,000	Replacement of flare parts such as flame arrestor media etc.
C. Blower Maintenance	$6,000	$180,000	Routine inspection and maintenance of blower & control system
D. System Operation and Inspection	$5,000	$150,000	General system operation & maintenance
E. Well Tuning	$10,000	$300,000	Well tuning once per month
F. Compliance Monitoring	$5,000	$150,000	
G. Electrical Costs to Operate Blowers, Heat & Control Panel Year 1-30	$60,700	$1,821,000	electricity for blowers assume varying horsepoer requirement as gas decreases @$0.18/kwhr
H. Landfill Gas Treatment Costs Year 1-5	$70,300	$2,109,000	included treatment cost for H2S removal to 1000 ppm using thiopaq system
	Subtotal Total	**$5,175,000**	
Landfill Maintenance			
A. Cover Maintenance Include Annual Mowing & Erosion Repair	$5,900	$177,000	Assumes 3 man crew 7.5 days/ year
B.1 Pump Stations Inspections	$10,400	$312,000	Assumes 4 hr week @ $50 per hour
B.2 Pump Replacement	$4,500	$135,000	Assumes replace 9 onsite pumps every 5 years at $2,500 a piece
C. General Site Maintenance	$8,000	$240,000	Assumes snow plowing 20 storms per year @ $400 per storm
D. Leachate Line Cleaning	$15,000	$450,000	Assumes leachate line cleaning 2 time per years 1-5, once per year 6-10, then every other year, years 11-30 @ $18,000 per cleaning
Subtotal	**$43,800**	**$1,314,000**	
Professional Services			
A. Engineering Services	$4,000	$120,000	General Services
Subtotal	**$4,000**	**$120,000**	
TOTAL		**$10,280,390**	

SUPPLEMENT TO VOLUME I SECTION 11

ALTERNATE DESIGN ASSESSMENT FOR 12 INCH COMPACTED LIFT THICKNESS

ALTERNATIVE DESIGN ASSESSMENT
FOR 12 INCH COMPACTED LIFT THICKNESS

This alternative design assessment addresses an alternative method for placing both the barrier soil layer of the liner systems and the base soil below the liner to those required by Chapter 401.2.D.(1)(g)(iv) and Chapter 401.2.D(3)(e) of the Maine Solid Waste Management Rules. These requirements specify that soils be placed with “maximum allowable compacted lift thickness of 9 inches.” The proposed alternative is to allow these materials to be placed in one compacted lift thickness of 12 inches, consistent with the practice that has been used successfully to construct the last four landfill cells at the Juniper Ridge Landfill (JRL). As part of each of these construction projects, a test pad program was completed, which demonstrated the current construction techniques used to compact these soils and achieve the performance criteria (i.e., densities, moisture content, and hydraulic conductivity) required by the Rules. A similar test pad program will be undertaken during each expansion cell construction project.

This assessment addresses the items in Chapter 401.2.E to demonstrate technical equivalency of the proposed alternative.

(**1**) **A discussion of the benefits of the proposed alternative technology**

The benefit of the proposed alternate technology is that it allows construction contractors to place the barrier soil layers and base soils in lift thicknesses that are consistent with the capabilities of equipment used currently in landfill construction, thus resulting in a more efficient construction technique. In addition, because the clay barrier soil thickness for this project is one foot, placing the soil in more than one lift presents a number of logistical construction issues. These issues are associated with placing the clay soil in thin lifts at moisture contents of zero to four percent over the optimum using a kneading action as required by Chapter 401.1.(g)(iii). Placing the clay to meet this requirement in a thin lift (i.e., less than 9 inches) often results in “peeling” of the clay off the sub-base material as it is compacted with various roller/kneading type compaction equipment. If thin lifts are required, then it is necessary to scarify and then re-compact a significant portion of previously placed clay to achieve good lift bonding. These issues have been observed less frequently by using a 12-inch compacted lift thickness.

(2) A discussion of the risks and drawbacks of the proposed alternative technology

The only risk or drawback to this approach would be the reduced ability to achieve a uniform compacted effort throughout the entire lift thickness. This would have the potential to affect the hydraulic properties of the soils. This risk is reduced, however, by using a test pad program, completed as part of each cell or cover construction project, to demonstrate that both uniform compaction and hydraulic conductivity are achieved using the specific equipment being used for the project.

(3) An assessment of similar applications of the proposed alternative technology

This proposed application has been used successfully at the JRL for the last four landfill cell construction projects. It also has been used successfully at other landfill construction projects in the State, including the Pine Tree Landfill closure, completed between 2008 and 2010.

(4) **A demonstration that the alternative technology will provide equal or superior performance to the component it is proposed to replace, or that its inclusion within a system will result in equal or superior performance of that system**

The construction specifications include the requirement that a test pad program be completed as part of each construction project when the soils will be placed in the proposed manner (i.e., in a 12-inch lift). The test pad will be used to demonstrate that the construction equipment being used on the project has the ability to achieve the required material properties for the compacted soils. The use of a test pad program to allow use of a thicker lift is a well-accepted construction process that has been used previously at JRL.

(5) An assessment of the feasibility of constructing the proposed alternative, including the ability to provide an adequate level of quality assurance and

quality control. A demonstration of the feasibility of construction may be required

This technique is feasible and the preferred construction technique by contractors who perform this work. The proposed QA/QC procedures that are outlined in the project specifications (i.e., Section 02200 part 3.9 C.) have been previously approved by the MEDEP for other landfill construction projects, including projects at JRL. The proposed QA/QC procedures to demonstrate the requirements of the Rules can be achieved for projects using this proposed soil placement techniques.

(**6**) **An assessment of the likelihood that the proposed alternative will perform as designed through landfill operations, closure, and post-closure periods**

Past uses of this soil placement technique demonstrate that this technique will perform as designed through landfill operation, closure and post closure periods. This is further documented in a study that was completed on the Pine Tree Landfill final cover five years after closure. In that study (SME 2015), the clay used in the cover material was exposed and samples were taken of the clay and observed for continuity and hydraulic conductivity five years after placement. The clay that was evaluated by this study was placed in a manner similar to what is proposed for the expansion, and has performed as designed.

Reference: Sevee & Maher Engineers, Inc. 2015 Pine Tree Landfill Facility, Hampden Maine Condition 3 Compliance Department Order # S-001987-WD-HD-C/S#001987-WN-MI-N Results of Investigation of Landfill Cover System dated November 12, 2015.

SME 2

VOLUME III UPDATED DESIGN AND CELL 11 DRAWINGS

- **VOLUME III APPENDIX F DESIGN DRAWINGS**
  - **C-104 LEAK DETECTION PIPING PLAN**
  - **C-107 FINAL SITE DRAINAGE PLAN**
  - **C-201 TRAVERSE CROSS SECTIONS STA 14+00 TO STA 24+00**
  - **C-202 TRAVERSE CROSS SECTIONS STA 26+00 TO STA 28+00**
  - **C-300 SECTIONS AND DETAILS**
  - **C-301 SECTIONS AND DETAILS**
  - **C-302 SECTIONS AND DETAILS**
  - **C-306 SECTIONS AND DETAILS**
  - **C-307 SECTIONS AND DETAILS**
  - **C-308 SECTIONS AND DETAILS**

- **VOLUME III APPENDIX K CELL 11 DESIGN DRAWINGS**
  - **C-102 BASE GRADING PLAN**
  - **C-104 LEACHATE COLLECTION PIPING PLAN**
  - **C-200 FORCE MAIN AND LANDFILL GAS HEADER PLAN AND PROFILE**
  - **C-300 SECTIONS AND DETAILS**
  - **C-301 SECTIONS AND DETAILS**
  - **C-302 SECTIONS AND DETAILS**
  - **C-303 SECTIONS AND DETAILS**
  - **C-304 SECTIONS AND DETAILS**
  - **C-305 SECTIONS AND DETAILS**
  - **C-306 SECTIONS AND DETAILS**

VOLUME III APPENDIX F DESIGN DRAWINGS

- **C-104 LEAK DETECTION PIPING PLAN**
- **C-107 FINAL SITE DRAINAGE PLAN**
- **C-201 TRAVERSE CROSS SECTIONS STA 14+00 TO STA 24+00**
- **C-202 TRAVERSE CROSS SECTIONS STA 26+00 TO STA 28+00**
- **C-300 SECTIONS AND DETAILS**
- **C-301 SECTIONS AND DETAILS**
- **C-302 SECTIONS AND DETAILS**
- **C-306 SECTIONS AND DETAILS**
- **C-307 SECTIONS AND DETAILS**
- **C-308 SECTIONS AND DETAILS**

LEGEND

7 CELL IDENTIFICATION

WETLAND DELINEATED

WETLAND PHOTO INTERPRETED

NOTES:

1. BASE MAP PREPARED BY AERIAL SURVEY & PHOTO INC., NORRIDGEWOCK, MAINE. PHOTO DATE 12/31/14. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE. STANDARD PRACTICE DICTATES THAT PLANS COMPILED IN THIS MANNER SHOULD BE FIELD VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION. SITE BENCHMARK INFORMATION:

   PLUG 1 IS A BRASS PLUG ON FORMER LEACHATE POND PUMP STATION LOCATED AT COORDINATES NORTHING 478242.05, EASTING 925376.35 ELEVATION 167.93

2. WETLAND BOUNDARIES DELINEATED BY WOODLOT ALTERNATIVES, INC. IN 2004 AND STANTEC CONSULTING SERVICES 2008, 2014 AND 2015.

3. AREA IDENTIFIED TO RECEIVE UNDERDRAIN SHALL BE UNDERCUT TO 1-FOOT BELOW BASE GRADE. THE UNDERDRAIN SHALL BE CONSTRUCTED ON THE RESULTING GRADE.

75 0 150 300 FEET

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE ... LICENSED PROFESSIONAL ENGINEER

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

LEAK DETECTION PIPING PLAN

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	3/4/2015
CHECKED BY:	
LMN:	LD-PIPE
CTB:	SME-STD

JOB NO. 14101.00 DWG FILE BASE

C-104

LEGEND

WETLAND DELINEATED

WETLAND PHOTO INTERPRETED

NOTES:

1. BASE MAP PREPARED BY AERIAL SURVEY & PHOTO INC., NORRIDGEWOCK, MAINE. PHOTO DATE 12/31/14. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE. STANDARD PRACTICE DICTATES THAT PLANS COMPILED IN THIS MANNER SHOULD BE FIELD VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION. SITE BENCHMARK INFORMATION:

   PLUG 1 IS A BRASS PLUG ON FORMER LEACHATE POND PUMP STATION LOCATED AT COORDINATES NORTHING 478242.05, EASTING 925376.35 ELEVATION 167.93

2. WETLAND BOUNDARIES DELINEATED BY WOODLOT ALTERNATIVES, INC. IN 2004 AND STANTEC CONSULTING SERVICES 2008, 2014 AND 2015.

3. PERMITTED LANDFILL FINAL WASTE GRADES REPRESENT GRADES PRIOR TO CONSTRUCTION OF FINAL COVER SYSTEM.

4. PROPOSED FINAL WASTE GRADES REPRESENT GRADES PRIOR TO CONSTRUCTION OF FINAL COVER SYSTEM.

5. CULVERT SCHEDULE IS SHOWN ON DRAWING C-306. CULVERT SCHEDULE INCLUDES CULVERTS FOR DETENTION BASIN OUTLET STRUCTURES.

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE · LICENSED PROFESSIONAL ENGINEER

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

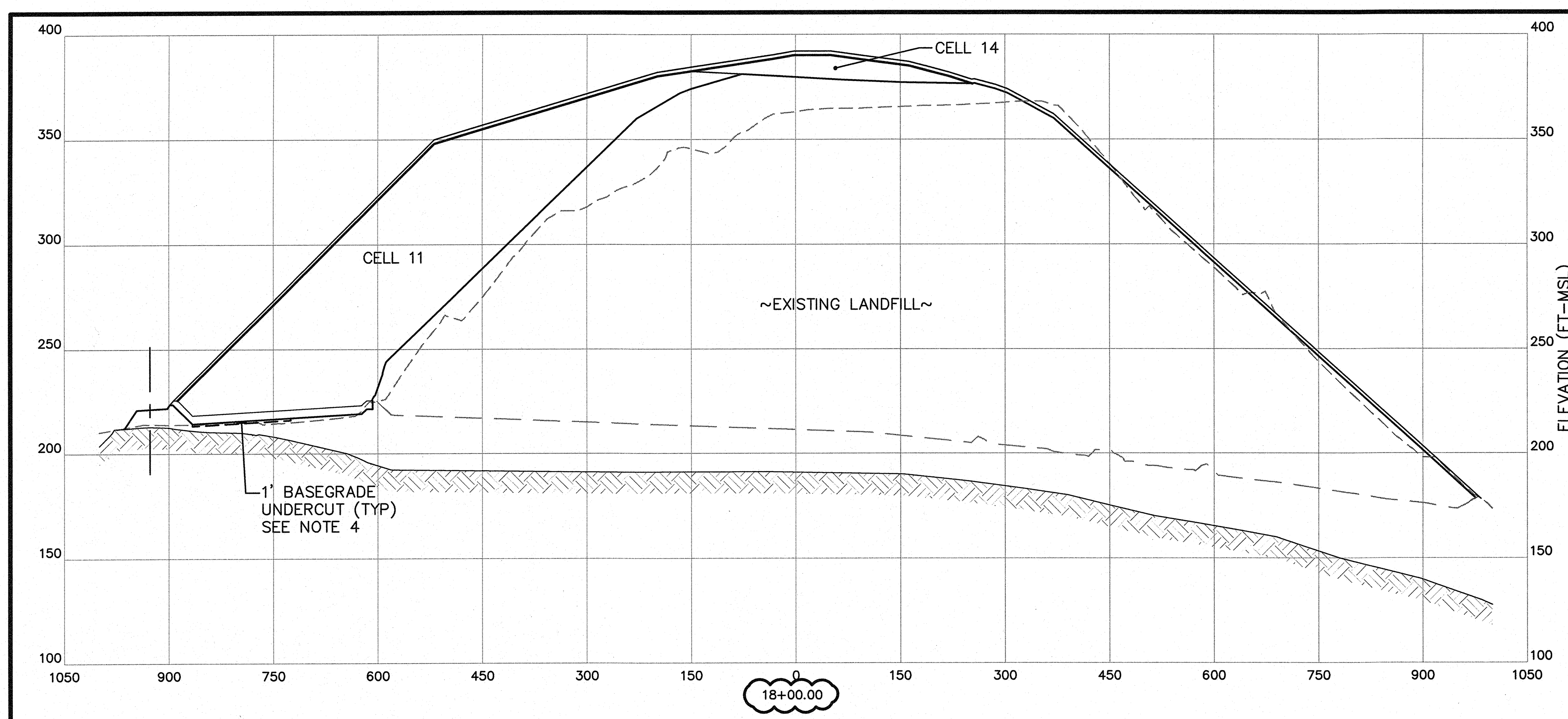
FINAL SITE DRAINAGE PLAN

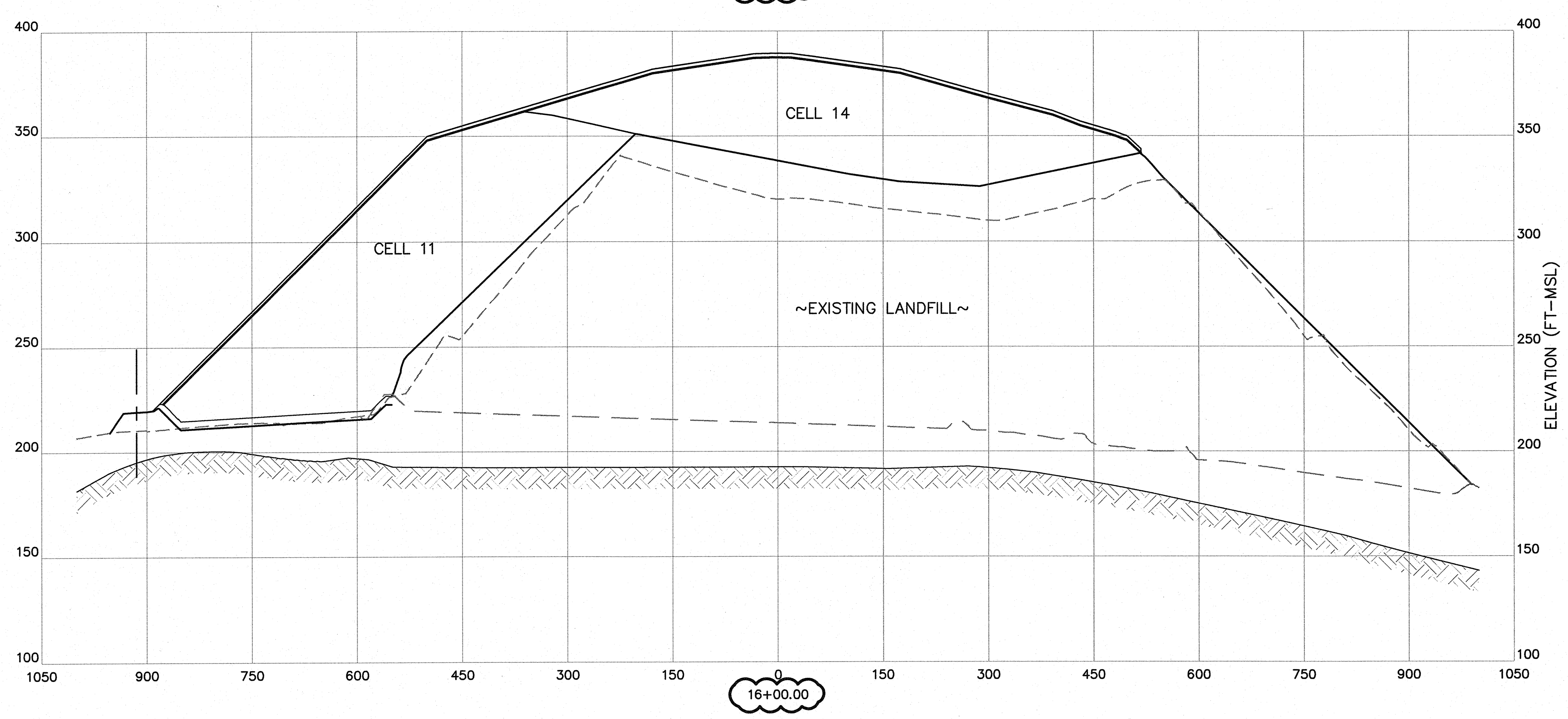
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

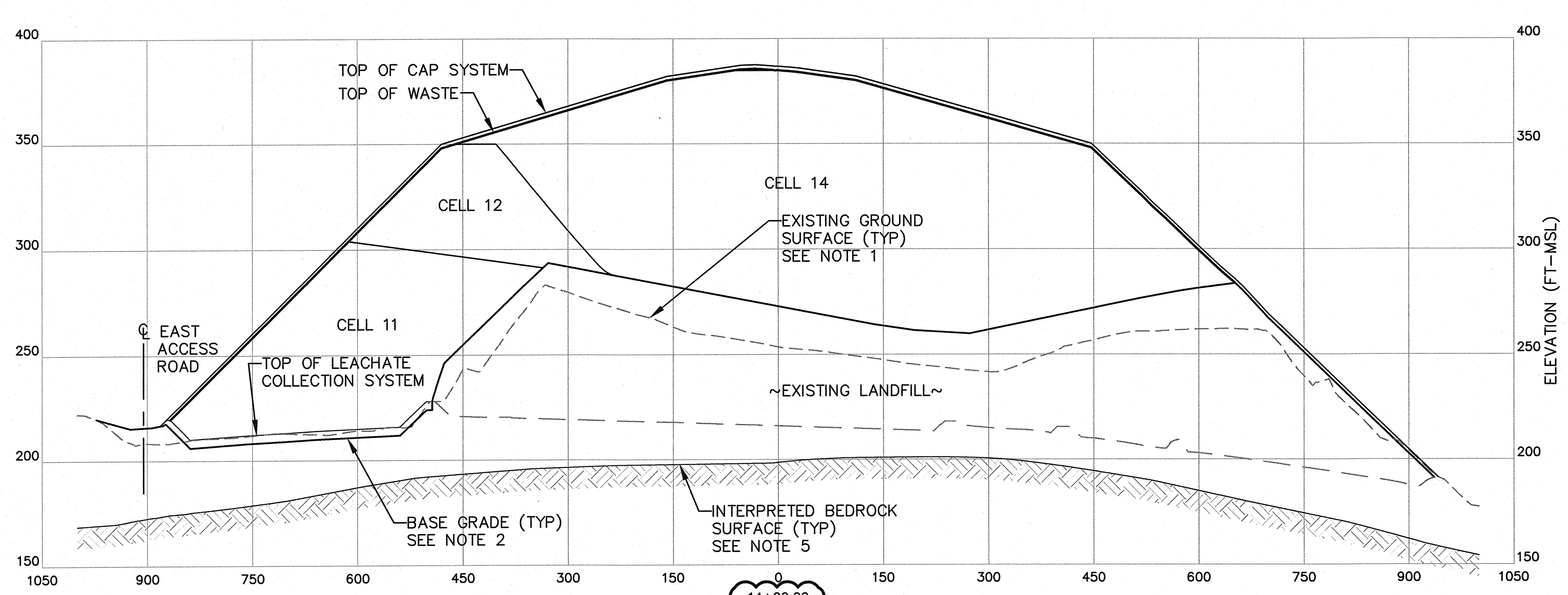
DESIGN BY: PCM
DRAWN BY: SJM
DATE: 3/4/2015
CHECKED BY:
LMN: FINAL-DRAIN
CTB: SME-STD

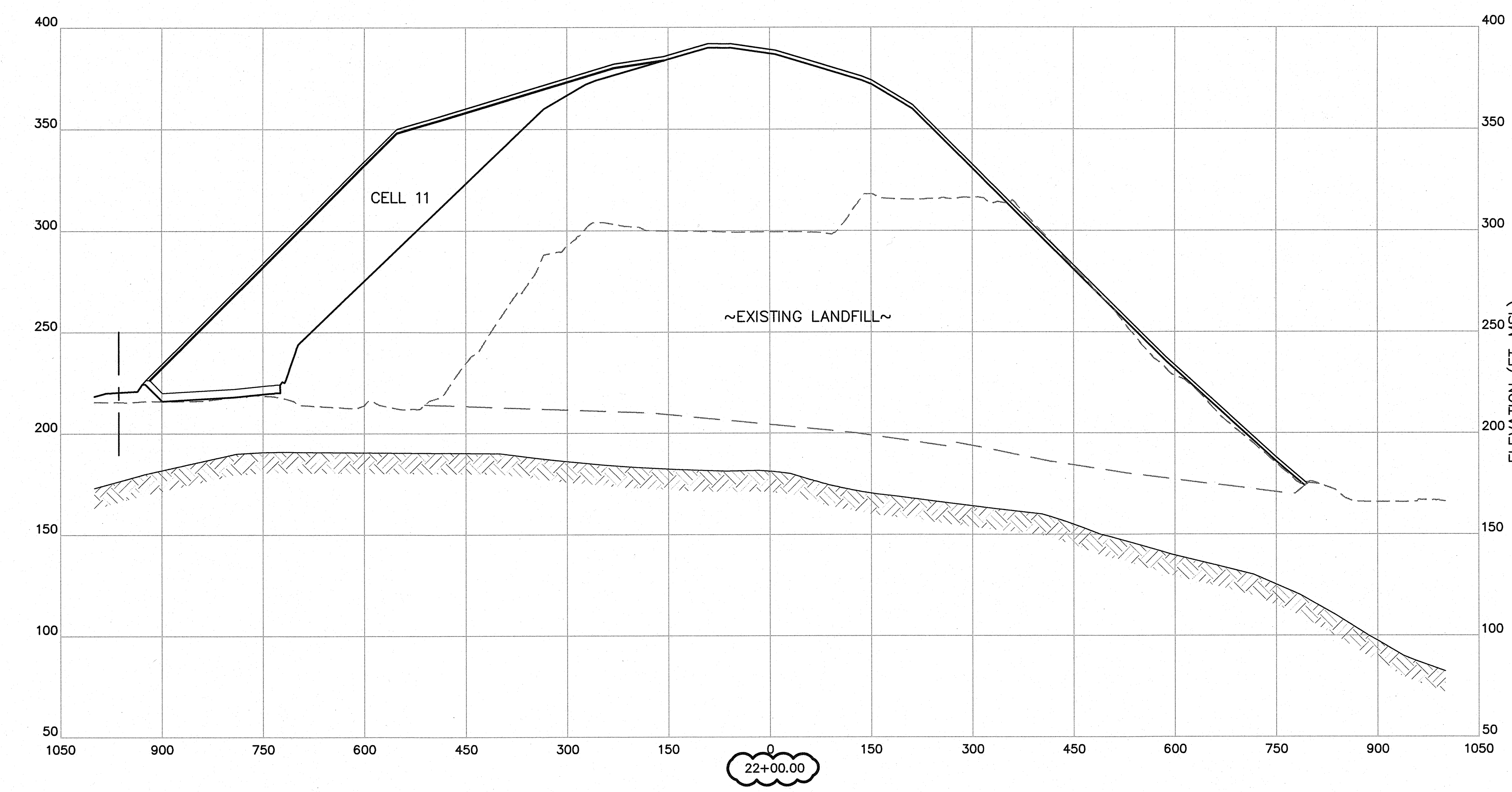
JOB NO. 14101.00 DWG FILE BASE

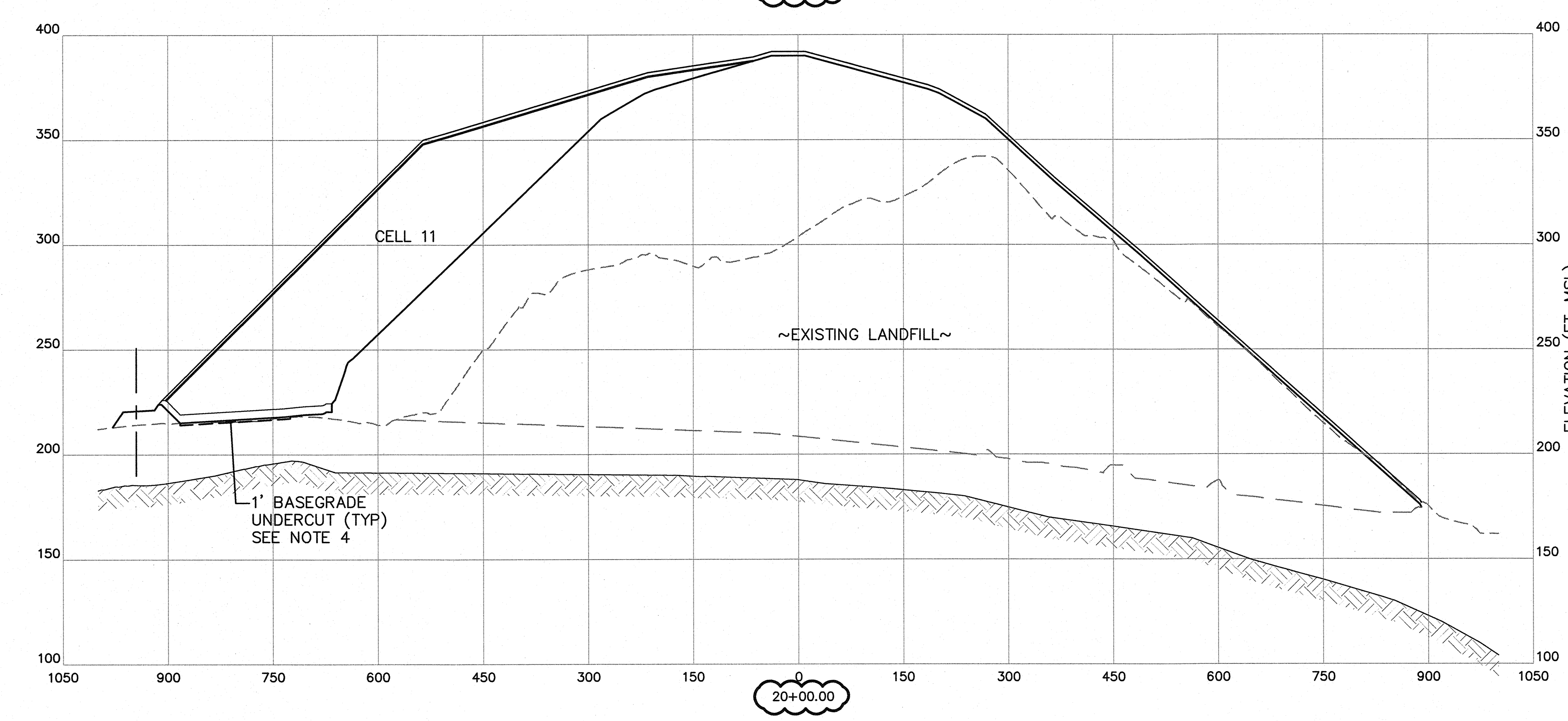
C-107











NOTES:

1. EXISTING GROUND SURFACE INTERPRETED FROM TOPOGRAPHIC BASE MAP PREPARED BY AERIAL SURVEY AND PHOTO, NORRIDGEWOCK, MAINE. PHOTO DATED 12/31/2014.
2. PROPOSED EXPANSION GRADES WITHIN THE PROPOSED SOLID WASTE LIMIT REPRESENT BASE GRADES PRIOR TO PLACEMENT OF THE IMPORTED CLAY AND CONSTRUCTION OF THE LINER SYSTEM. THE PROPOSED GRADES SHOWN OUTSIDE THE PROPOSED SOLID WASTE LIMIT ARE SUBBASE ROAD GRADES.
3. AREA IDENTIFIED TO RECEIVE UNDERDRAIN SHALL BE UNDERCUT TO 1-FOOT BELOW BASE GRADE. THE UNDERDRAIN SHALL BE CONSTRUCTED ON THE RESULTING GRADE.
4. AREA IDENTIFIED SHALL BE UNDERCUT 1-FOOT BELOW BASEGRADE. THE SECONDARY LINER SHALL BE AUGMENTED IN THIS AREA AS DETAILED ON DRAWING C-300.
5. BEDROCK SURFACE INTERPRETATION FROM SECTION 4.2 VOLUME II SITE ASSESSMENT REPORT FOR JUNIPER RIDGE LANDFILL EXPANSION APPLICATION, DATED JULY, 2014.

75 0 150 300 FEET (HORZ)
25 0 50 100 FEET (VERT)

REV.	BY	DATE	STATUS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION
	PCM	2/2016	REVISED PER MEDEP COMMENTS

STATE OF MAINE — GUY H. COTE — No. 5… — LICENSED PROFESSIONAL ENGINEER — 2/10/16

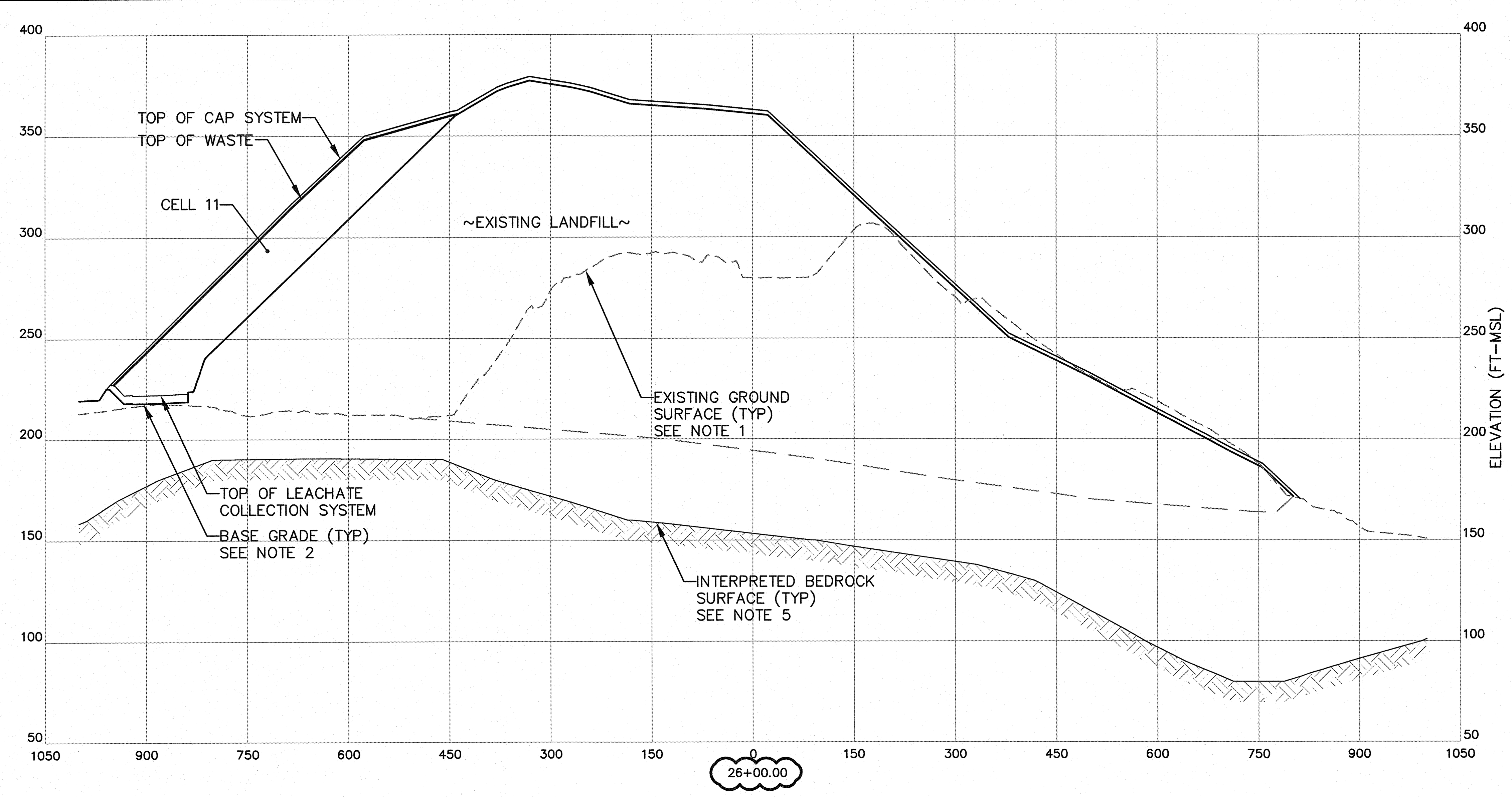
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

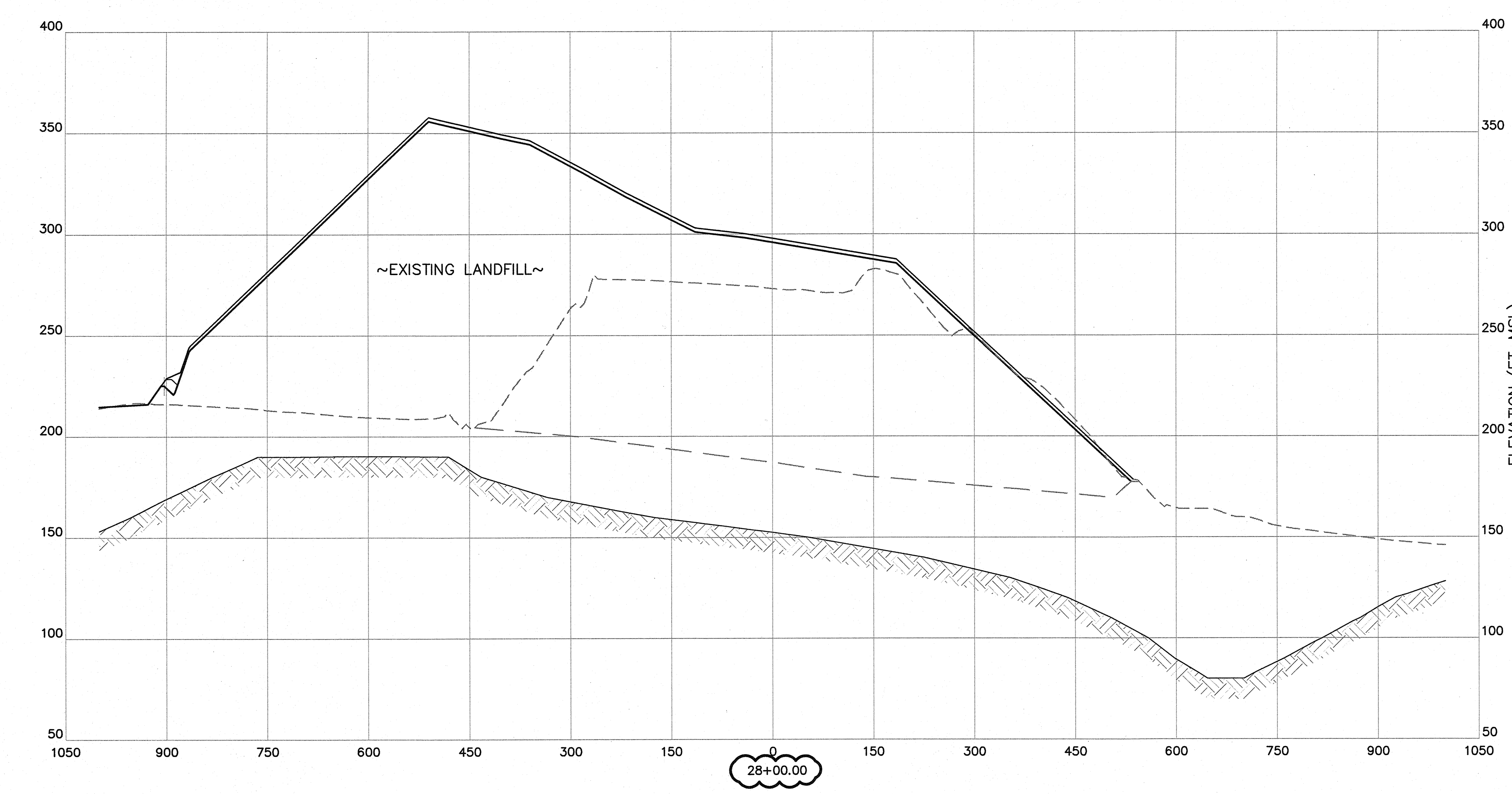
TRANSVERSE CROSS SECTIONS STA 14+00 TO STA 24+00

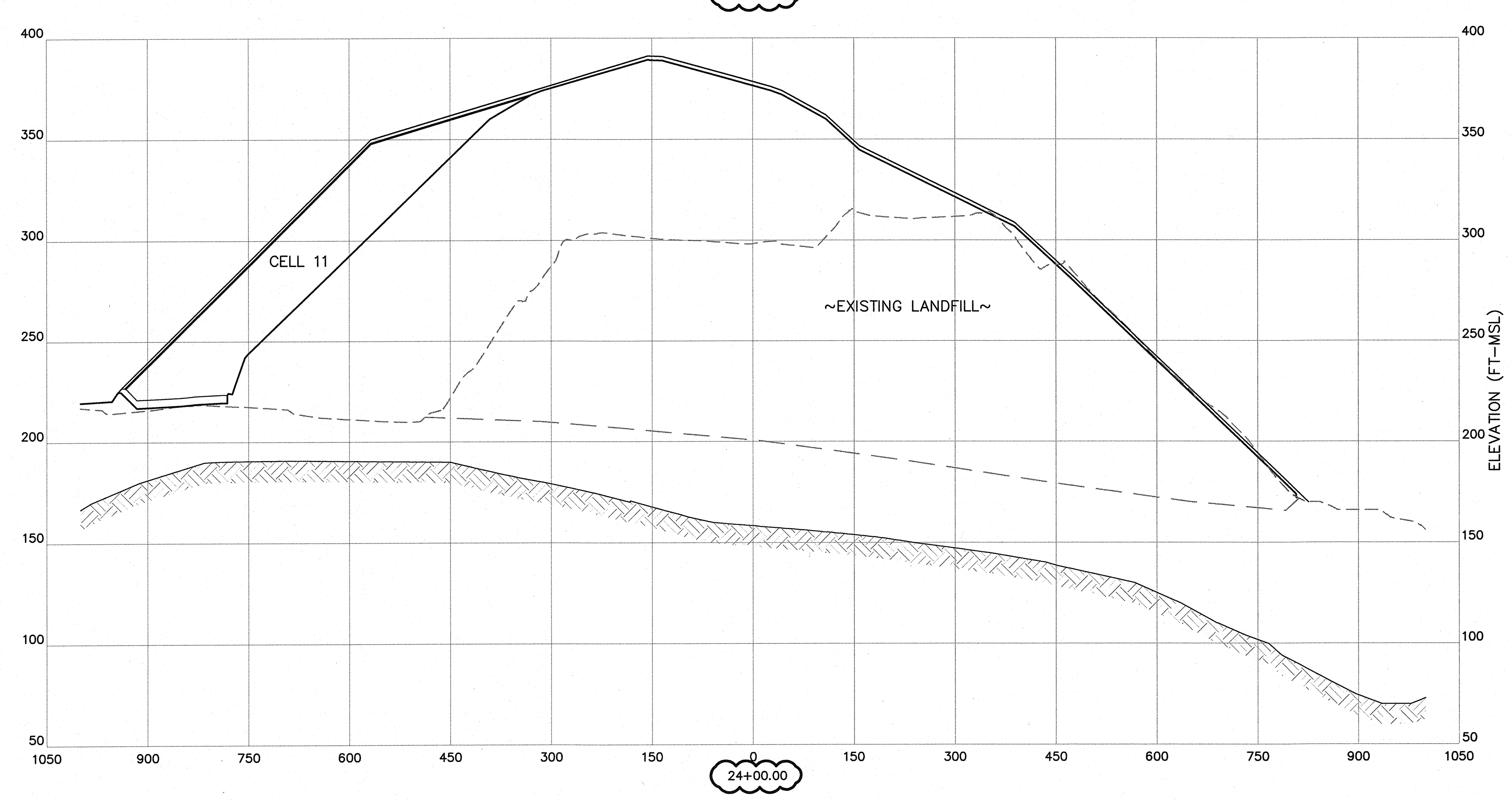
SME Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	5/2015
CHECKED BY:	
LMN:	
CTB:	SME-STD

JOB NO. 14101 DWG FILE SECTIONS C-201

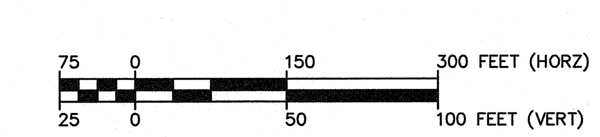






NOTES:

1. EXISTING GROUND SURFACE INTERPRETED FROM TOPOGRAPHIC BASE MAP PREPARED BY AERIAL SURVEY AND PHOTO, NORRIDGEWOCK, MAINE. PHOTO DATED 12/31/2014.
2. PROPOSED EXPANSION GRADES WITHIN THE PROPOSED SOLID WASTE LIMIT REPRESENT BASE GRADES PRIOR TO PLACEMENT OF THE IMPORTED CLAY AND CONSTRUCTION OF THE LINER SYSTEM. THE PROPOSED GRADES SHOWN OUTSIDE THE PROPOSED SOLID WASTE LIMIT ARE SUBBASE ROAD GRADES.
3. AREA IDENTIFIED TO RECEIVE UNDERDRAIN SHALL BE UNDERCUT TO 1-FOOT BELOW BASE GRADE. THE UNDERDRAIN SHALL BE CONSTRUCTED ON THE RESULTING GRADE.
4. AREA IDENTIFIED SHALL BE UNDERCUT 1-FOOT BELOW BASEGRADE. THE SECONDARY LINER SHALL BE AUGMENTED IN THIS AREA AS DETAILED ON DRAWING C-300.
5. BEDROCK SURFACE INTERPRETATION FROM SECTION 4.2 VOLUME II SITE ASSESSMENT REPORT FOR JUNIPER RIDGE LANDFILL EXPANSION APPLICATION, DATED JULY, 2014.



REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE · LICENSED · PROFESSIONAL ENGINEER

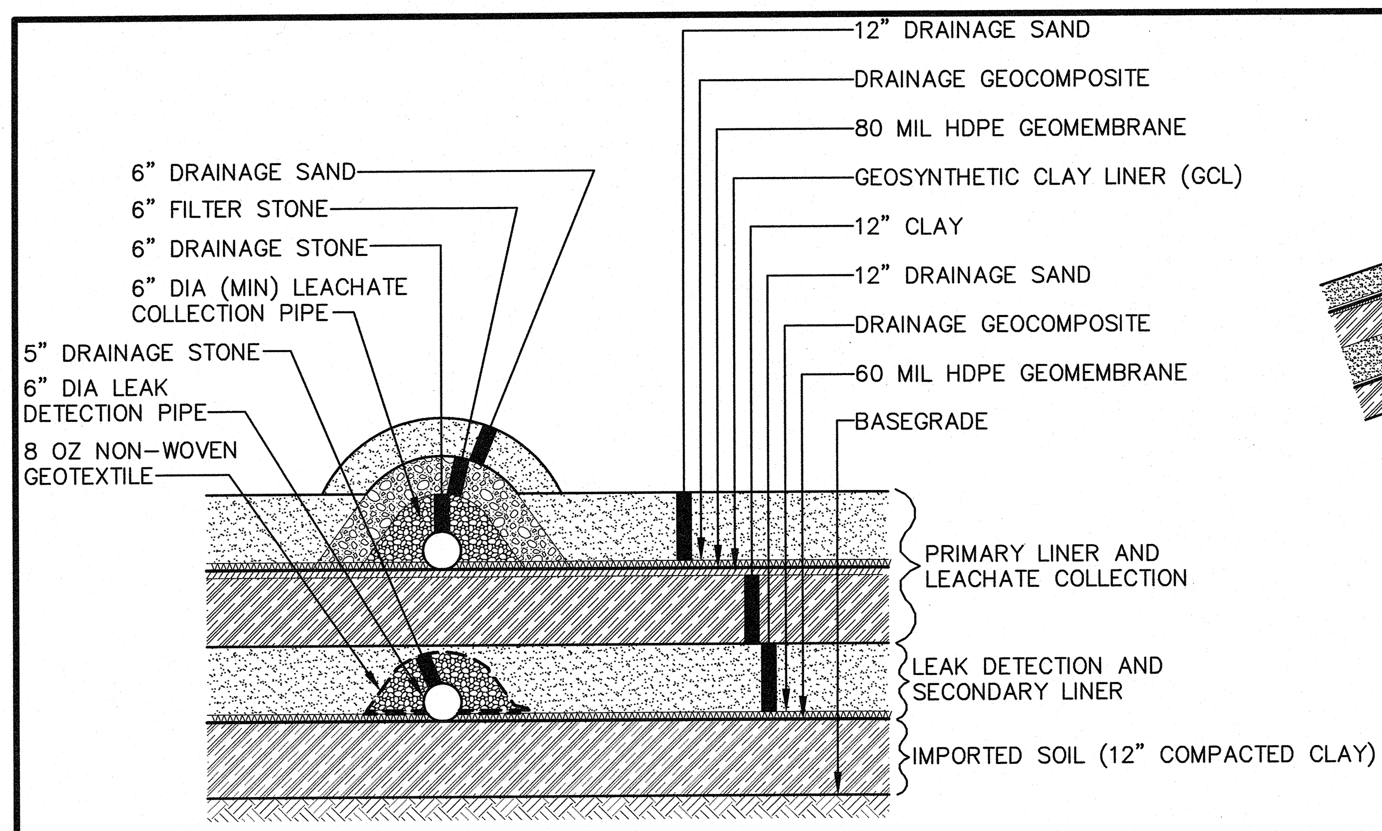
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

TRANSVERSE CROSS SECTIONS
STA. 26+00 TO STA. 28+00

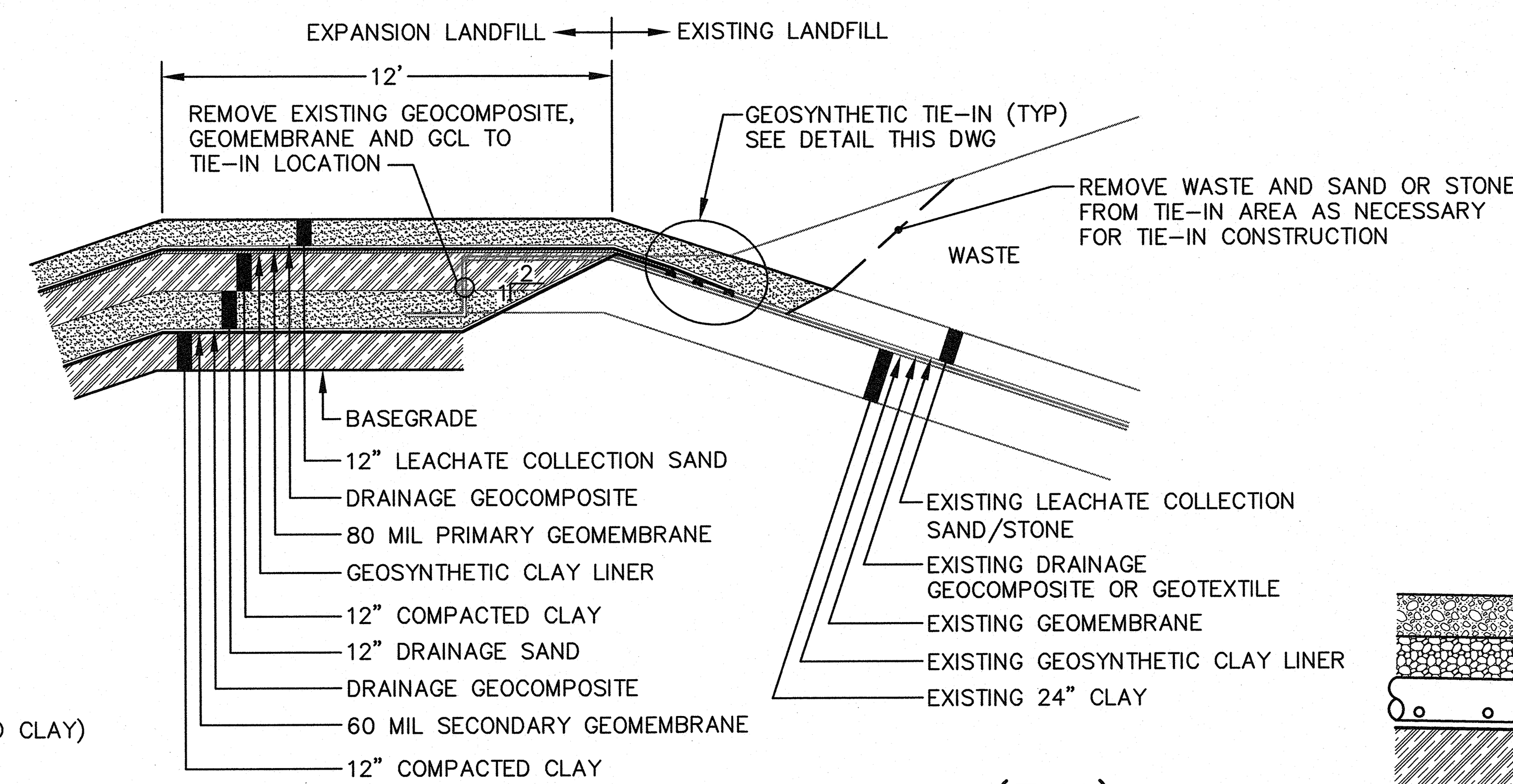
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	5/2015
CHECKED BY:	
LMN:	
CTB:	SME-STD

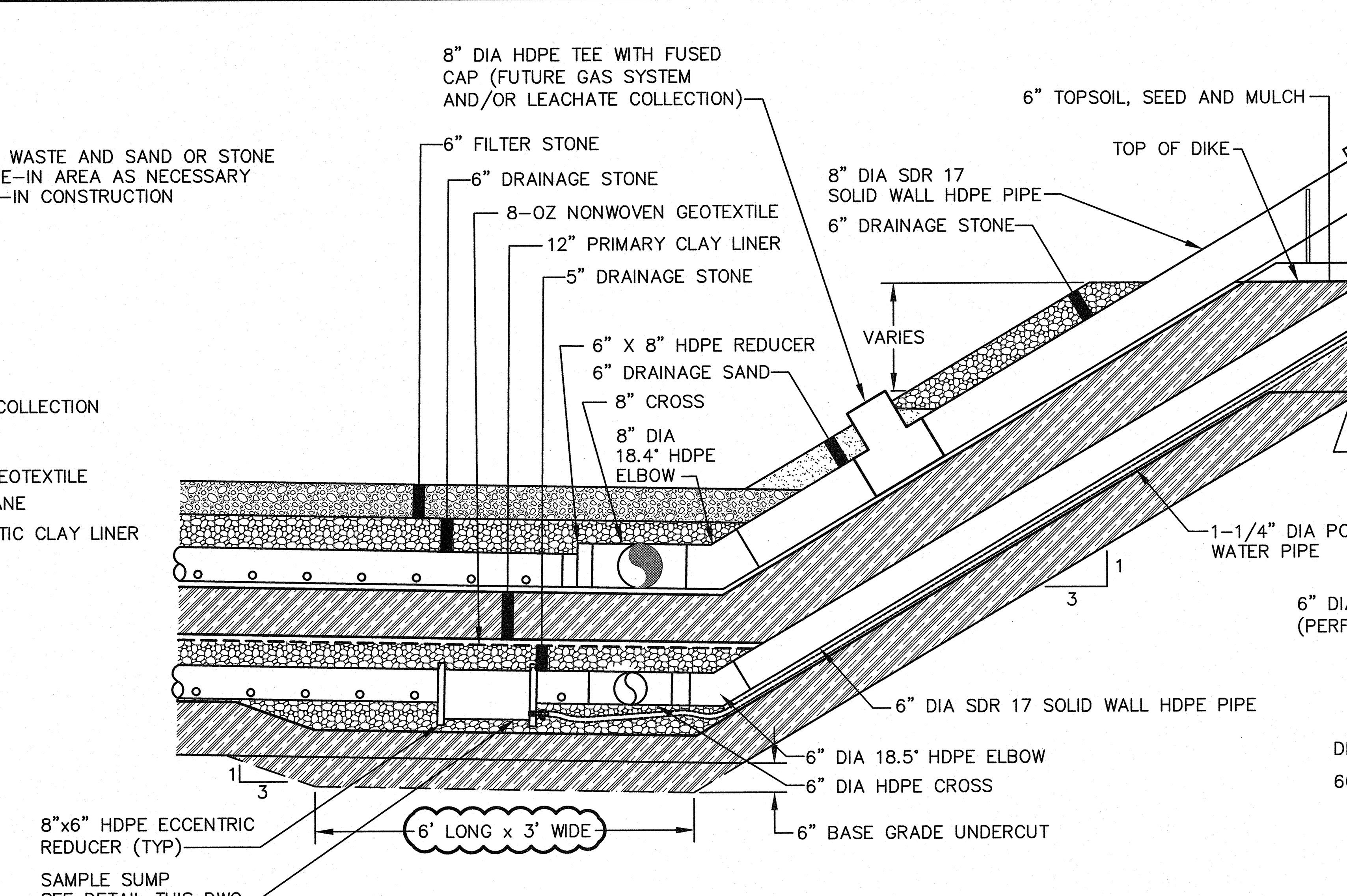
JOB NO. 14101 DWG FILE SECTIONS C-202



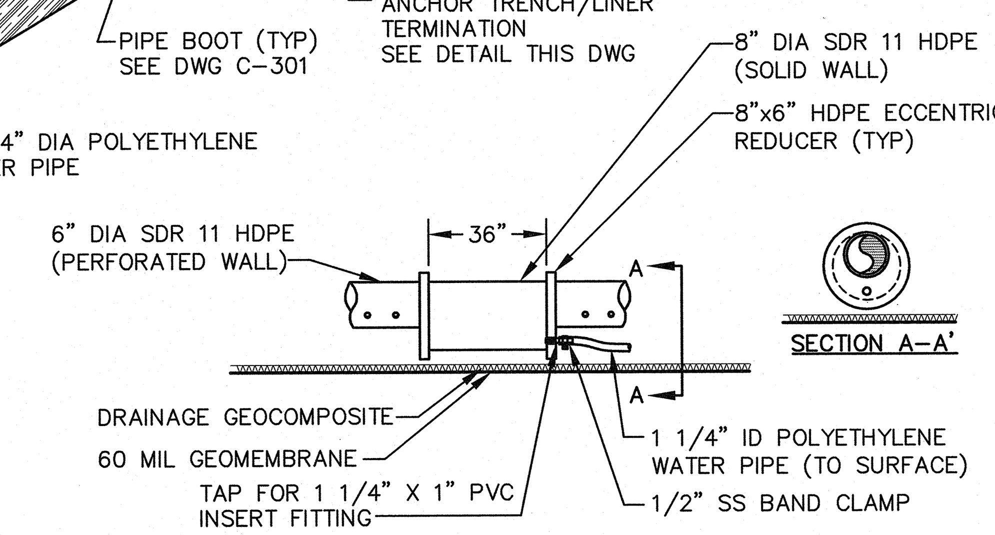
LINER SYSTEM
NTS



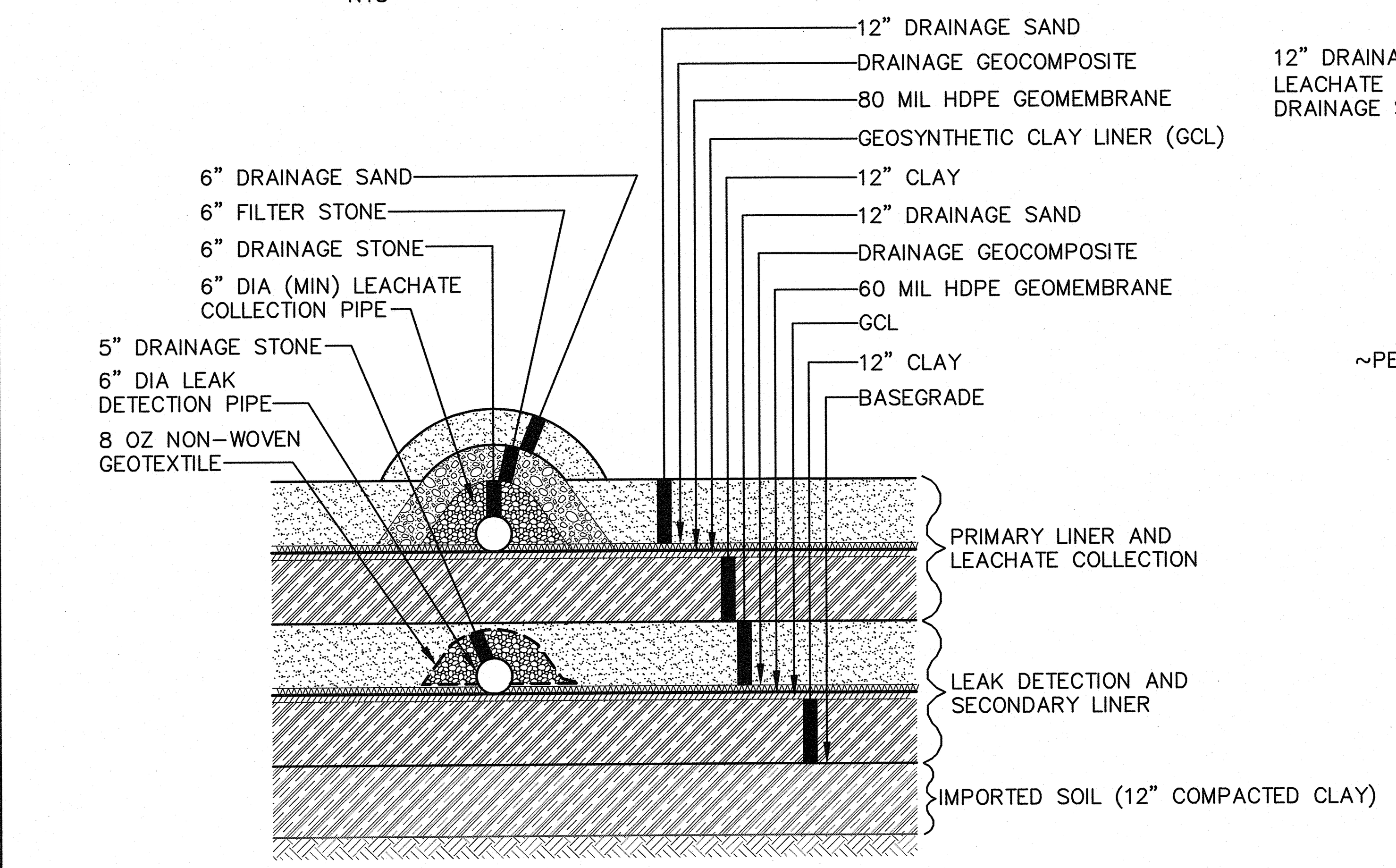
LANDFILL LINER SYSTEM TIE-IN (TYP)
NTS



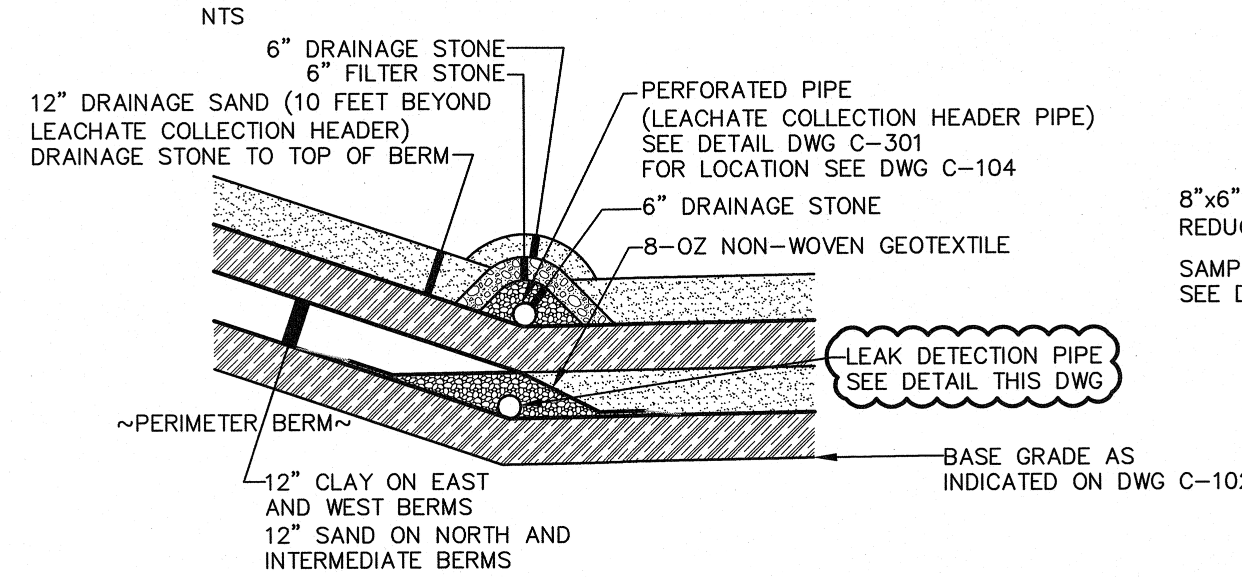
LEACHATE COLLECTION & LEAK DETECTION CLEANOUTS
NTS



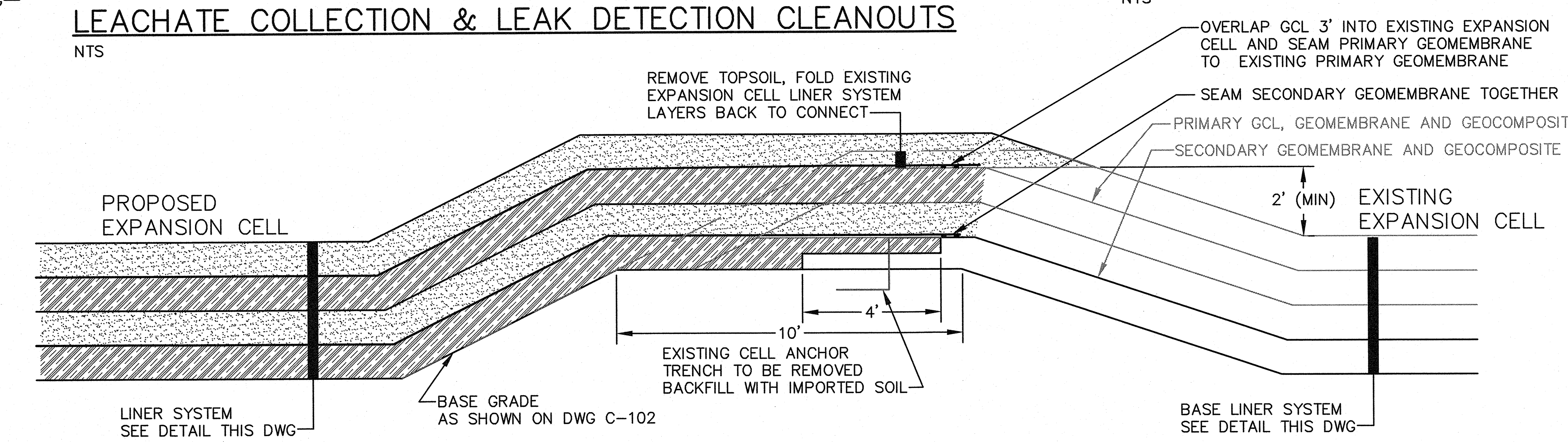
SAMPLE SUMP
NTS



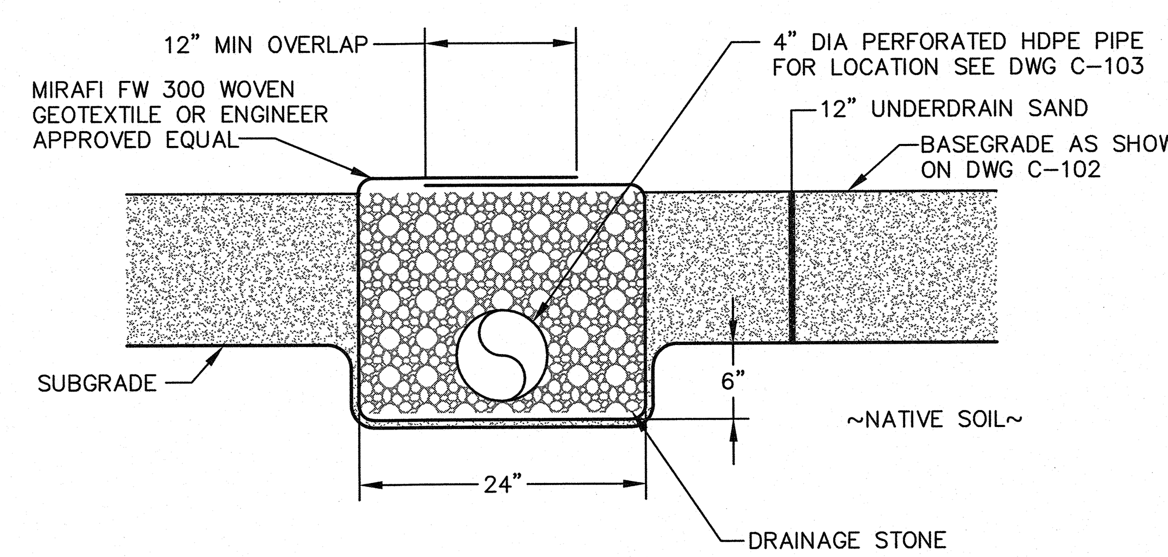
LINER SYSTEM WITH AUGMENTED SECONDARY LINER
NTS



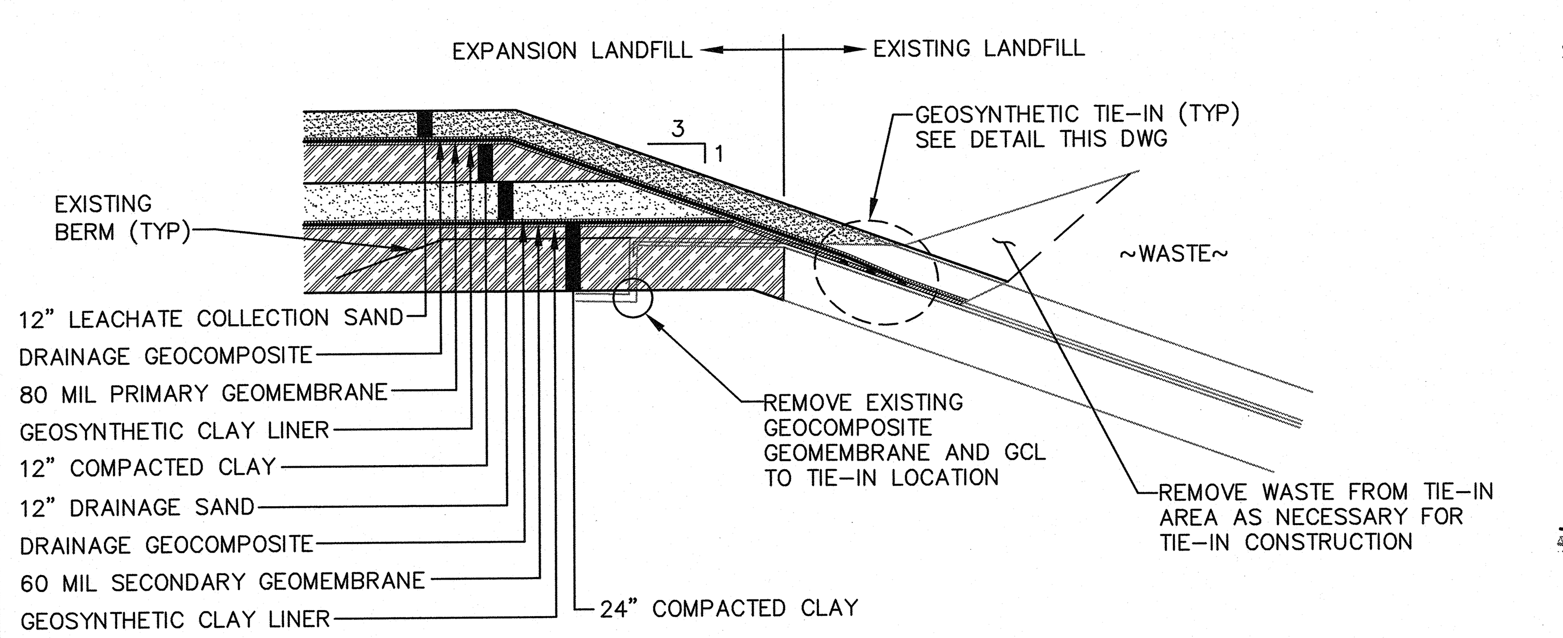
PIPING AT PERIMETER BERM
NTS



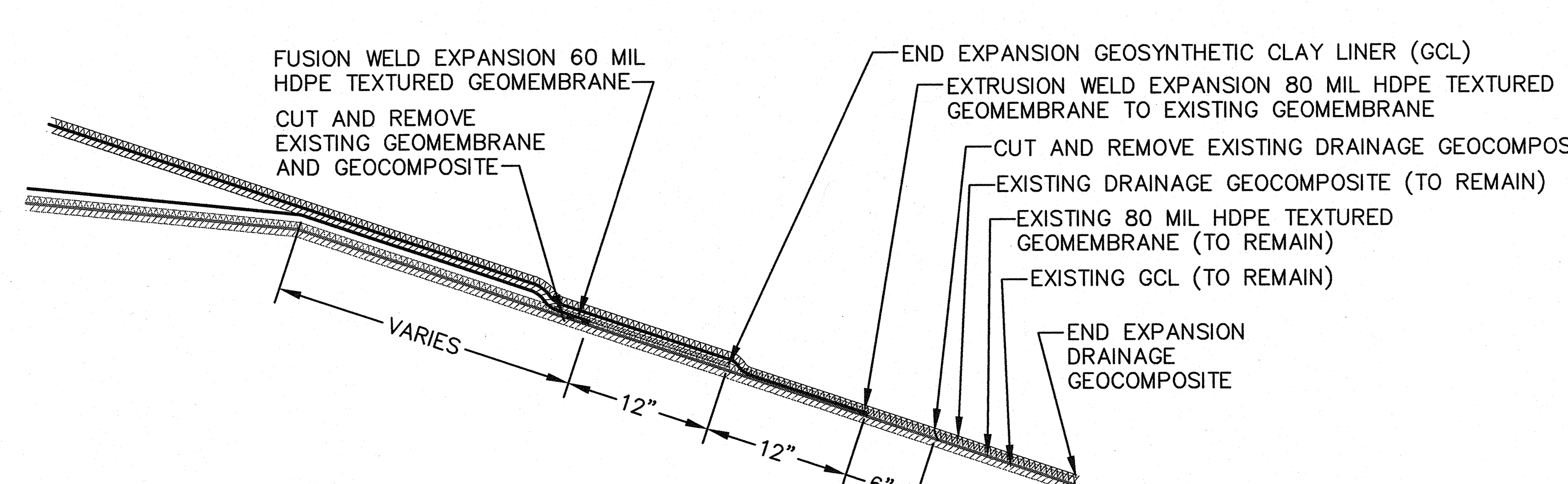
INTERMEDIATE BERM BASE LINER CONNECTION
NTS



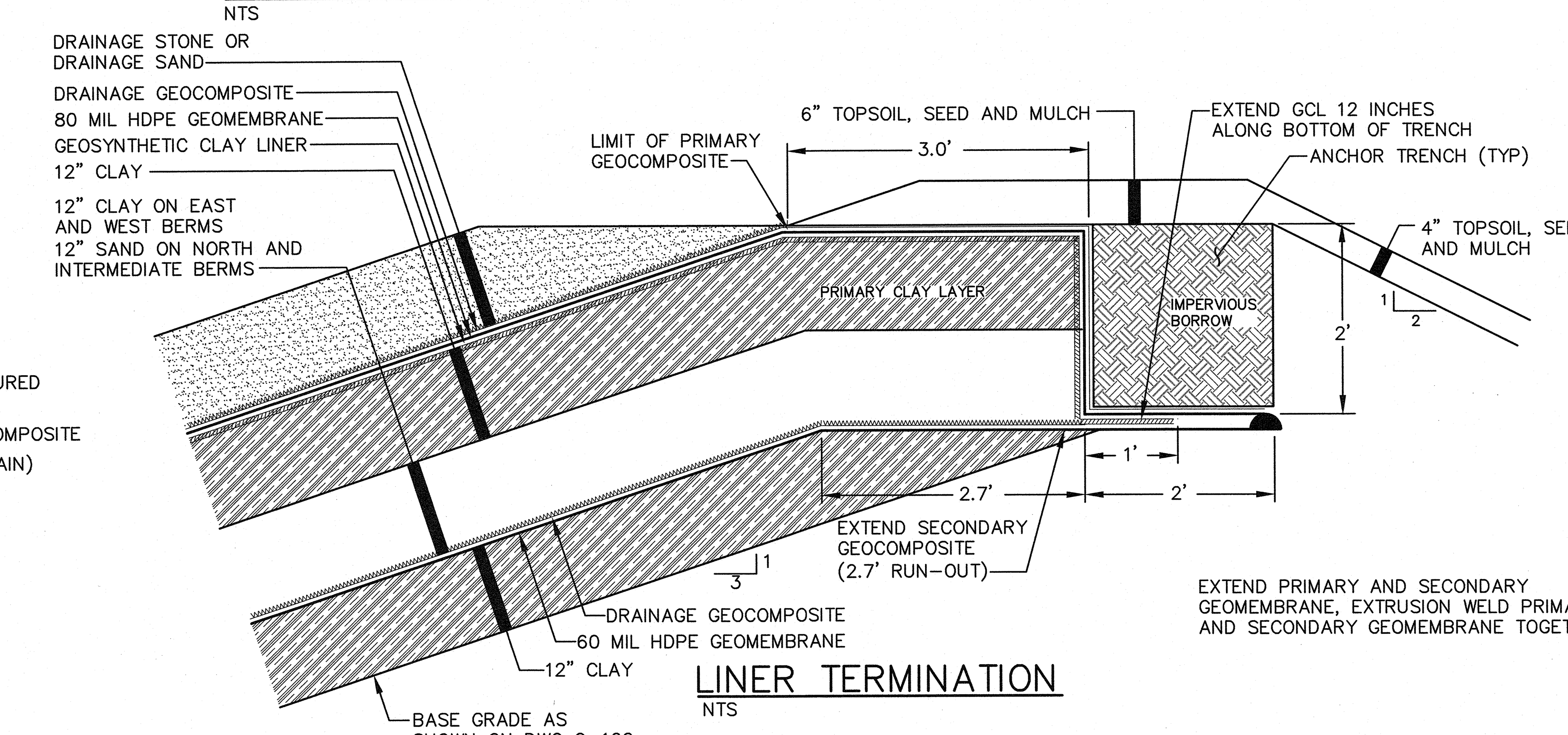
UNDERDRAIN PIPE
NTS



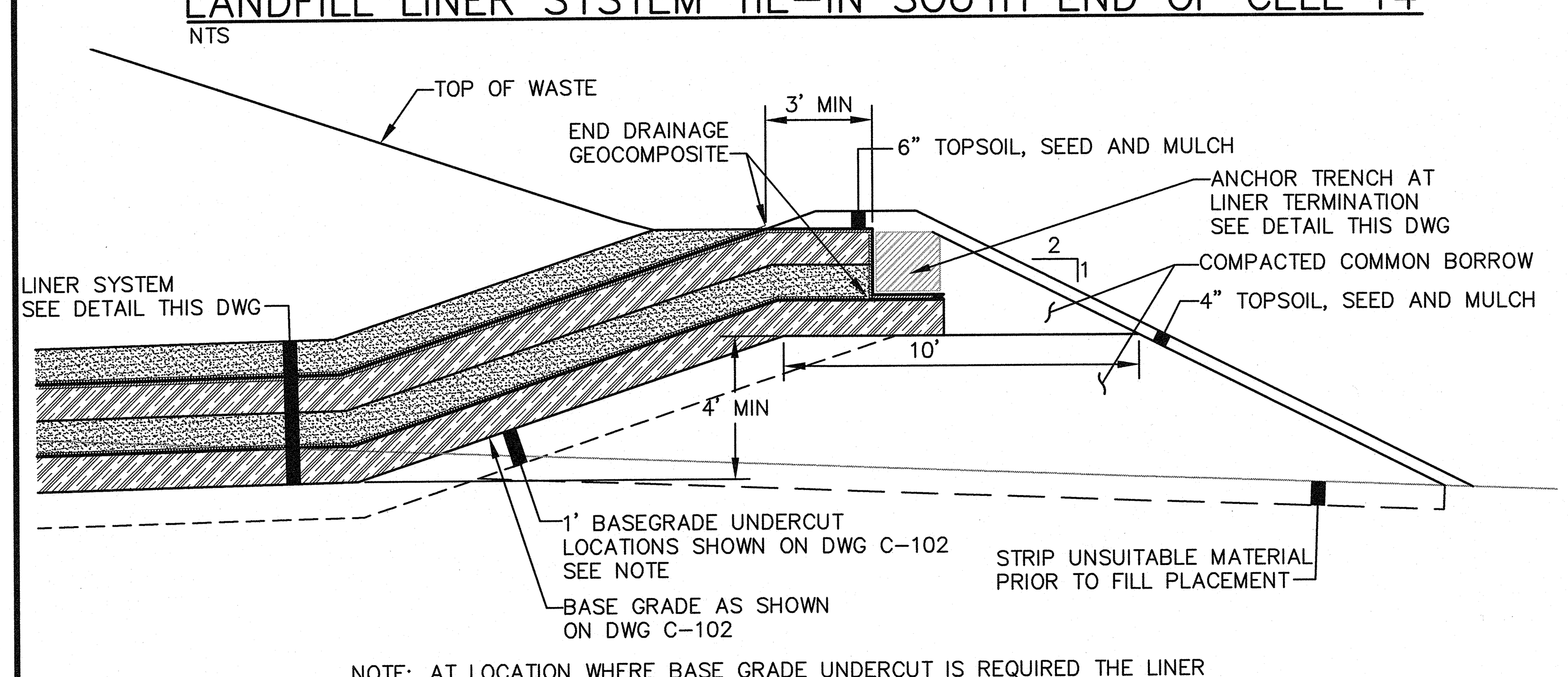
LANDFILL LINER SYSTEM TIE-IN SOUTH END OF CELL 14
NTS



GEOSYNTHETIC TIE-IN
NTS

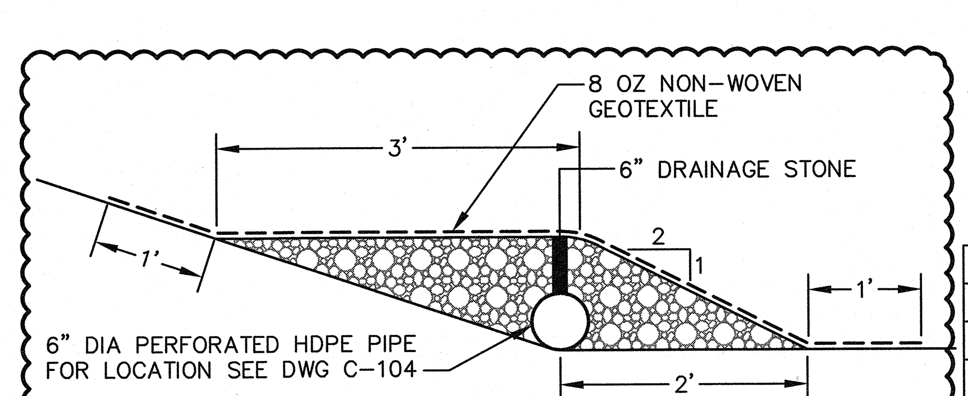


LINER TERMINATION
NTS



NOTE: AT LOCATION WHERE BASE GRADE UNDERCUT IS REQUIRED THE LINER SYSTEM WITH AUGMENTED SECONDARY LINER SHALL BE CONSTRUCTED.

INTERMEDIATE CELL BERM
NTS



LEAK DETECTION PIPE
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE — LICENSED PROFESSIONAL ENGINEER (seal)

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

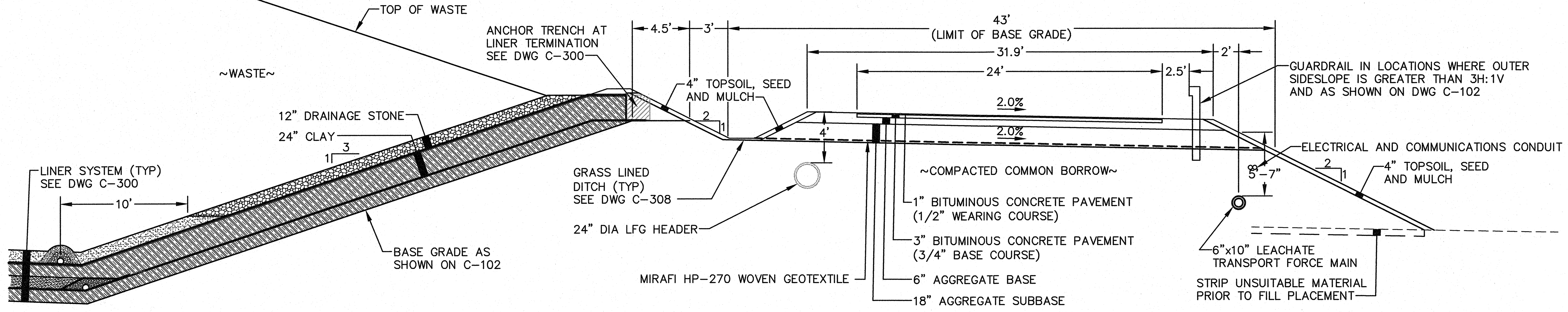
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

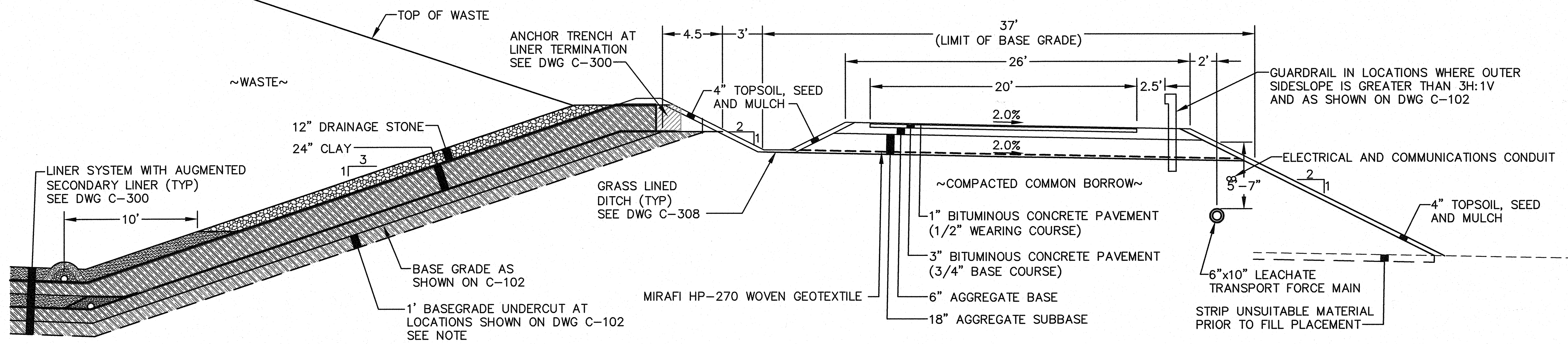
DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

C-300

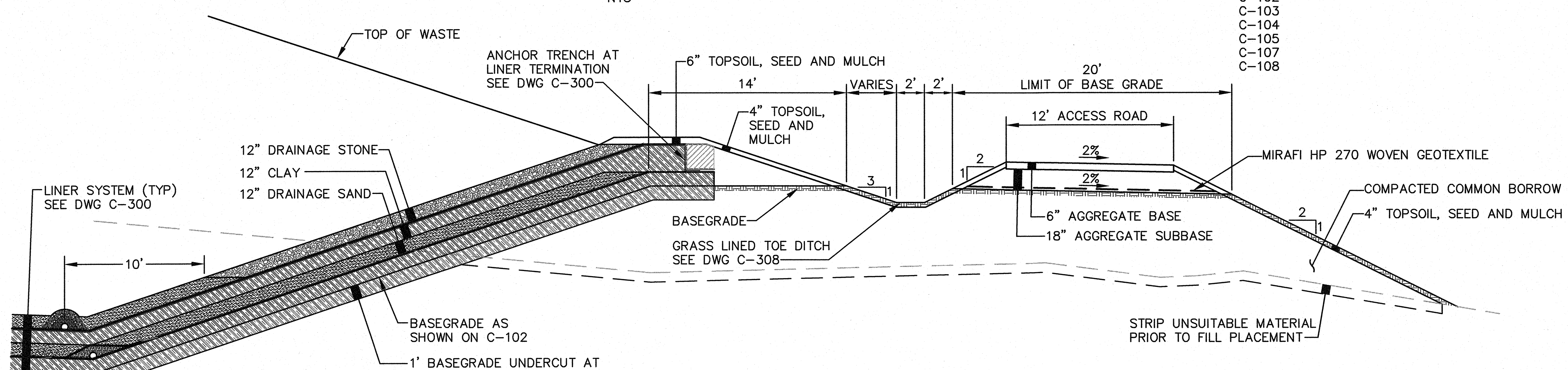


EAST SIDE PERIMETER BERM AND ACCESS ROAD 1
NTS
C-102
C-103
C-104
C-105
C-107
C-108

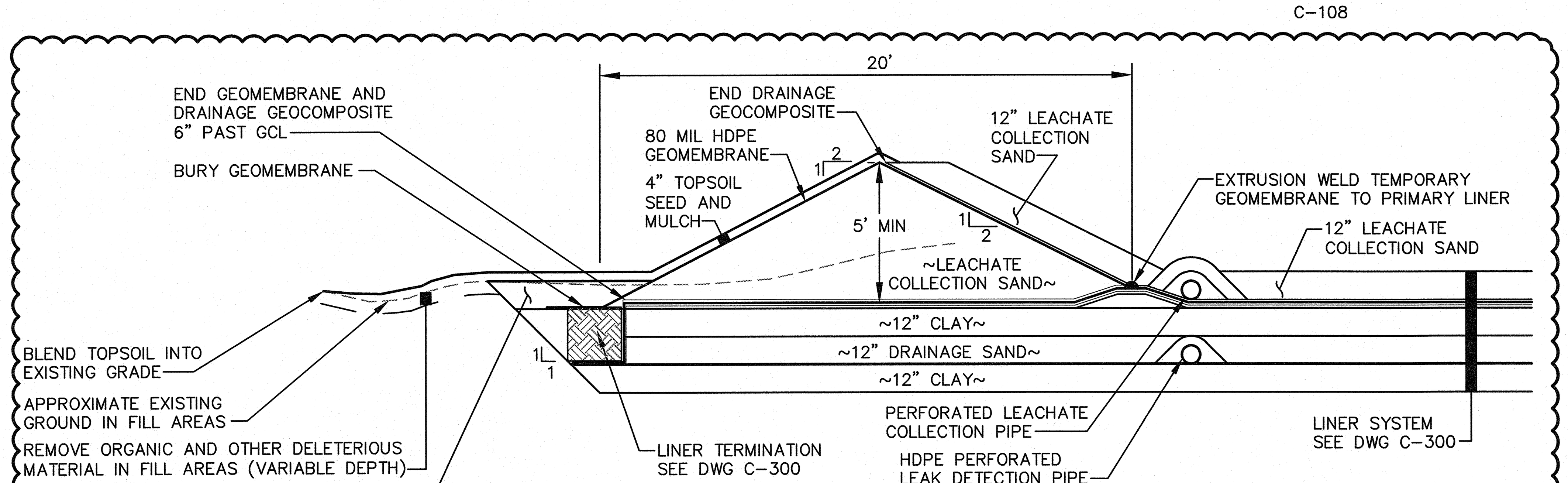


NOTE: AT LOCATION WHERE BASE GRADE UNDERCUT IS REQUIRED THE LINER SYSTEM WITH AUGMENTED SECONDARY LINER SHALL BE CONSTRUCTED.

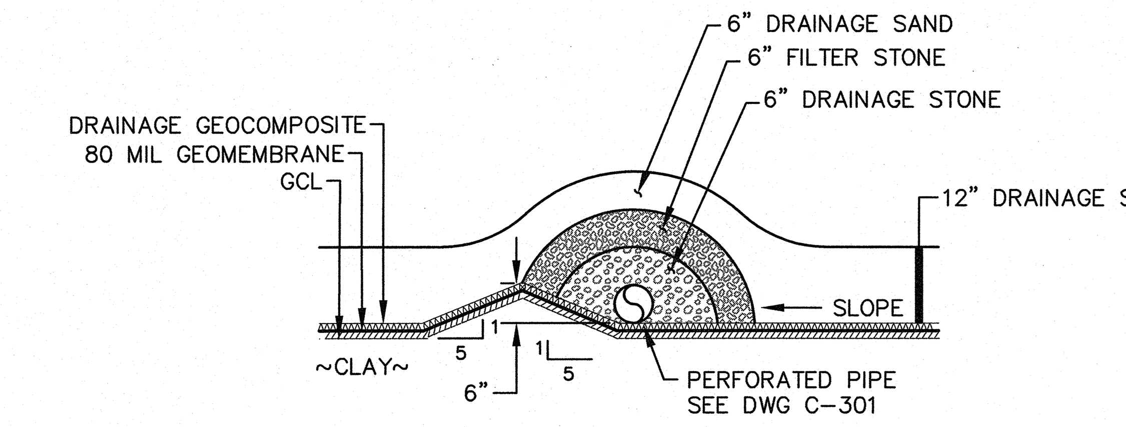
WEST SIDE PERIMETER BERM AND ACCESS ROAD 2
NTS
C-102
C-103
C-104
C-105
C-107
C-108



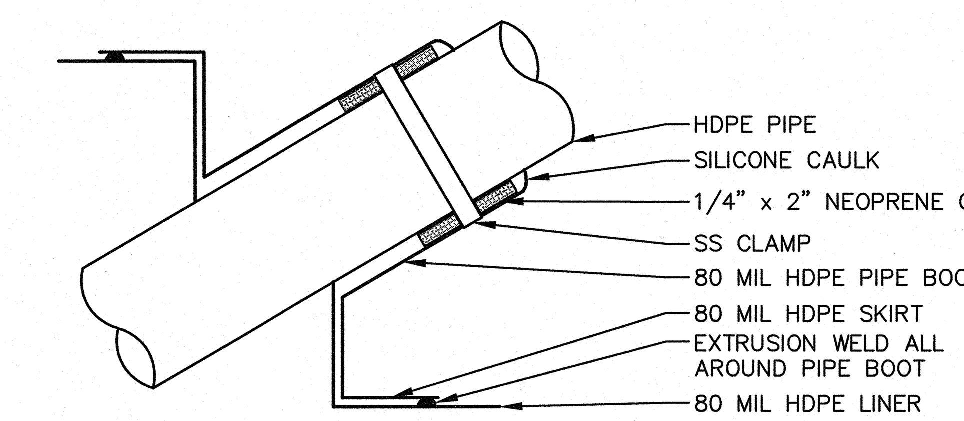
NORTH BERM 3
NTS
C-102
C-103
C-104
C-105
C-107
C-108



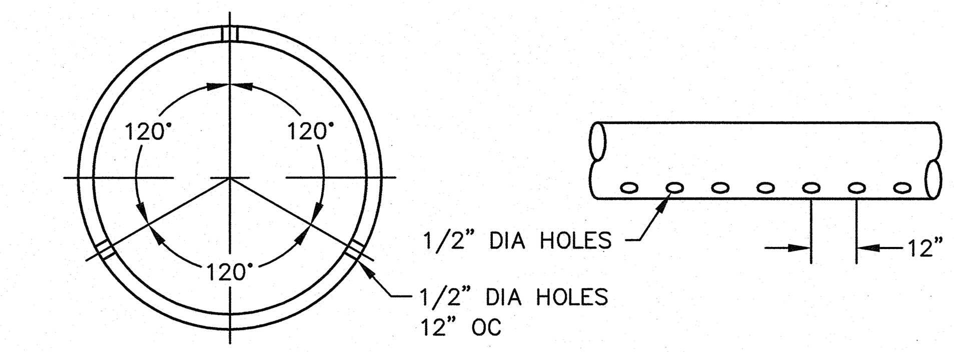
TYPICAL TEMPORARY BERM
NTS



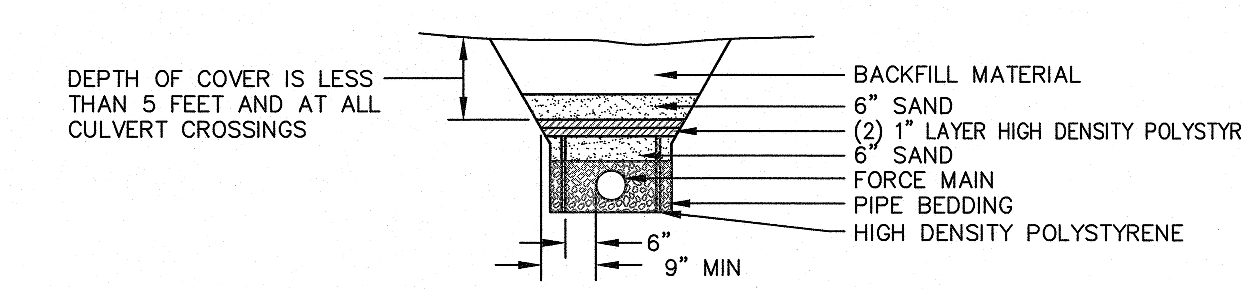
LEACHATE COLLECTION LATERAL (TYP)
NTS



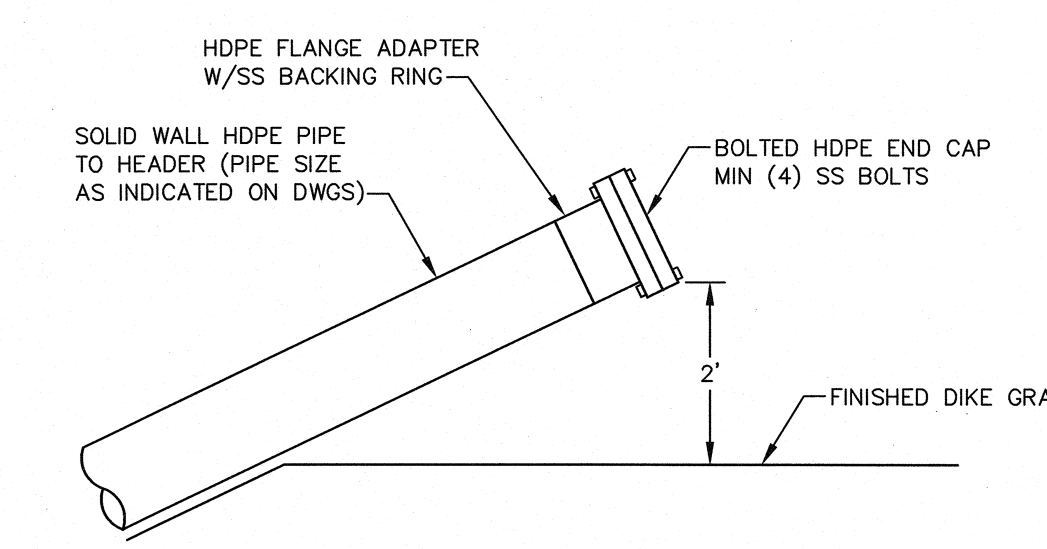
PIPE BOOT
NTS



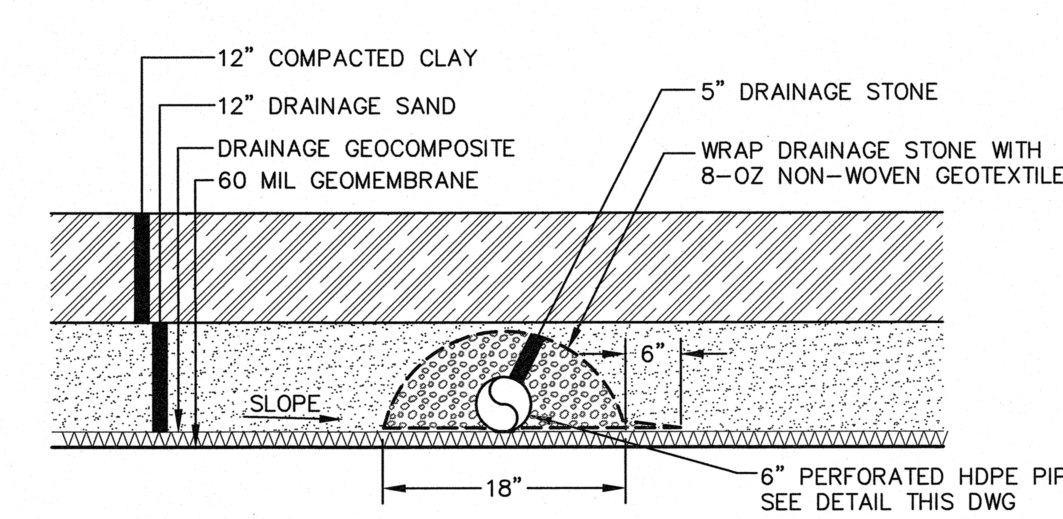
PERFORATED PIPE
NTS



PIPE INSULATION
NTS



FLANGED END CAP
NTS



LEAK DETECTION LATERAL (TYP)
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE ... LICENSED PROFESSIONAL ENGINEER 2/19/16

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

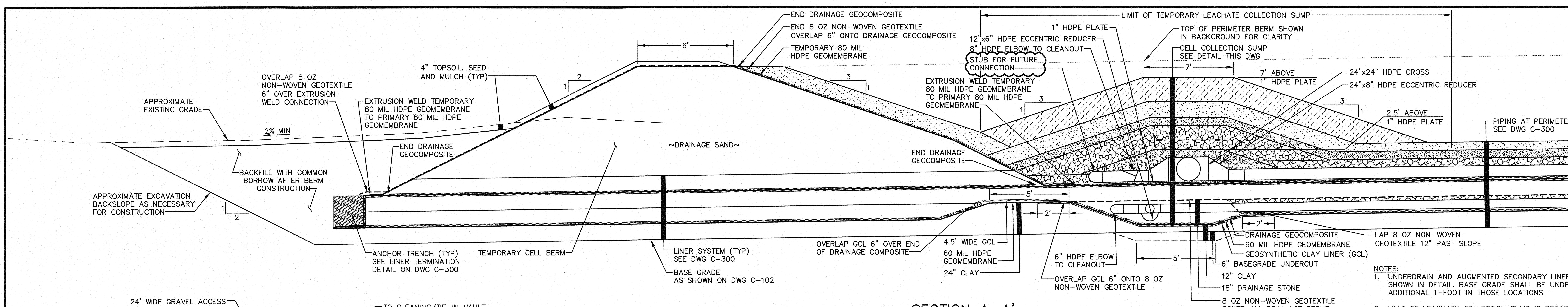
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

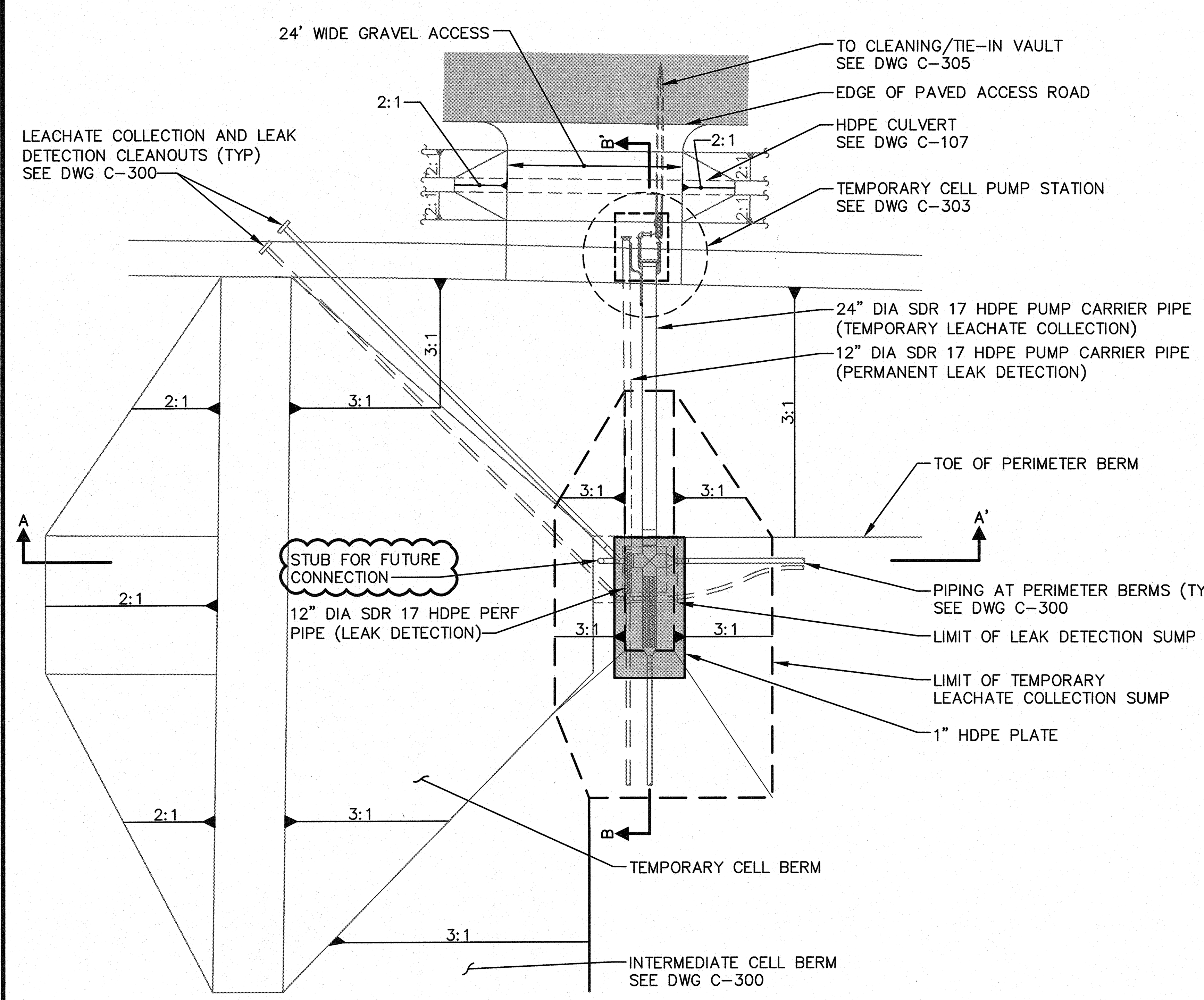
JOB NO. 14101.00 DWG FILE DETAILS

C-301

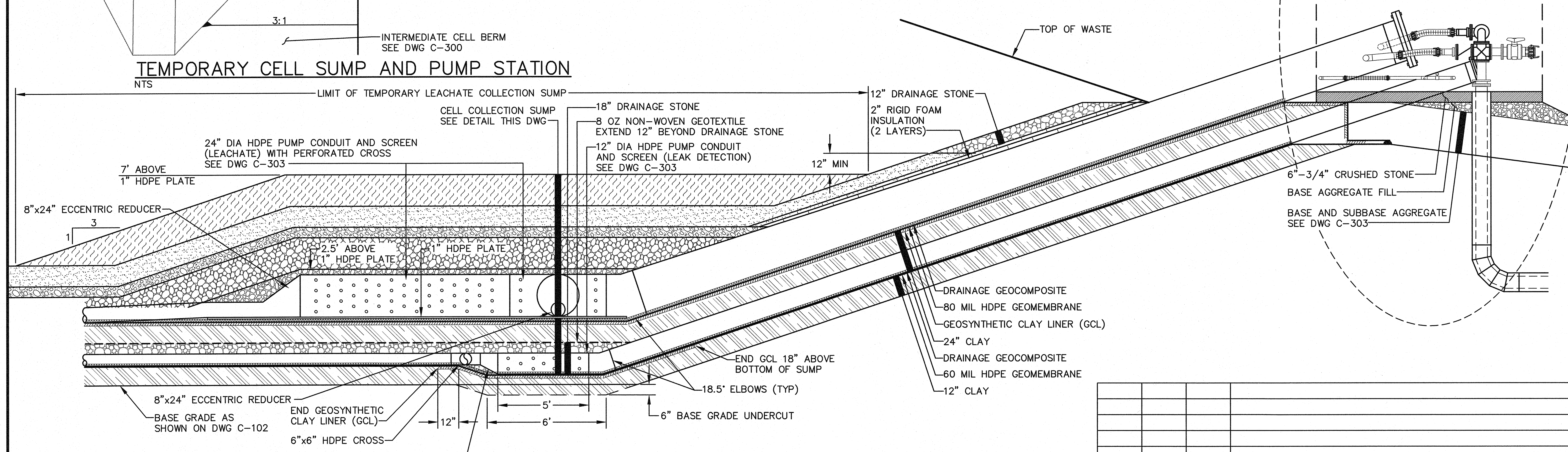


SECTION A–A'
NTS

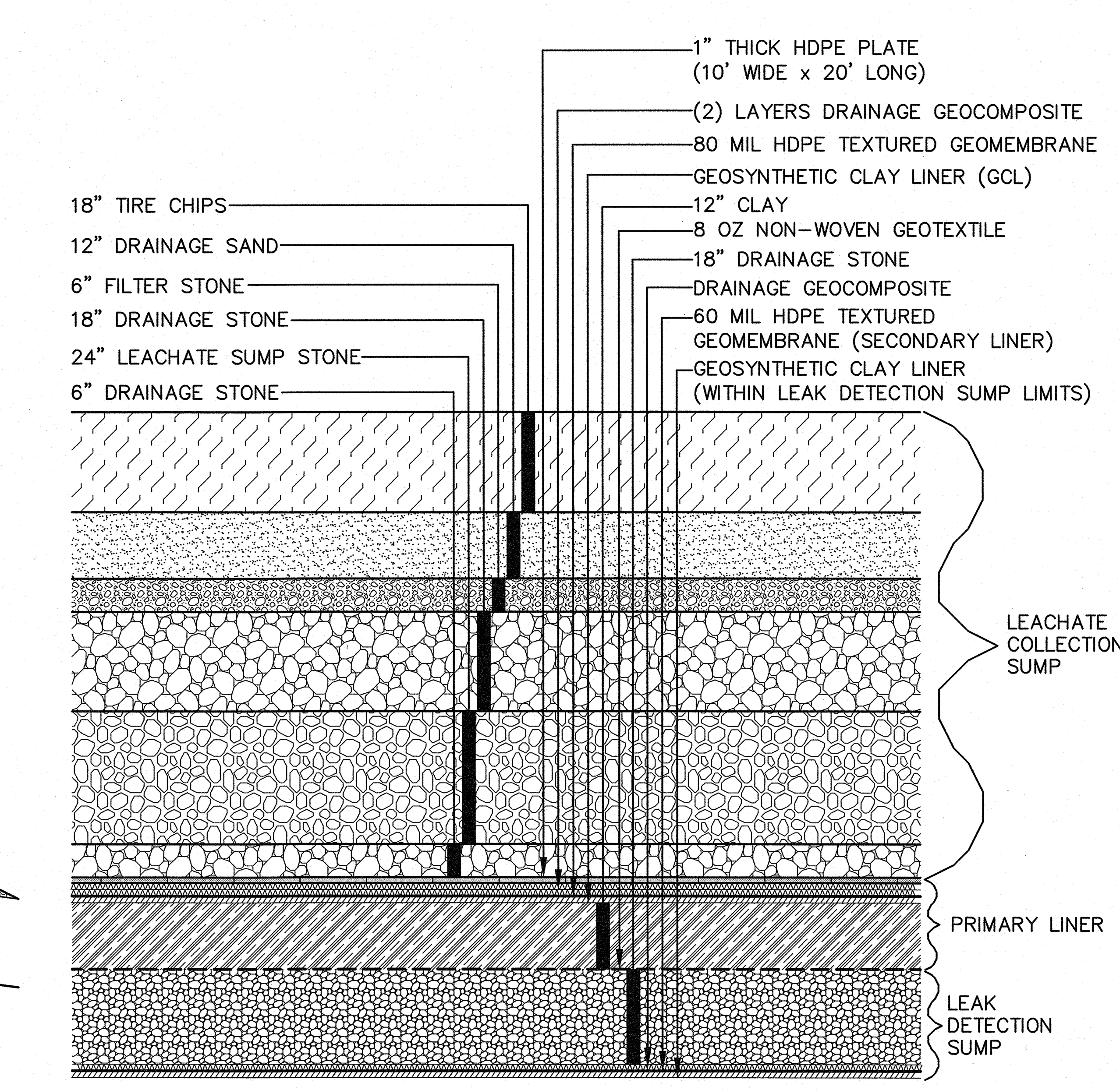
NOTES:
1. UNDERDRAIN AND AUGMENTED SECONDARY LINER ARE NOT SHOWN IN DETAIL. BASE GRADE SHALL BE UNDERCUT AN ADDITIONAL 1-FOOT IN THOSE LOCATIONS
2. LIMIT OF LEACHATE COLLECTION SUMP IS DEFINED BY THE LIMIT OF THE 18" TIRE CHIPS LAYER



TEMPORARY CELL SUMP AND PUMP STATION
NTS



SECTION B–B'
NTS



CELL COLLECTION SUMP
NTS

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SECTIONS AND DETAILS

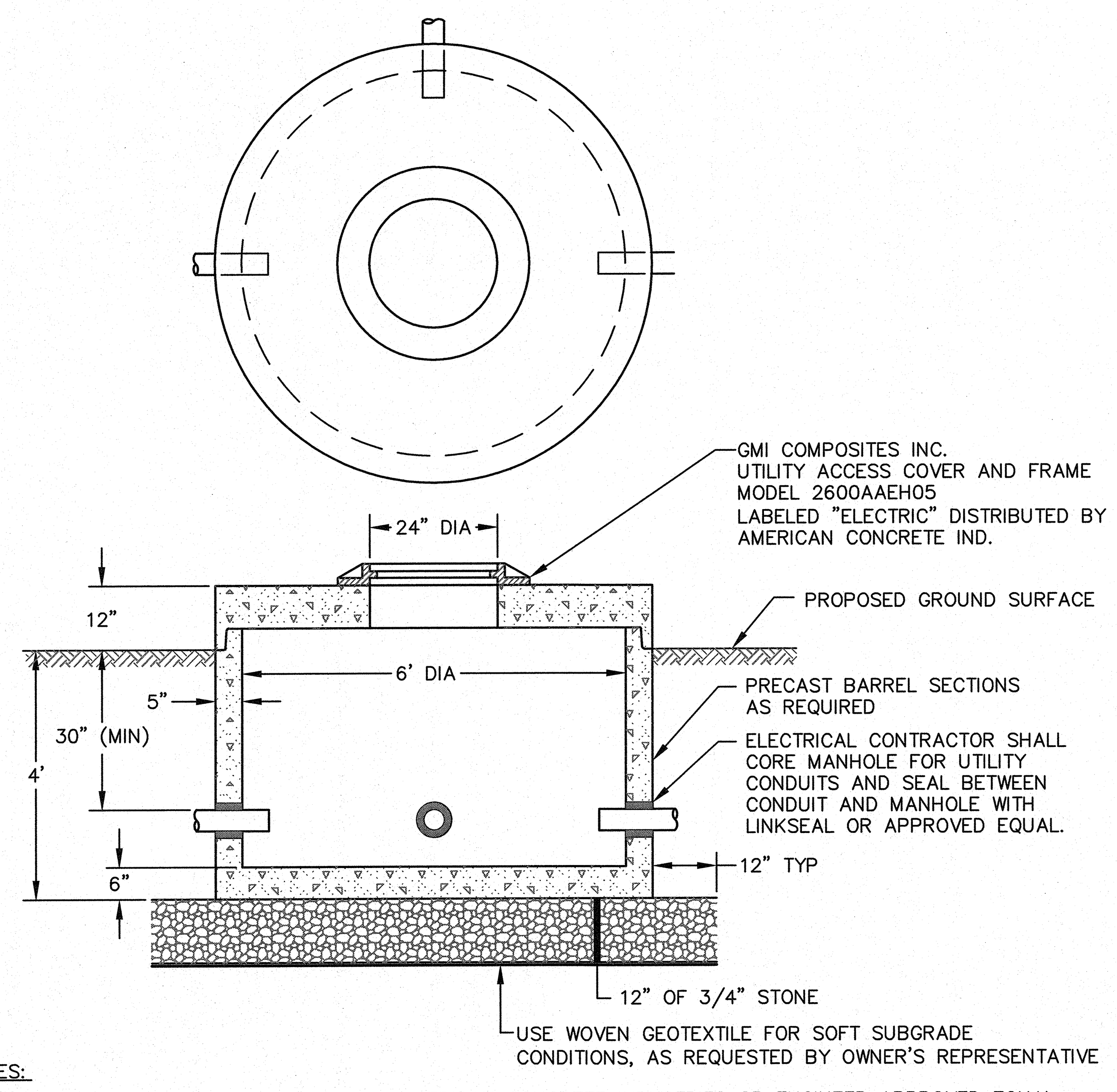
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	12/5/2014
CHECKED BY:	
LMN:	NONE
CTB:	SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

C-302

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

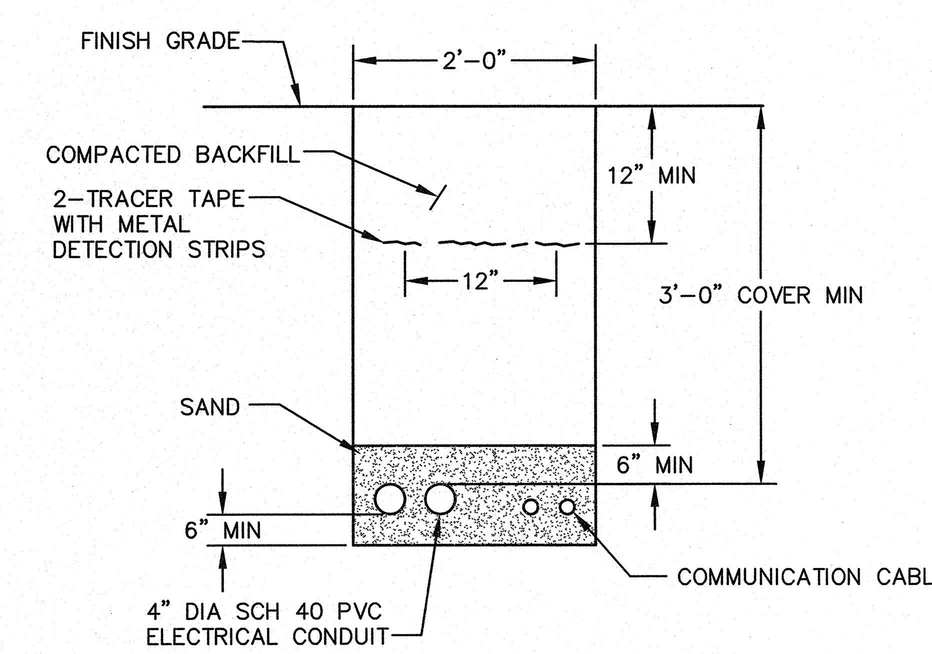


NOTES:

1. 6' DIA MANHOLE AS MANUFACTURED BY AMERICAN CONCRETE INDUSTRIES OR ENGINEER APPROVED EQUAL.
2. 4000 PSI CONCRETE AT 28 DAYS.
3. DESIGNED FOR H-20 WHEEL LOADING.
4. CONFORMS TO ASTM C-478 SPECIFICATIONS.
5. REINFORCED TO 0.12 IN SQ/LF.
6. SHIPLAP JOINTS SEALED WITH BUTYL RUBBER.
7. EXTERIOR COATED WITH ASPHALTIC PROTECTIVE DAMPROOFING.
8. BOTTOM MIN 5'-0" BELOW FINISH GRADE.
9. PRECAST CONCRETE VAULT MANUFACTURER TO PROVIDE ANTI-FLOATATION EXTENDED BASE SLAB AS NECESSARY. ANTI-FLOATATION DESIGN AND SHOP DRAWINGS SHALL BE PREPARED BY THE MANUFACTURER AND SUBMITTED TO THE ENGINEER FOR APPROVAL.

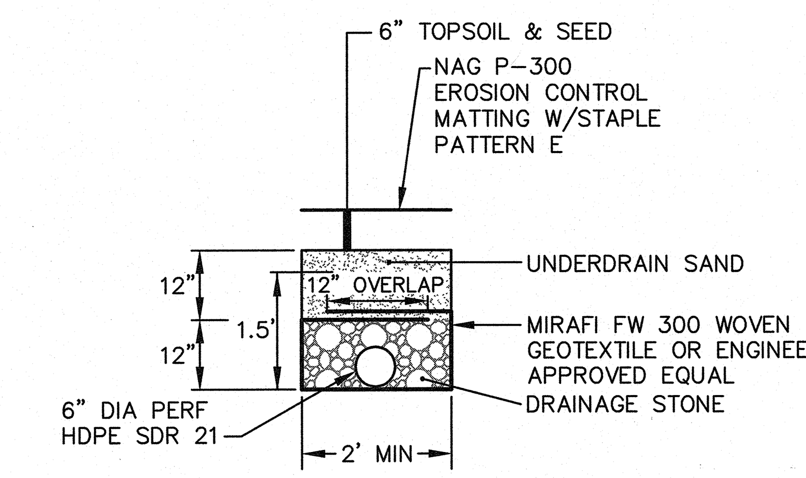
ELECTRIC UTILITY MANHOLE

NTS



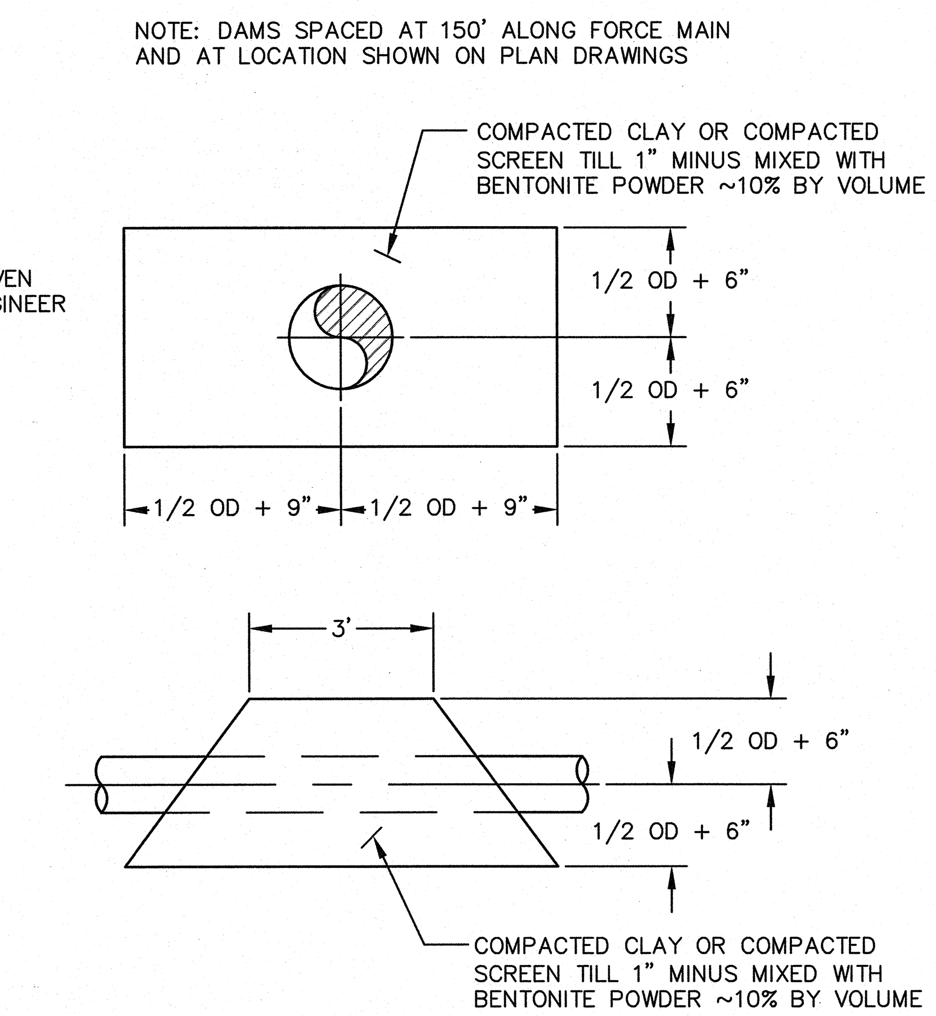
UNDERGROUND ELECTRIC AND COMMUNICATION TRENCH

NTS



POND UNDERDRAIN

NTS



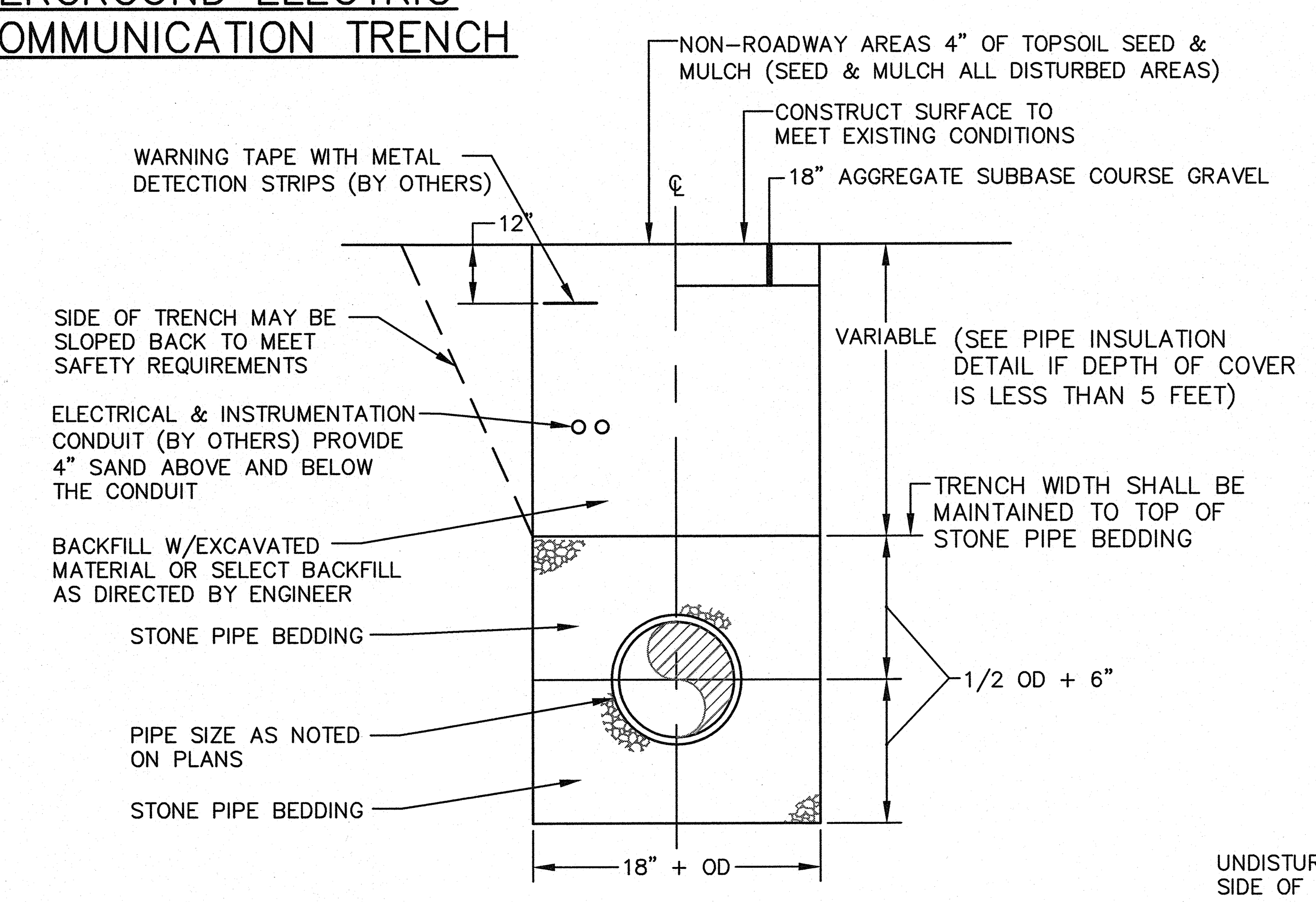
PIPE DAM

NTS

CULVERTS	DIAMETER (IN)	LENGTH (FT)	SLOPE (FT/FT)	INV IN (FT)	INV OUT (FT)
C-2BA	36	40	0.08	203.2	202.9
C-2BB	24	96	0.01	195.0	194.0
C-4BA	24	78	0.01	204.4	203.7
C-4BB	24	78	0.01	204.4	203.7
C-4F	18	78	0.04	165.0	162.0
C-4G	24	36	0.03	175.0	174.0
C-4HA	18	40	0.03	201.9	200.9
C-4HB	18	101	0.03	178.5	176.0
C-4I	18	80	0.13	202.5	192.0
C-4IA	18	40	0.02	212.9	212.2
C-4JA	18	60	0.03	214.0	212.3
C-4JB	24	73	0.02	211.5	210.0
C-4JC	24	73	0.02	211.5	210.0
C-4K	24	51	0.04	216.5	214.3
C-4L	18	121	0.02	213.0	211.0
C-4N	18	33	0.03	184.0	183.0

CULVERT SCHEDULE

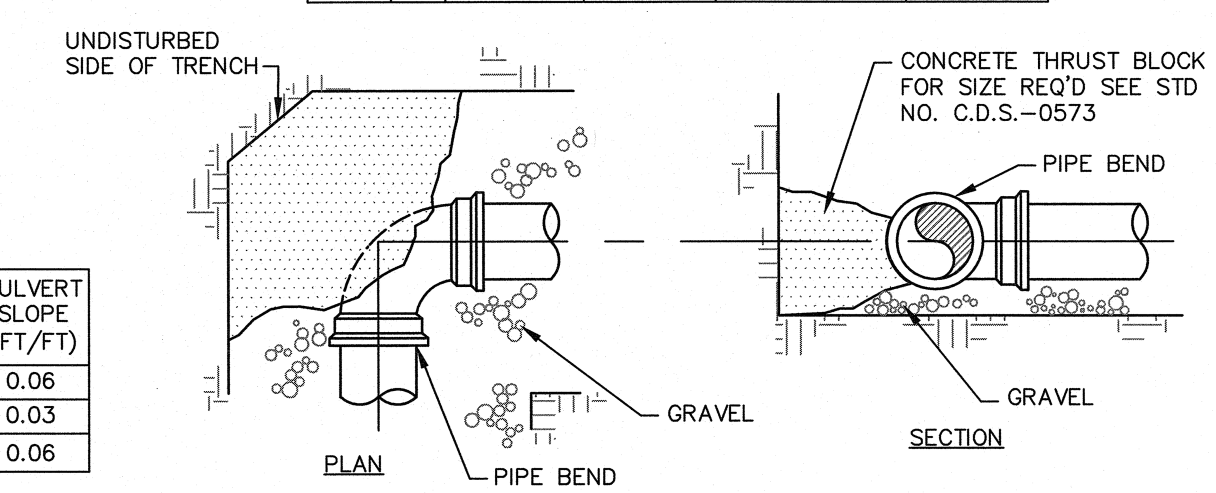
NTS



PIPE TRENCH

NTS

SIZE REQUIREMENTS FOR CONCRETE THRUST BLOCKS					
		BEARING ON UNDISTURBED SOIL (SQ FT)			
FITTINGS		90° BENDS	45° BENDS	TEES & PLUGS	HYDRANTS
PIPE SIZE	4"	2.0	1.0	1.0	N/A
	6"	3.0	2.0	2.0	6.0
	8"	5.0	3.0	4.0	N/A
	10"	7.0	4.0	5.0	N/A
	12"	10.0	6.0	7.0	N/A
	14"	13.0	7.0	10.0	N/A
	16"	17.0	9.0	12.0	N/A

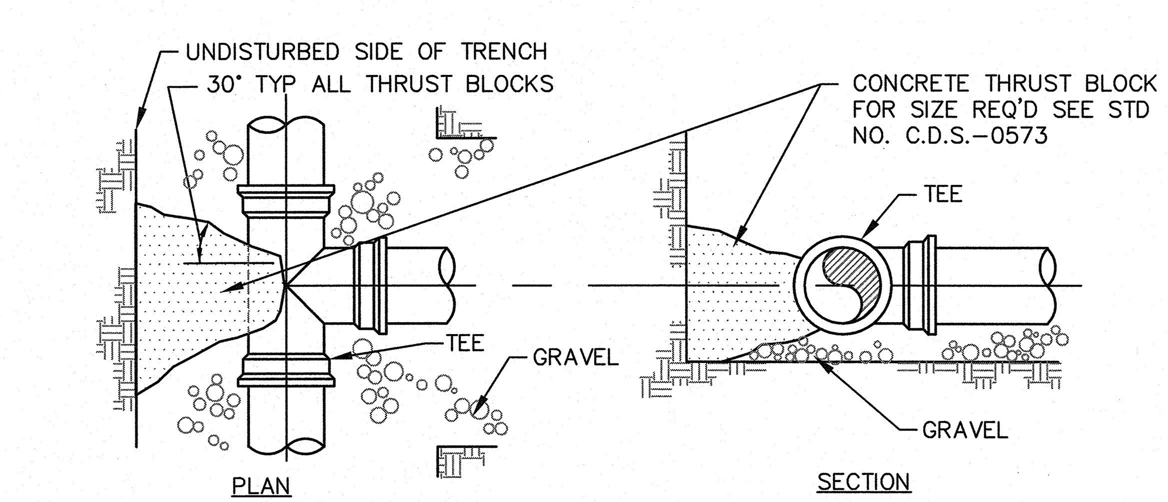


TYPICAL THRUST BLOCK PLACEMENT ON BENDS

NTS

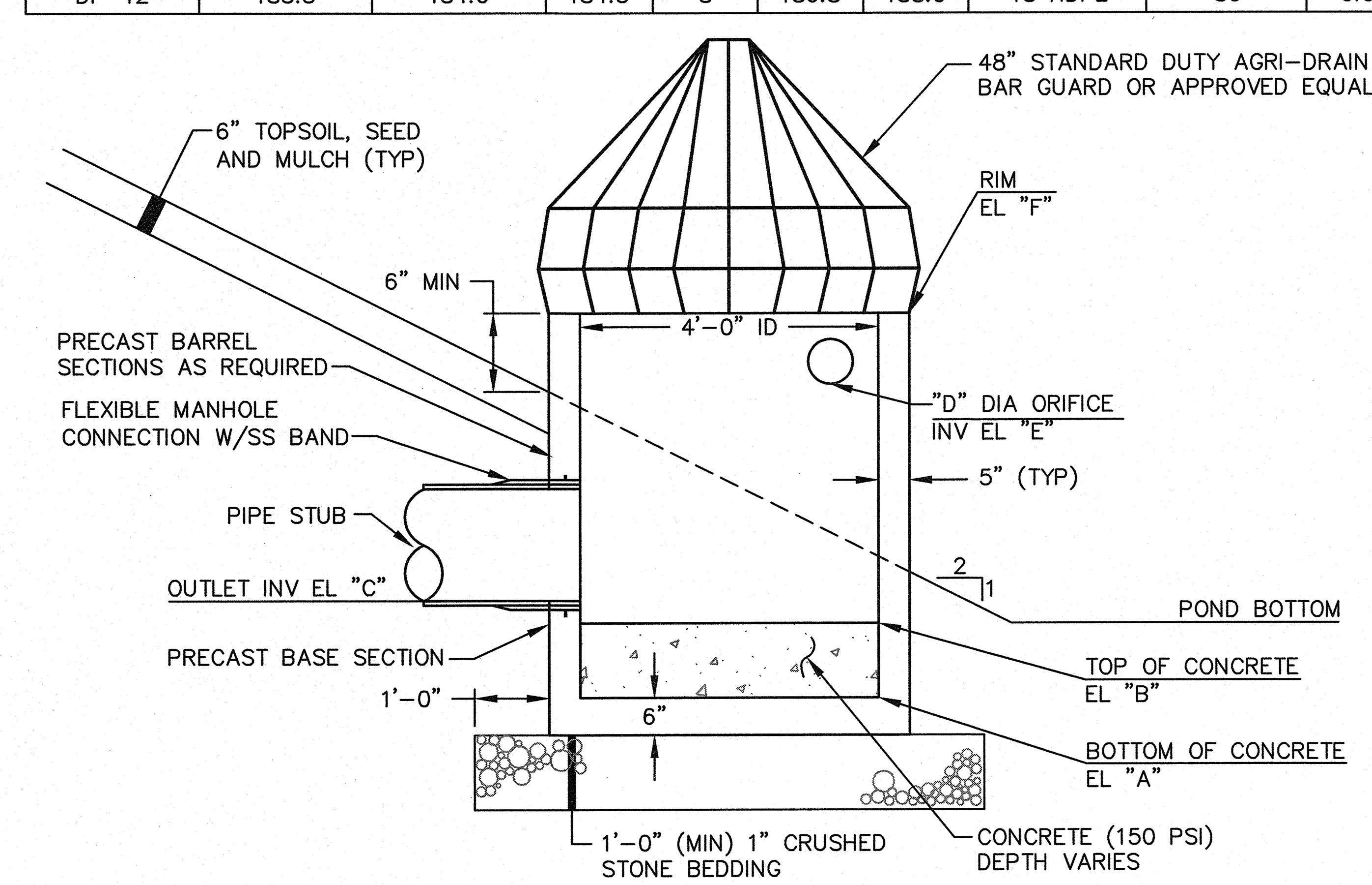
OUTLET CONTROL STRUCTURE SCHEDULE

STRUCTURE DESIGNATION	BOTTOM OF CONCRETE EL "A" (FT)	TOP OF CONCRETE EL "B" (FT)	OUTLET INV EL "C" (FT)	ORIFICE DIA "D" (IN)	ORIFICE INV EL "E" (FT)	RIM EL "F" (FT)	OUTLET (C) CULVERT DIAMETER (IN)	CULVERT LENGTH (FT)	CULVERT SLOPE (FT/FT)
DP-10	174.2	174.7	175.2	6	178.0	179.0	18 HDPE	52	0.06
DP-11	163.5	164.0	164.3	6	167.5	168.4	18 HDPE	92	0.03
DP-12	183.5	184.0	184.5	8	186.8	188.0	18 HDPE	80	0.06



TYPICAL THRUST BLOCK PLACEMENT ON TEES

NTS



OUTLET CONTROL STRUCTURE

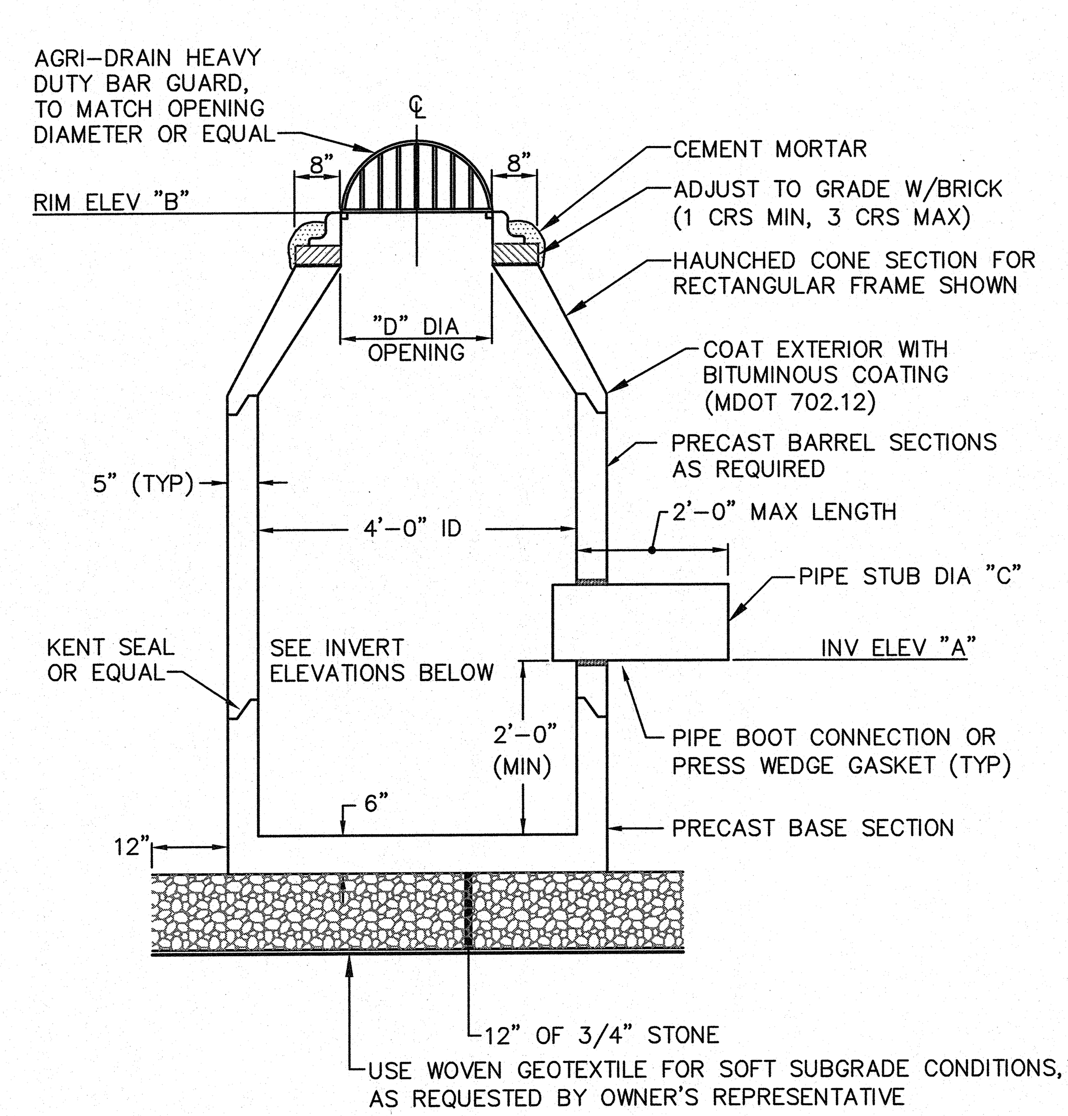
NTS

CATCH BASIN SCHEDULE A

CATCH BASIN DESIGNATION	PIPE INV EL "A" (FT)	RIM EL "B" (FT)	PIPE DIA "C" (IN)	TOP OPENING DIA "D" (IN)
CB-2BB	195.0	200.2	24"	30"
CB-4G	175.0	181.0	24"	24"
CB-4HB	178.5	183.4	18"	24"
CB-4I	202.5	207.6	18"	24"
CB-4JA	214.0	218.7	18"	24"

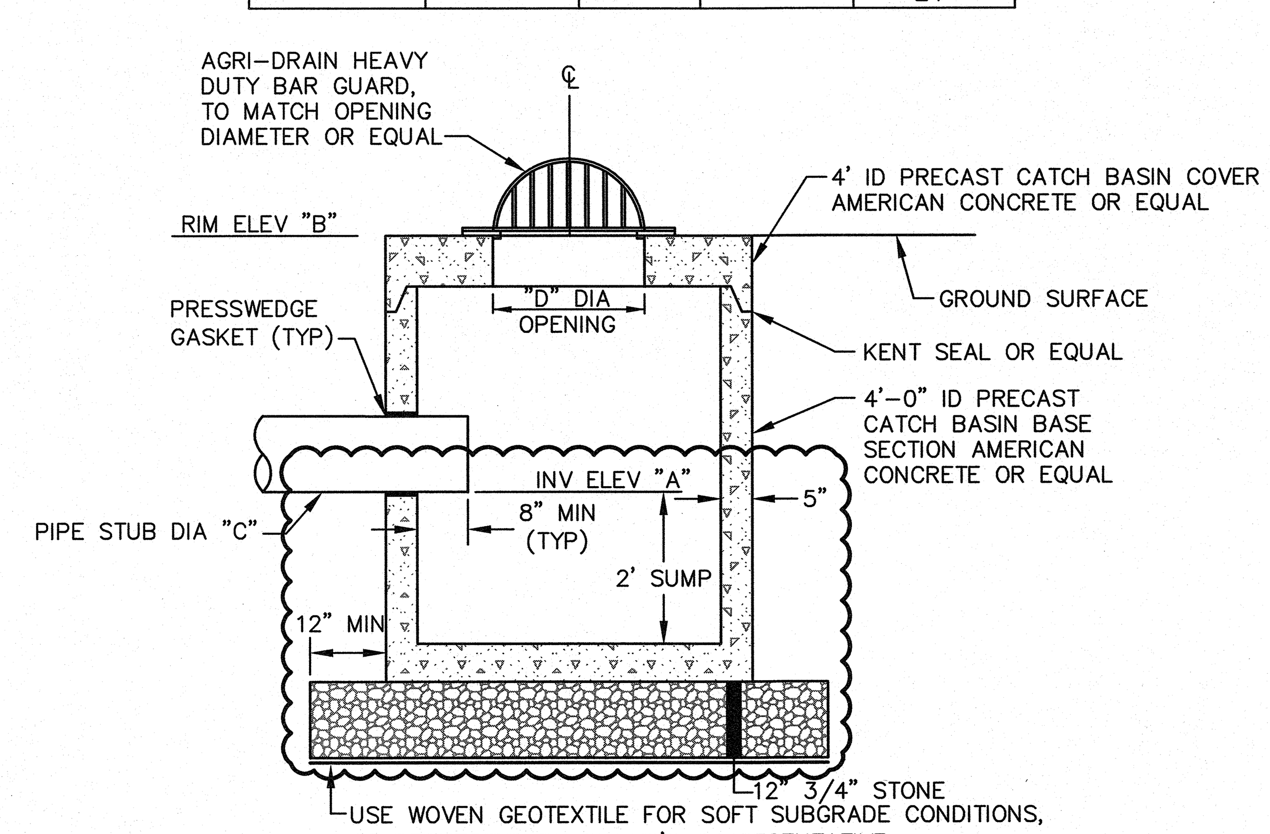
CATCH BASIN SCHEDULE B

CATCH BASIN DESIGNATION	PIPE INV EL "A" (FT)	RIM EL "B" (FT)	PIPE DIA "C" (IN)	TOP OPENING DIA "D" (IN)
CB-4K	216.5	220.0	24"	30"
CB-4L	213.0	215.0	18"	24"



CATCH BASINS 2BB, 4G, 4HB, 4I, & 4JA

NTS



CATCH BASINS 4K & 4L

NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

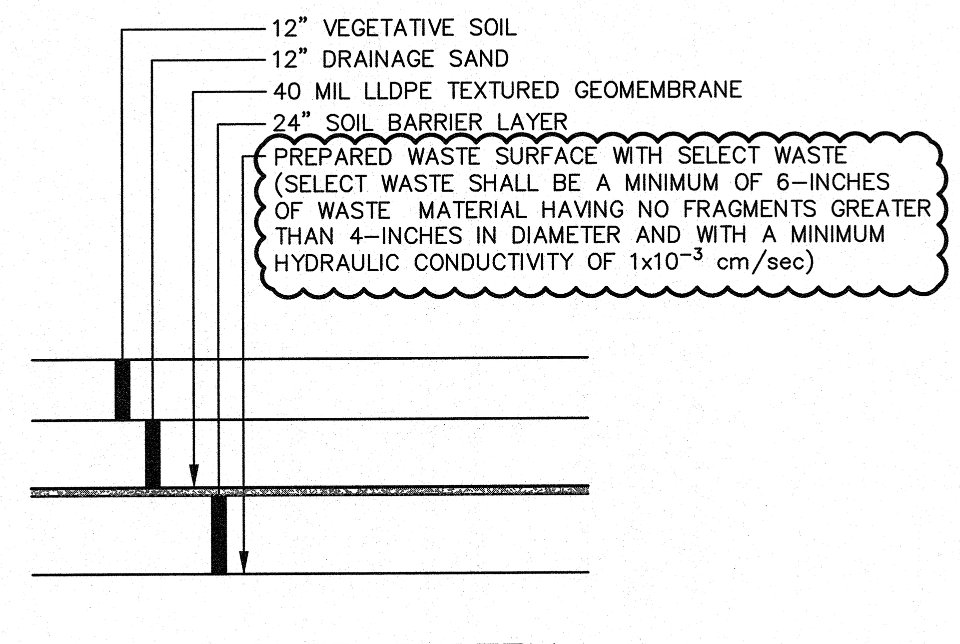
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

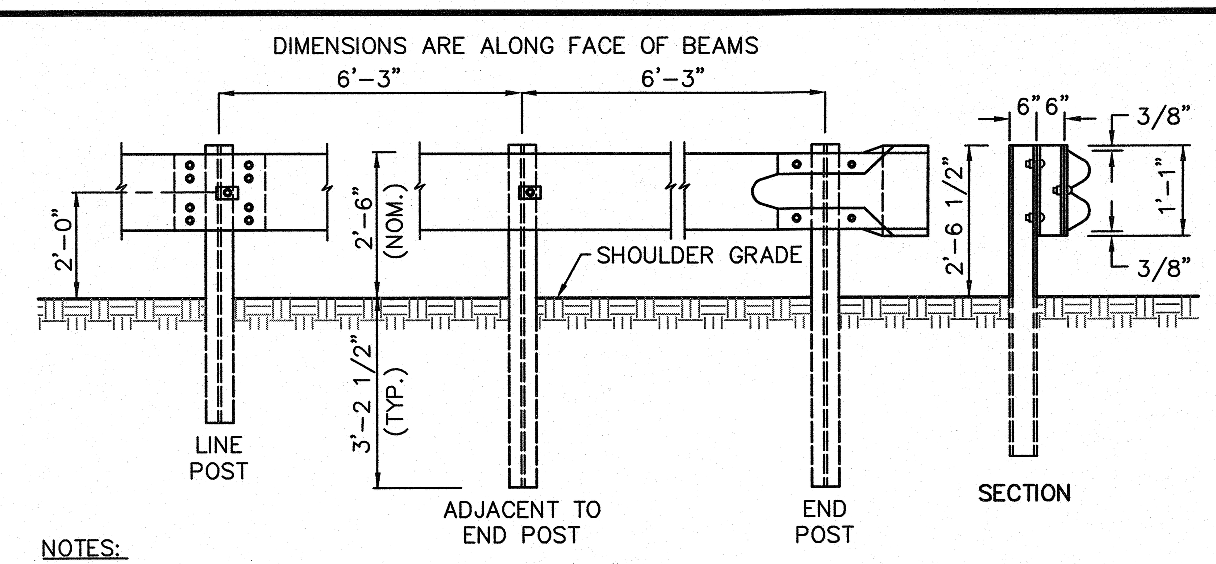
DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

C-306



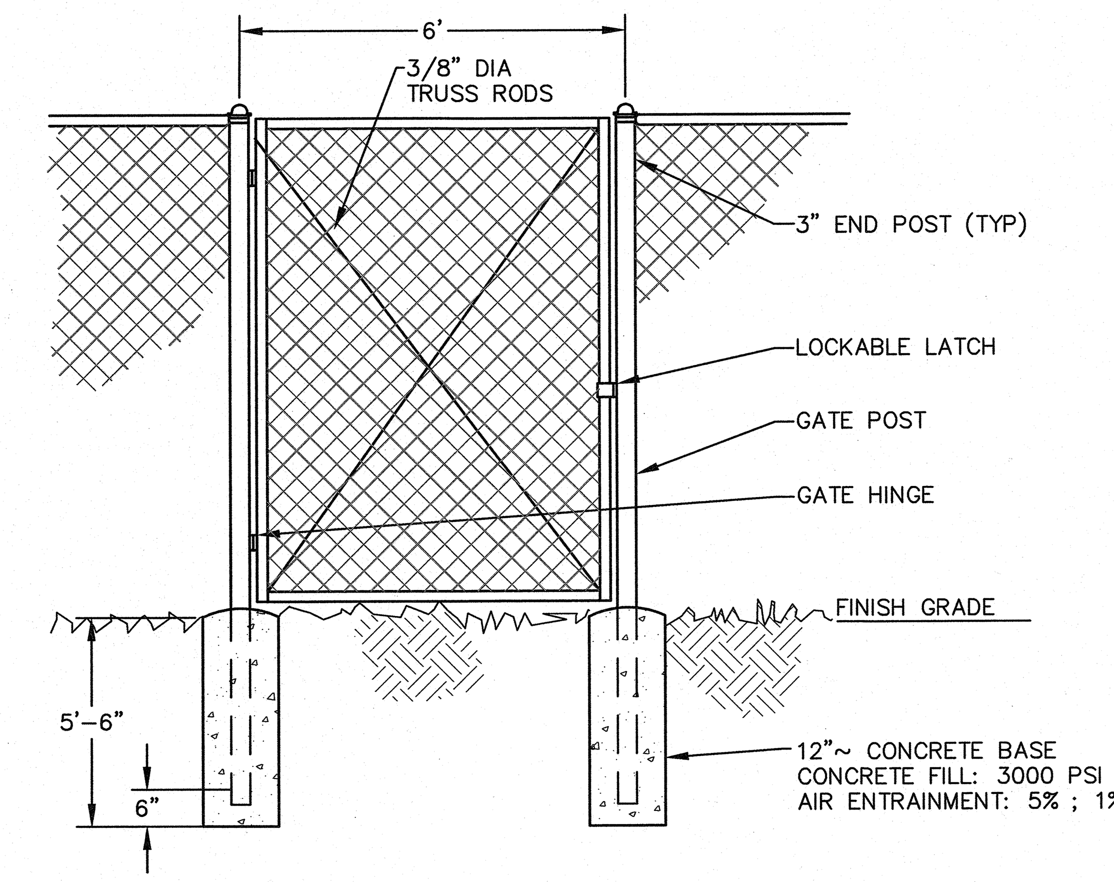
FINAL COVER SYSTEM
NTS



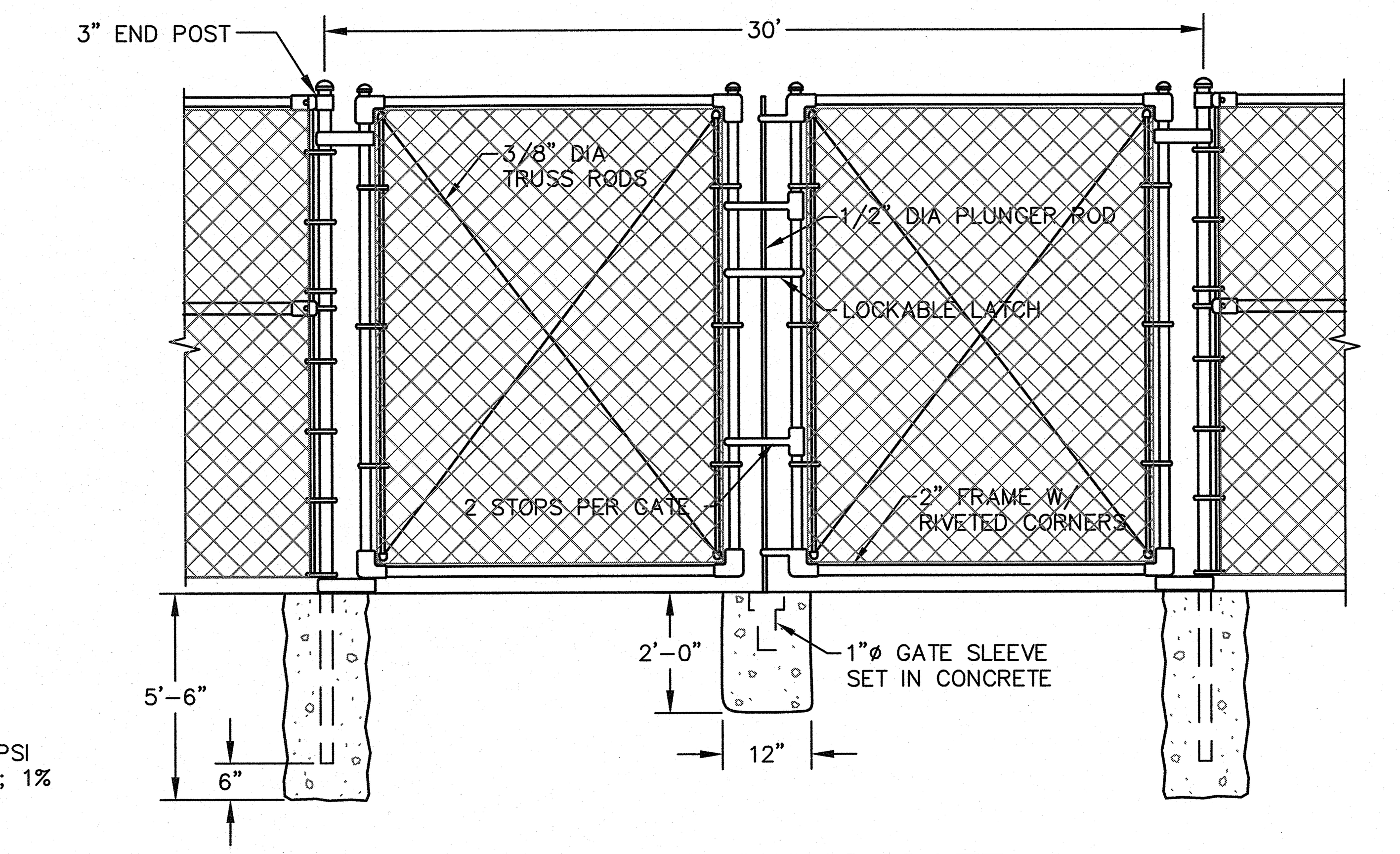
NOTES:

1. INTERMEDIATE POST SPACING SHALL BE 6'-3" UNLESS OTHERWISE SHOWN.
2. POST AND OFFSET BRACKETS SHALL BE 4"x 6" I-BEAM 8.5 OR 9.0 LBS/FT. LENGTH OF 5'-9" ATTACHED WITH 5/8" DIA BOLTS WITH HEX NUTS.
3. ALL HOLES IN BEAM TO BE SHOP-PUNCHED PRIOR TO GALVANIZING.
4. RAIL PANELS AND END SECTIONS TO BE 12 GAUGE STEEL.
5. BACK-UP PLATE TO BE PLACED BEHIND RAIL ELEMENTS AT INTERMEDIATE STEEL POSTS (NON-SPLICE POSTS).
6. ALL PARTS SHALL CONFORM TO CURRENT MAINE DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS.
7. WHEN GUARDRAIL IS CONSTRUCTED AT UP TO FOUR FEET FROM THE EDGE OF PAVEMENT, THE GUARDRAIL WILL BE SET FROM THE GRADE AT THE FACE OF RAIL.
8. END SECTIONS TO BE IN ACCORDANCE WITH MAINE DEPARTMENT OF TRANSPORTATION STANDARDS.
9. GUARDRAIL SET ON A RADIUS OF 150 FEET OR LESS TO BE CIRCULAR.

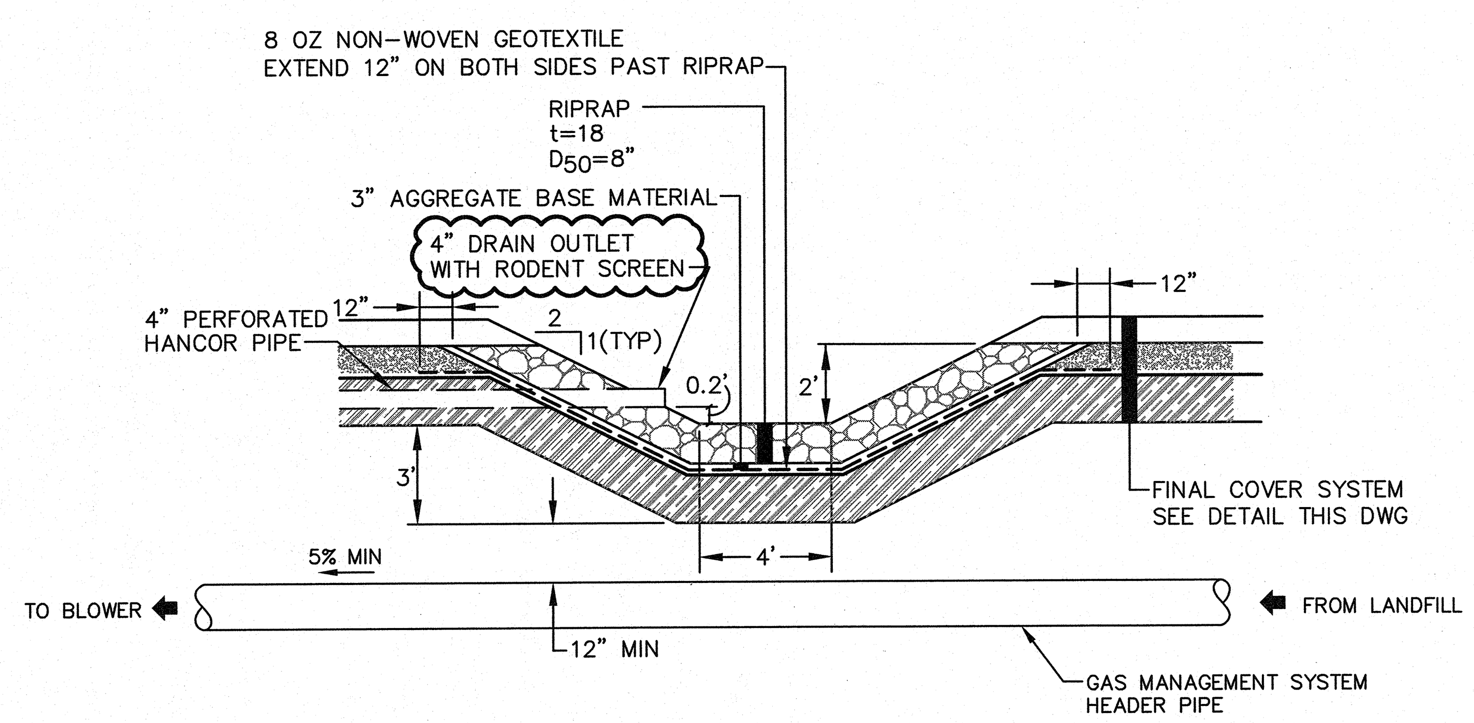
GUARDRAIL
NTS



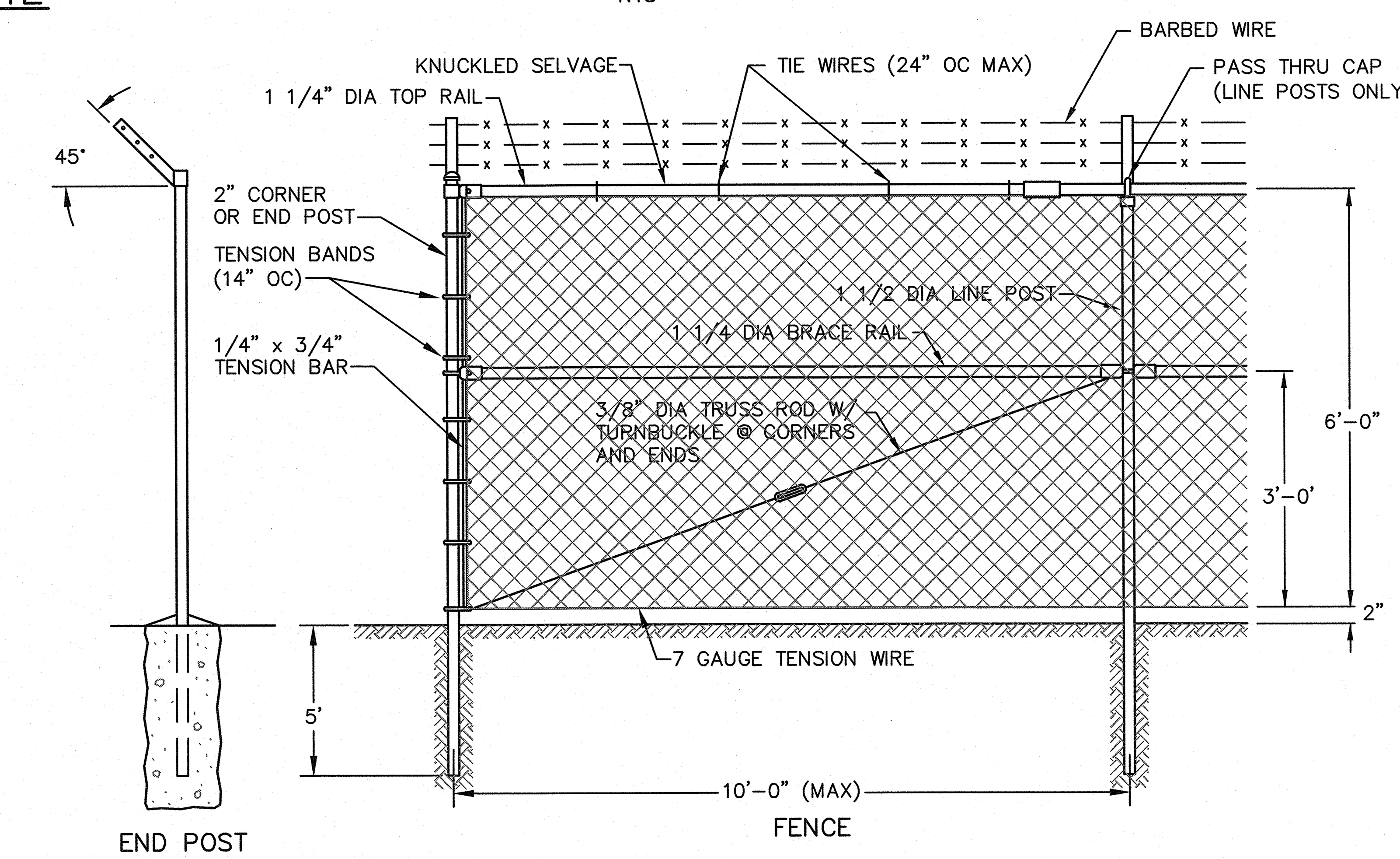
CHAIN LINK FENCE PEDESTRIAN GATE
NTS



DOUBLE SWING GATE
NTS



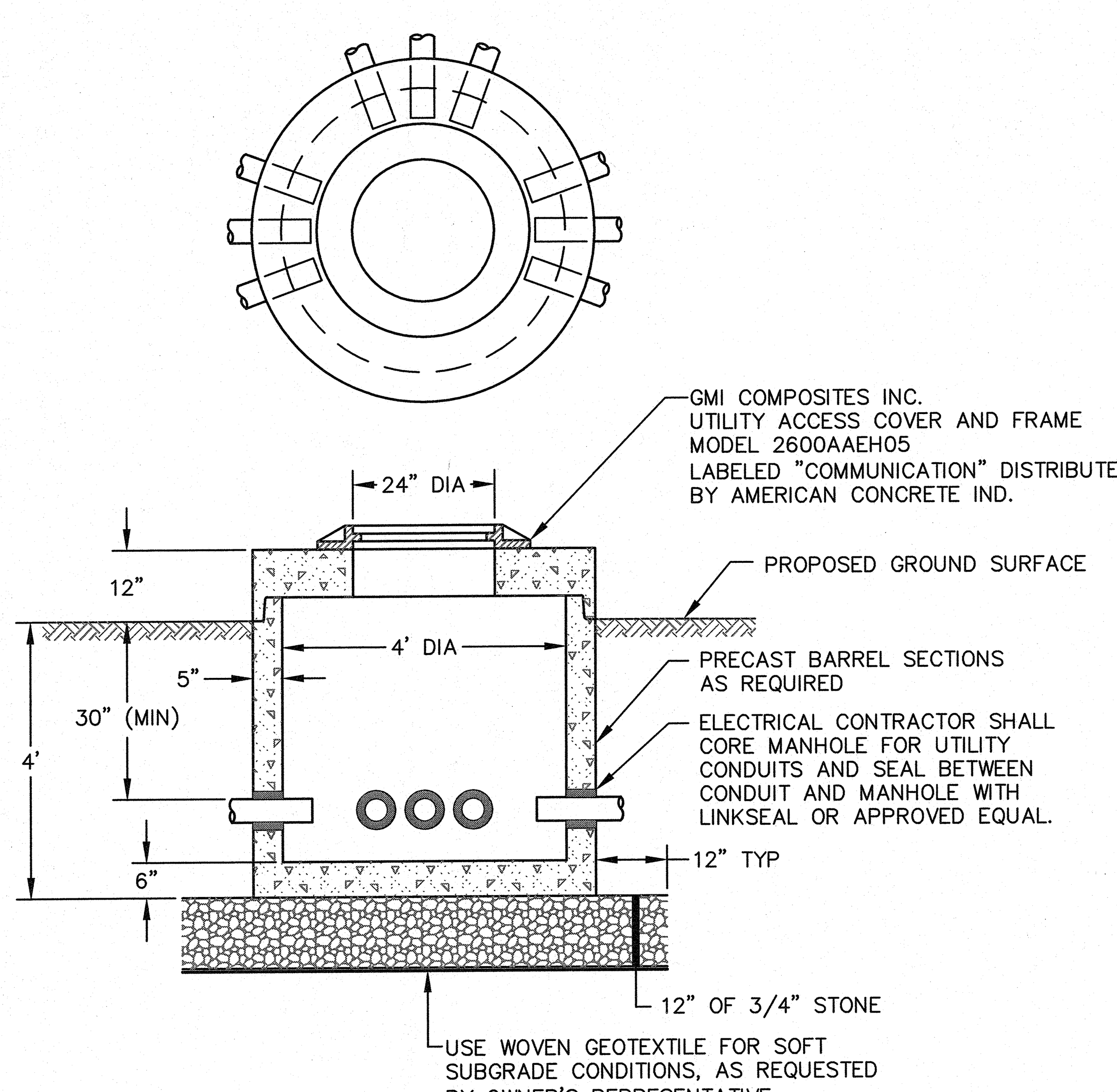
RIPRAP DOWN SPOUT
NTS



NOTES:

1. END OR CORNER POSTS: NOM 2" DIA GALV STL PIPE, MIN 3.65 LB/LF OR 2 1/4" x 2" "H" SECTION, 4.10 LB/LF, OR 3 1/2" x 3 1/2" "L" SECTION WITH INTEGRAL FABRIC LOOPS 5.14 LB/LF.
2. LINE POSTS: NOM 1 1/2" DIA GALV STL PIPE, MIN 2.72 LB/LF OR 1 7/8" x 1 5/8" "H" SECTION, 2.70 LB/LF, OR 1 7/8" x 1 5/8" "C" SECTION, 2.28 LB/LF.
3. TOP & BRACE RAILS: NOM 1 1/4" DIA GALV STL PIPE, MIN 2.27 LB/LF OR 1 5/8" x 1 1/4" "⊓" SHAPED, ROLLED FORMED SECTION.
4. STRETCHER BARS: LENGTH TO BE 1" LESS THAN FULL HEIGHT OF FABRIC. ONE STRETCHER BAR FOR EACH GATE AND END POST, AND TWO STRETCHER BARS FOR CORNER AND BRACING.
5. CLEARING LIMIT 10 FEET EACH SIDE OF FENCE.

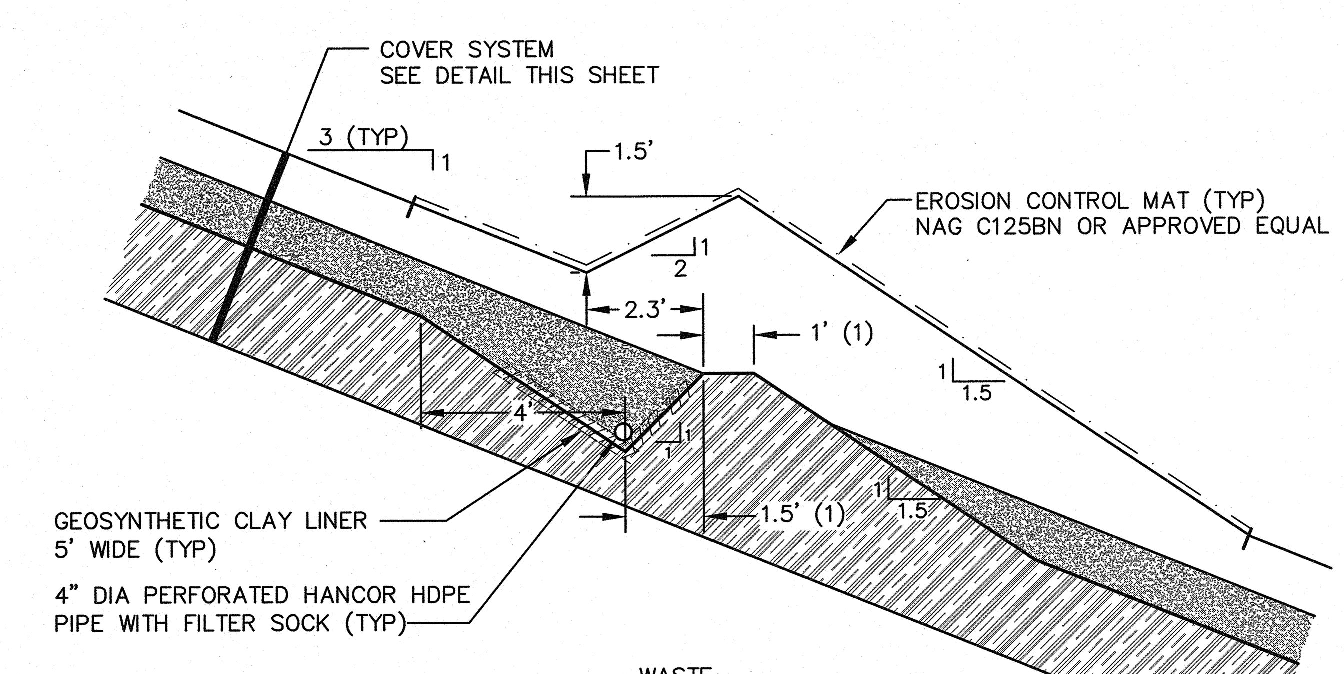
CHAIN LINK FENCE
NTS



NOTES:

1. 4' DIA MANHOLE AS MANUFACTURED BY AMERICAN CONCRETE INDUSTRIES OR ENGINEER APPROVED EQUAL.
2. 4000 PSI CONCRETE AT 28 DAYS.
3. DESIGNED FOR H-20 WHEEL LOADING.
4. CONFORMS TO ASTM C-478 SPECIFICATIONS.
5. REINFORCED TO 0.12 IN SQ/LF.
6. SHIPLAP JOINTS SEALED WITH BUTYL RUBBER.
7. EXTERIOR COATED WITH ASPHALTIC PROTECTIVE DAMPPROOFING.
8. BOTTOM MIN 5'-0" BELOW FINISH GRADE.
9. PRECAST CONCRETE VAULT MANUFACTURER TO PROVIDE ANTI-FLOATATION EXTENDED BASE SLAB AS NECESSARY. ANTI-FLOATATION DESIGN AND SHOP DRAWINGS SHALL BE PREPARED BY THE MANUFACTURER AND SUBMITTED TO THE ENGINEER FOR APPROVAL.

COMMUNICATION UTILITY MANHOLE
NTS



TERRACE DRAINAGE SWALE
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE ... No. 5144 LICENSED PROFESSIONAL ENGINEER

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SECTIONS AND DETAILS

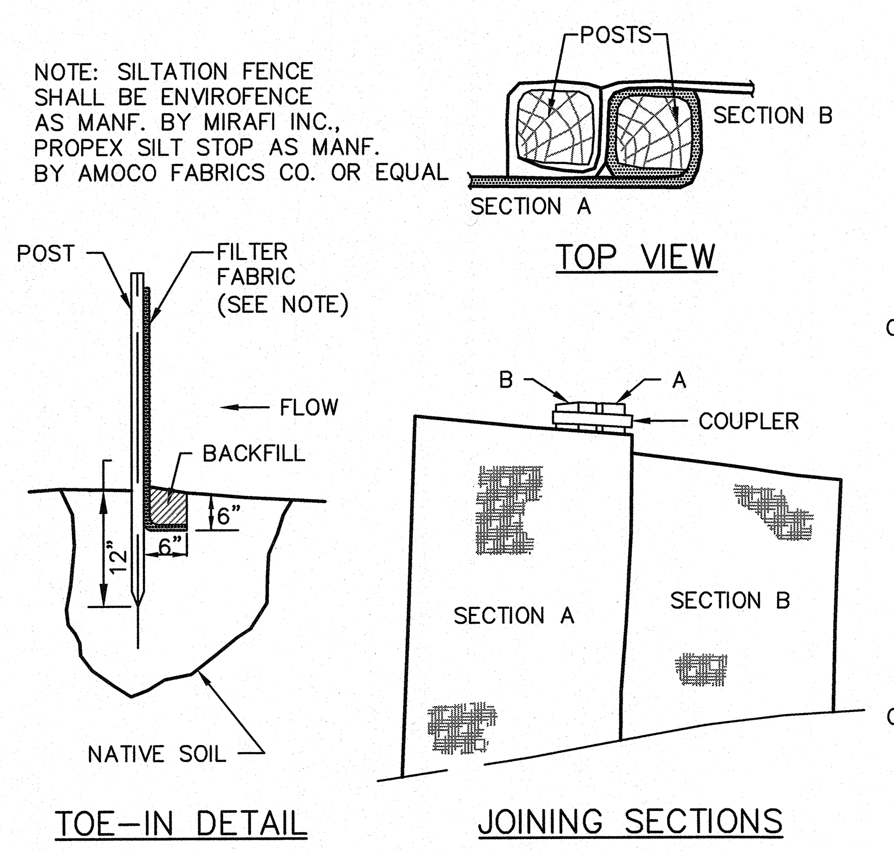
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

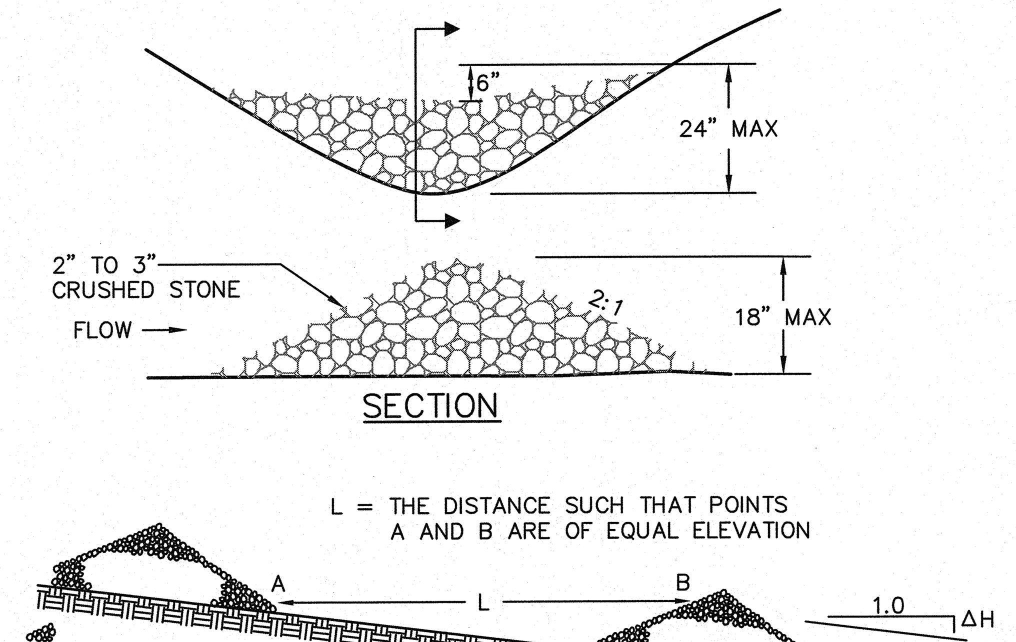
C-307

NOTE: SILTATION FENCE SHALL BE ENVIROFENCE AS MANF. BY MIRAFI INC., PROPEX SILT STOP AS MANF. BY AMOCO FABRICS CO. OR EQUAL



SILTATION FENCE

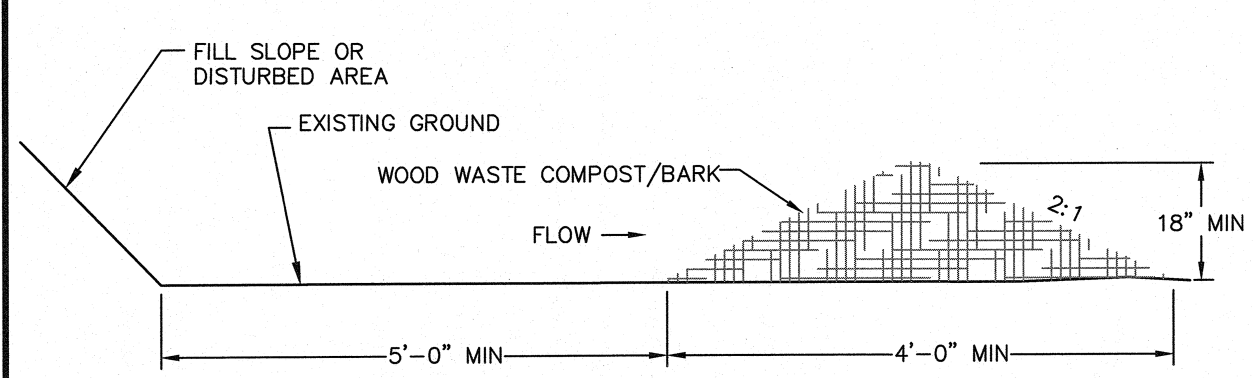
NTS



SPACING BETWEEN CHECK DAMS S_o (FT/FT)	L (FT)
0.020	75
0.030	50
0.040	40
0.050	30
0.080	20
0.100	10

STONE CHECK DAM

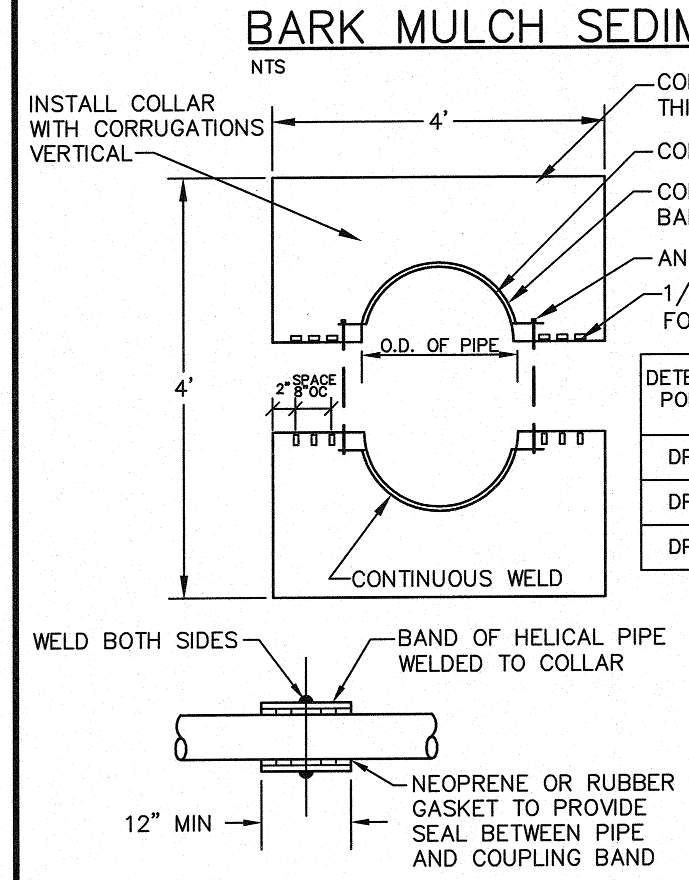
NTS



NOTE:
BARK MULCH SEDIMENT BARRIERS MAY BE USED AS AN ALTERNATE TO SILT FENCE WHEN APPROVED BY THE ENGINEER.

BARK MULCH SEDIMENT BARRIER

NTS

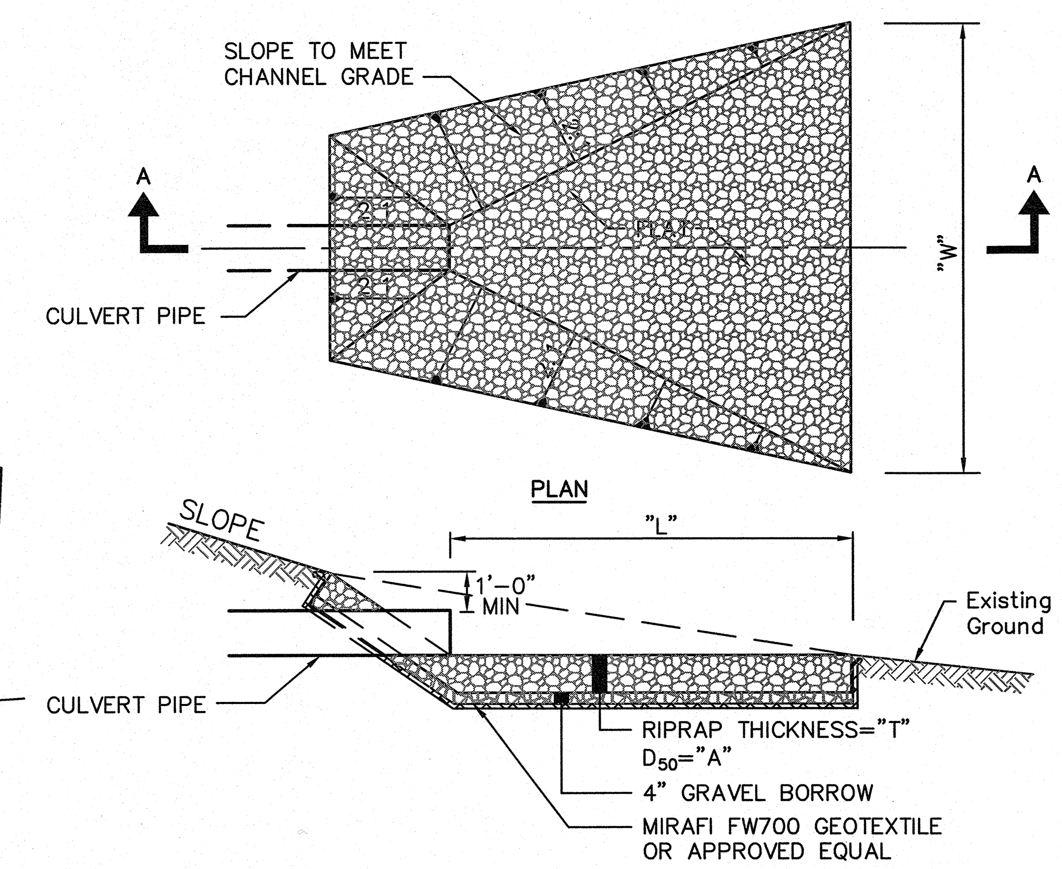


DETENTION POND #	OUTLET PIPE DIA (IN)	PIPE LENGTH (FT)	SATURATED LENGTH (FT)	# OF COLLARS	SPACING (FT)
DP-10	18	52	42	2	11
DP-11	18	92	37	2	10
DP-12	18	80	31	2	10

NOTES:
1. UNASSEMBLED COLLAR SHALL BE MARKED BY PAINTING OR TAGGING TO IDENTIFY MATCHING PARTS.
2. THE LAP BETWEEN THE TWO HALF SECTIONS AND BETWEEN THE PIPE AND CONNECTING BAND SHALL BE CAULKED WITH ASPHALT MASTIC AT TIME OF INSTALLATION.
3. RIPLEY'S DAM BY MCRIP MANUFACTURING MAY BE USED WITH ENGINEERS APPROVAL PROVIDING THAT LENGTH AND WIDTH OF COLLAR IS EQUAL TO OR GREATER THAN THAT SPECIFIED IN THE DETAIL.

ANTI-SEEP COLLAR

NTS

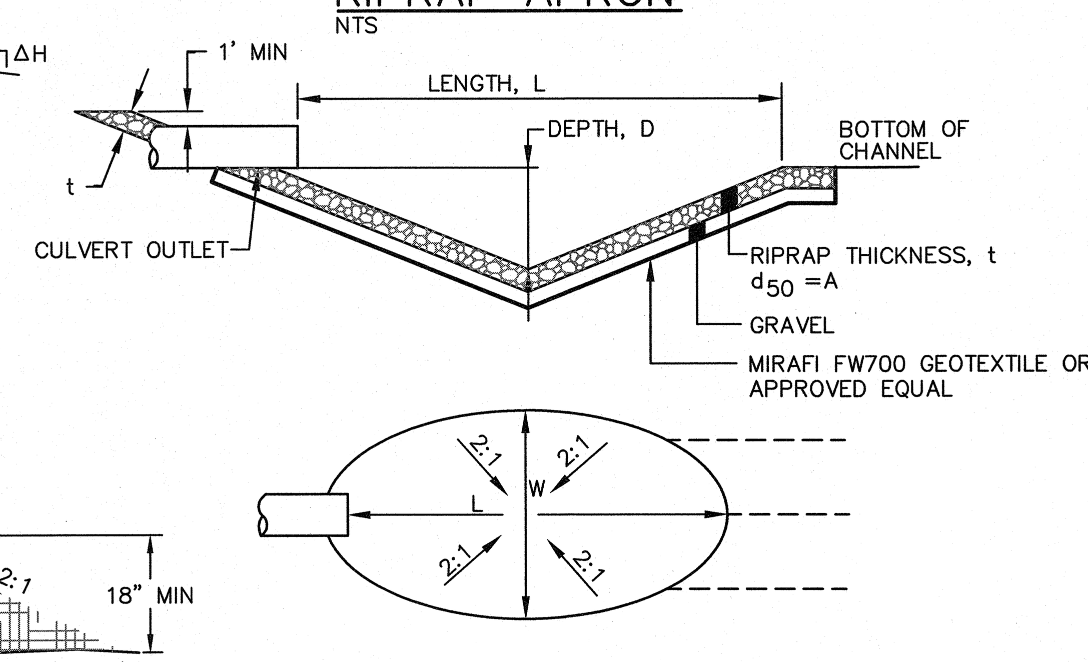


SECTION A-A

CULVERT OUTLET	L (FT)	W (FT)	A (d_{50}) IN.	t (IN)
2BA	18	20	8	18
2BB	18	20	6	14
4BA	12	14	5	12
4BB	12	14	5	12
4F	10	12	4	9
4G	12	14	5	12
4HA	10	12	4	9
4HB	10	12	4	9
4I	18	20	10	23
4IA	10	12	4	9
4JA	18	20	10	23
4JB	12	14	5	12
4JC	12	14	5	12
4K	12	14	5	12
4L	14	16	8	18
4N	10	12	4	9

RIPRAP APRON

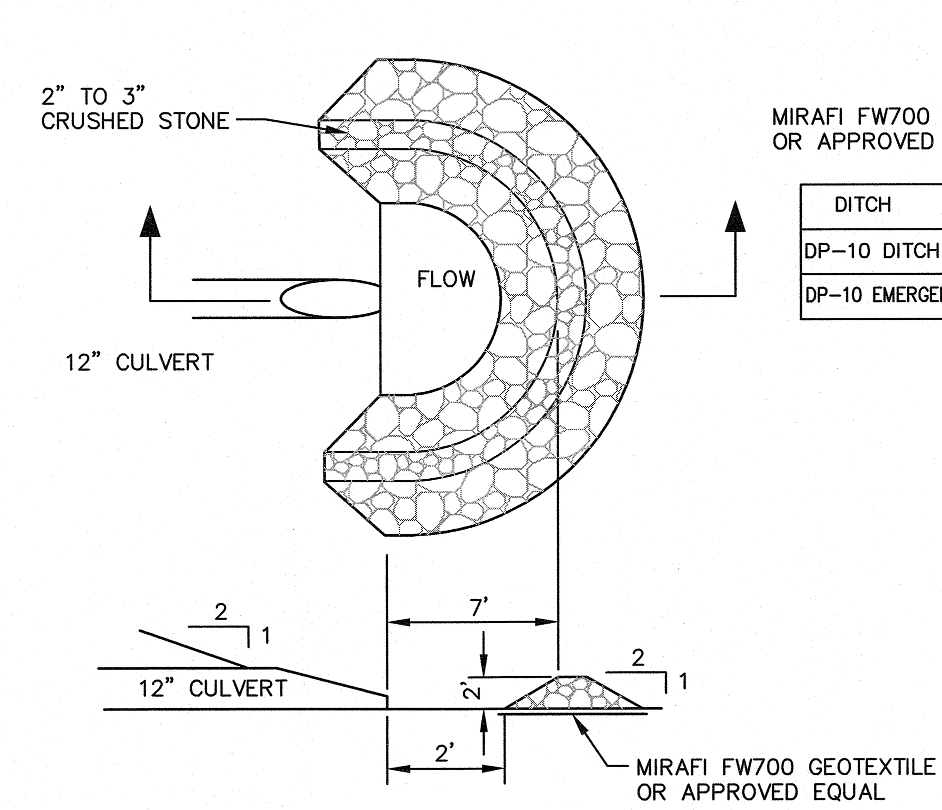
NTS



CULVERT OUTLET	L (FT)	W (FT)	A (d_{50}) IN.	t (IN)	D (FT)
DP-10	6	6	8	18	1.5
DP-11	6	6	4	9	1.5
DP-12	6	6	4	9	1.5

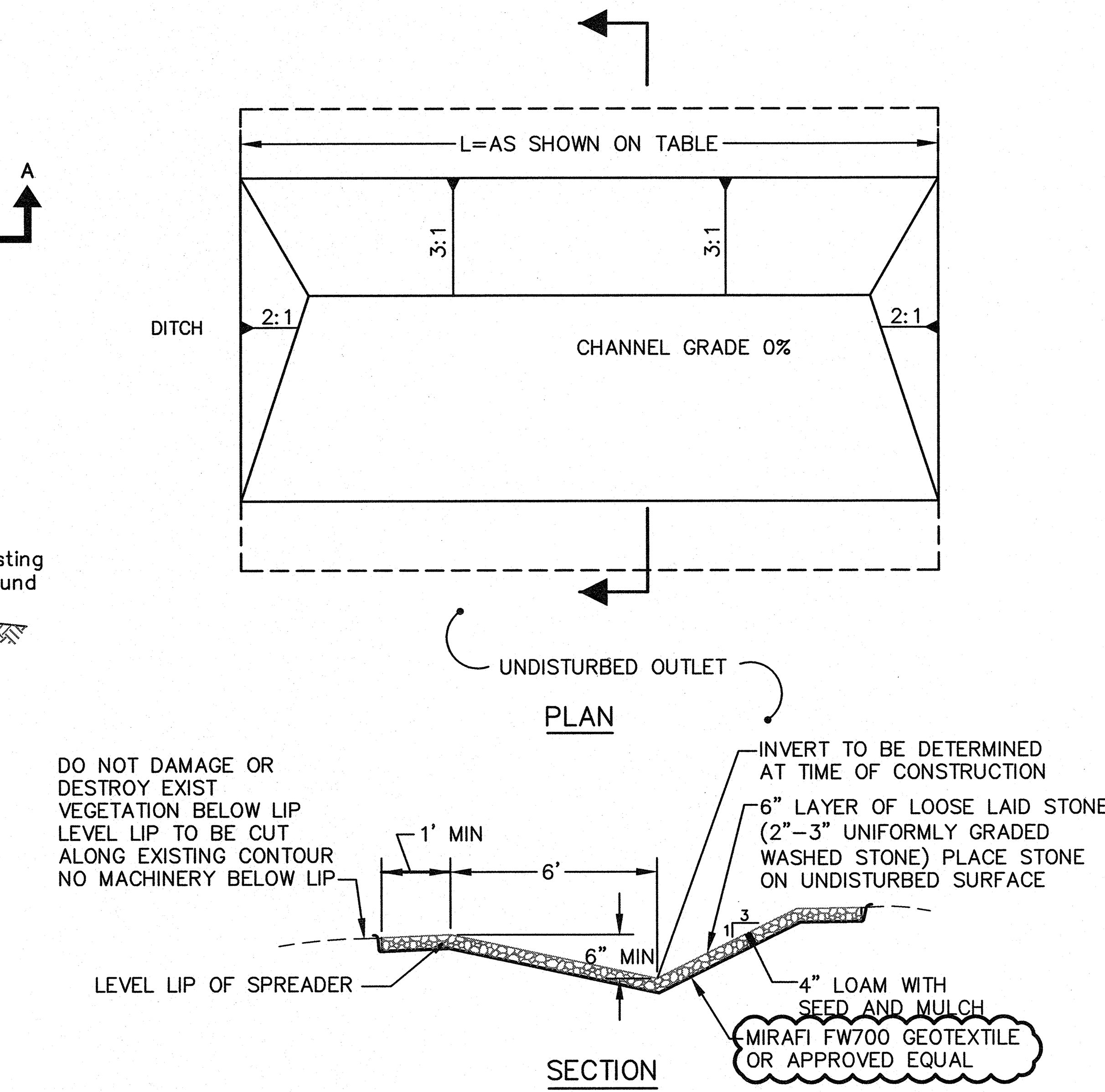
RIPRAP PLUNGE POOL

NTS



RIPRAP CULVERT INLET PROTECTION (TEMPORARY)

NTS

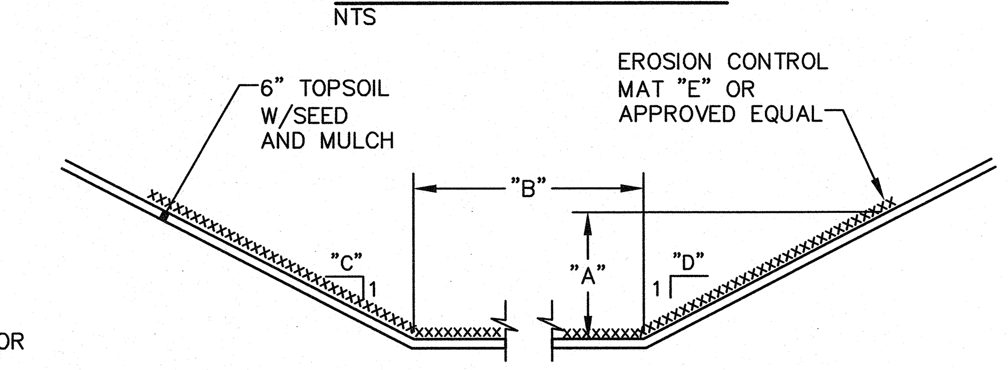


NOTES:
1. CONSTRUCT LEVEL LIP ON ZERO PERCENT GRADE TO INSURE UNIFORM SPREADING OF SEDIMENT - FREE RUNOFF (CONVERTING CHANNEL FLOW TO SHEET FLOW).
2. LAST 20 FT OF DRAINAGE DITCH NOT TO EXCEED 1% GRADE

LEVEL SPREADER	LENGTH, L (FT)
DP-10	20
DP-11	15
DP-12	15

LEVEL SPREADER

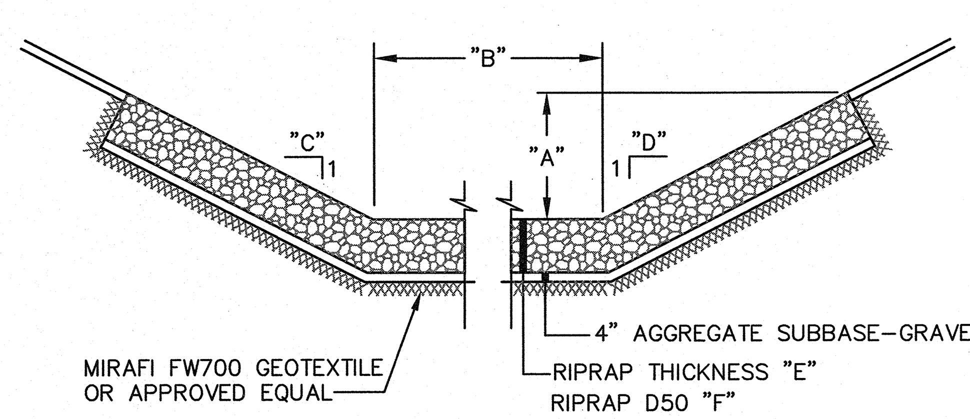
NTS



DITCH	"A"	"B"	"C"	"D"	"E"
TOE DITCH	2'	2'	2'	2'	NAG S75
DP-10 DITCH SEGMENTS 1 & 2	2'	2'	2'	2'	NAG S75
DITCH 4B-1	2'	2'	2'	2'	NAG S75
MAINTENANCE ROAD DITCH	3'	2'	2'	2'	NAG S75

GRASS DITCH

NTS



DITCH	"A"	"B"	"C"	"D"	"E"	"F"
DP-10 DITCH SEGMENT 3	2'	2'	2'	2'	9"	4"
DP-10 EMERGENCY SPILLWAY	2'	10'	2'	2'	9"	4"

RIPRAP DITCH

NTS

EROSION AND SEDIMENTATION CONTROL
TEMPORARY AND PERMANENT

A. GENERAL

1. Soil erosion and sediment control will be done in accordance with the Maine Erosion and Sediment Control: Best Management Practices, Maine Department of Environmental Protection, March 2003.

2. The contractor will be responsible for the repair/replacement/ maintenance of all erosion control measures until all disturbed areas are stabilized.

3. Disturbed areas will be permanently stabilized within 7 days of final grading. Disturbed areas not to be worked upon within 14 days of disturbance, shall be temporarily stabilized within 7 days of the disturbance.

4. Removal of trees, bushes and other vegetation, as well as disturbance of topsoil will be kept to a minimum while allowing proper site operations.

5. Suitable topsoil will be stripped and stockpiled for reuse in final grading. Topsoil will be stockpiled in a manner such that natural drainage is not obstructed and no off-site sediment damage will result. If a stockpile is necessary, the side slopes of the topsoil stockpile will not exceed 2:1. Silt fence will be installed around the perimeter of all topsoil stockpiles. Topsoil stockpiles will be temporarily seeded with Aroostook rye, annual or perennial ryegrass, within 7 days of formation, or temporarily mulched if seeding cannot be done within the recommended seeding dates. Recommended seeding dates and application rates are as follows:

* Aroostook Rye: Recommended Seeding Dates: 9/15 - 11/1
  Application Rate: 112 lbs/acre

* Annual Ryegrass: Recommended Seeding Dates: 4/1 - 7/1
  Application Rate: 40 lbs/acre

* Perennial Ryegrass: Recommended Seeding Dates: 8/15 - 9/15
  Application Rate: 40 lbs/acre

Mulch:

o Hay or Straw: Application Rate: 1.5 - 2.0 tons/acre. Anchor with mulch netting (installed per manufacturer's recommendations)

o Wood Fiber Cellulose: Application Rate: 4,000 lbs/acre. Anchoring not required

B. TEMPORARY MEASURES

1. Silt Fence

(a) Silt fence will be installed prior to and downgradient of all construction activity where soil disturbance may result in erosion.

(b) The height of a silt fence will not exceed 36 inches.

(c) Unless a prefabricated system is utilized, the filter fabric will be purchased in a continuous roll cut to the length of the barrier to avoid the use of joints. When joints are necessary, filter cloth will be spliced together only at a support post, with a minimum 6-inch overlap, and securely sealed.

(d) Posts will be spaced a maximum of 10 feet apart at the barrier location and driven securely into the ground (minimum of 12 inches). When extra strength fabric is used without the wire support fence, post spacing will not exceed 6 feet.

(e) A trench will be excavated approximately 6 inches wide and 6 inches deep along the line of posts and upgradient from the barrier.

(f) The fabric will not extend more than 36 inches above the original ground surface. Filter fabric will not be stapled to existing trees.

(g) When extra strength filter fabric and closer post spacing are used, the wire mesh support fence may be eliminated. In such a case, the filter fabric will be stapled or wired directly to the posts with all other provisions of item (f) applying.

(h) The trench will be backfilled and the soil compacted over the filter fabric.

(i) Silt fences will be removed when they have served their useful purpose, but not before the upgradient areas have been permanently stabilized.

(j) Silt fences will be inspected immediately after each rainfall, which exceeds 1 inch in a 24-hour period, and at least daily during prolonged rainfall. If there are any signs of erosion or sedimentation below them, appropriate repairs will be made. If there are signs of undercutting at the center or the edges, or impounding of large volumes of water behind them, they will be replaced with a temporary crushed stone check dam.

(k) Should the fabric on a silt fence decompose or become ineffective prior to the end of the expected usable life, and the barrier still be necessary, the fabric will be replaced promptly.

(l) Sediment deposits should be removed after each storm event if significant buildup has occurred or if deposits exceed 15 inches in depth.

(m) In lieu of providing the 4" x 4" trench for conditions of frozen ground, severe rocky soil or hummucky conditions with large roots, or other prohibitive conditions. A wood waste compost/bark mulch filler berm may be used in such situations.

2. Stone Check Dams

(a) Stone check dams should be constructed of 2 to 3 inch stone. The stone should be placed according to the configuration shown on the detail. Hand or mechanical placement will be necessary to achieve complete coverage of the ditch or swale and to ensure that the center of the dam is lower than the edges.

(b) Check dams should be installed as the swale is being constructed.

(c) Sediment will be removed from behind the check dams when it has accumulated to one half of the original height of the dam.

(d) Check dams will be removed when the grass has matured sufficiently to protect the ditch or swale. The area beneath the check dams will be seeded and mulched immediately after the check dams are removed.

(e) Regular inspections will be made to ensure that the center of the dam is lower than the edges. Erosion caused by high flows around the edges of the dam will be corrected. If evidence of siltation in the water is apparent downstream from the check dam, the check dam will be inspected and adjusted. Check dams will be checked for sediment accumulation after each significant rainfall.

CONSTRUCTION SEQUENCE

1. Construct temporary sediment and erosion control facilities. Erosion and sediment measures shall be installed prior to any earth moving operation in the area of work.
2. All permanent ditches are to be stabilized with vegetation or stone check dams prior to directing runoff to them.
3. Inspect and maintain all erosion and sediment control measures.
4. Complete permanent erosion control measures which may include seeding, mulching, and landscaping.
5. Remove all temporary erosion control measures.
6. Each stage will be stabilized prior to initiating the next stage.
7. Any exposed areas will be hay mulched prior to winter shutdown, if necessary.

EROSION CONTROL MEASURES

1. The smallest practical area of land shall be exposed to construction at any one time.
2. The temporary erosion control measures shall be maintained until permanent erosion control measures are present.
3. All areas disturbed by construction shall have available loam placed before seeding (or an acceptable alternative).
4. After construction is terminated, all temporary erosion control measures shall be removed and accumulated sediment disposed of in a secure location.
5. Mulch shall be mowings of acceptable herbaceous growth, free from noxious weeds or woody stems, and shall be dry.

3. Wood Waste Compost/Bark Mulch Filter Berms

(a) The filter berm shall consist of an approved wood waste compost/bark mulch mix or recycled composted bark flume grit and fragmented wood generated from water-flume log handling systems or small shredding of stumpage (6 inches long x 1/2" dia.). The mixture needs to be a well-graded blend of organic and mineral substance. The composition is usually manufactured on or off site and by blending it with a well graded sand and gravel. The objective is a tight, heavy, non-erodible mixture that is not composed of one uniform material, i.e. just bark mulch will not suffice. Comparable composted mixes can be used upon approval of the Department of Environmental Protection, Bureau of Land and Water Quality.

(b) The mix shall conform to the following standards:

* Moisture Content 30 - 60%
* PH-5.0-8.0
* Screen Size - 100% less than 3" max.; 70% less than one inch.
* No less than 40% organic material (dry weight) by loss of ignition.
* No stones larger than 2 inch diameter.
* Silts, clays or sugar sands are not acceptable in the mix.

(c) Installation and Size of Berm:

The dimensions of the berm are more a function of the strength of the material than the flows (forces) it will encounter. At a minimum the berm shall be 4 feet wide and 18 inches high. The berm shall be placed, uncompacted along a relatively level contour. Wherever possible the existing surface must be scoured and the mixture keyed in like any other sediment control measure.

(d) Maintenance:

All deficiencies shall be immediately corrected with additional material place on top of the berm to reach the desired height. When the berm is decomposed, clogged with sediment, eroded, or becomes ineffective, it shall be replaced.

(e) Clean up and Retrieval:

At the end of the job, an erosion control berm shall be removed or spread out so that the native earth can be seen below.

(f) Rock Filter Berms

To provide more filtering capacity or to act as a velocity check dam, a berm's center can be composed of clean crushed rock ranging in size from the French drain stone to riprap. The rocks shall be laid on geotextile to facilitate removal and the geotextile shall be wrapped over the core layer of stone and then covered with another layer of erosion control mix. The center core of stone shall be approx. 12 inches high or two-thirds the height of the filter berm. Rock filter berms shall be a minimum of 18 inches high by 4 feet wide.

4. Stabilized Construction Entrance

(a) Aggregate size: Use 2 inch stone, or reclaimed or recycled concrete equivalent.

(b) Aggregate thickness: Not less than eight inches.

(c) Width: 16 foot minimum, but not less than the full width of where ingress or egress occurs.

(d) Length: as required, but not less than 50 feet.

(e) Geotextile: To be placed over the entire area to be covered with aggregate. Piping of surface water under entrance shall be provided as required. All piping is impossible, a mountable berm with 5:1 slopes will be permitted.

(f) Criteria for Geotextile: The filter cloth shall be woven or NON-WOVEN fabric consisting only of continuous chain polymeric filaments or yards of polyester. The fabric shall be inert to commonly encountered chemicals, hydrocarbons, mildew and rot resistant.

(1) Acceptable materials are Trivira Spunbound 1135, Mirafi 600X, or equivalent.
(2) Fabrics not meeting these specifications may be used only when design procedure and supporting documentation are supplied to determine aggregate depth and fabric strength.

(g) Maintenance: The entrance shall be maintained in a condition which will prevent tracking of sediment onto public rights-of-way. When washing is required, it shall be done in on an area stabilized with aggregate which drains into an approved sediment trapping device. All sediment shall be prevented from entering storm drains, ditches, or waterways.

5. Erosion Control Mats

(a) During the growing season (April 15 to September 15) use mats specified in the drawings or, if not specifically identified, use North American Green S75 or equal or mulch with netting on:

* The base of grassed waterways and steep slopes (>15 percent)
* Any disturbed soil within 100 feet of streams and wetlands.

During the late fall and winter (September 15 to April 15) use heavy grade mats specified in the drawings or, if not specifically specified, use North American Green SC150 or equal on all areas noted above, plus use lighter grade mats or mulch with netting on:

* Sideslopes of grassed waterways
* Moderate slopes (>8 percent)

(2) Install mats in accordance with manufacturers' recommendations.

C. PERMANENT MEASURES

1. Riprapped Aprons and Plunge Pools

(a) Construct riprapped aprons in accordance with the details shown on the drawings.

(b) Stone for riprap will consist of sub-angular field stone or rough unhewn quarry stone. The stone will be hard and of such quality that it will not disintegrate on exposure to water or weathering, be chemically stable and suitable in all other respects for the purpose intended. The bulk specific gravity (saturated surface-dry basis) of the individual stones will be at least 2.5.

(c) The riprap should be placed so that it produces a dense well-graded mass of stone with a minimum of voids. The desired distribution of stones throughout the mass may be obtained by selective loading at the quarry, controlled clumping of successive loads during final placing, or by combination of these methods. The riprap should be placed to its full thickness on one operation. The riprap should not be placed in layers. The riprap should not be placed by dumping into chutes or similar methods which are likely to cause segregation of the various stone sizes. Care should be taken not to dislodge the underlying material when placing the stones.

The finished slope should be free of pockets of small stone or clusters of large stones. Hand placing may be necessary to achieve the required grades and a good distribution of stone sizes. Final thickness of the riprap blanket should be within plus or minus 1/4 of the specified thickness.

(d) Riprap will be inspected periodically to determine if high flows have caused scour beneath the riprap or dislodged any of the stone. If repairs are needed, they should be accomplished immediately.

2. Topsoil, Seed, Mulch

(a) Topsoil: Use stockpiled materials spread to the depths shown on the plans, if available. Approved topsoil substitutes may be used (refer to Section C-2 of Erosion and Sediment Control BMP, see Note 2).

(b) Seeding should be completed by August 15 of each year. Late season seeding may be done between August 15 and September 15. Areas not seeded or which do not obtain satisfactory growth by October 1, will be seeded with Aroostook Rye or mulched at rates previously specified herein. After November 1, or the first killing frost, disturbed areas should be treated as specified in (c) below.

SEEDING SPECIFICATIONS

Permanent Seeding (120 lbs/acre)		Temporary Seeding (120 lbs/acre) (Aroostook Rye 100%)
Tall Fescue	54 lbs/acre	
Red Fescue	25 lbs/acres	
Red Top	5 lbs/acres	
Ladino Clover	13 lbs/acre	
Annual Ryegrass	8 lbs/acre	
Birdsfoot Trefoil	5 lbs/acre	
Timothy	10 lbs/acre	

(2) Fertilizer: Apply 1300 pounds per acre of 10-10-10 fertilizer or equivalent per acre (29.8 lbs/1,000 sq. ft.).

(3) Lime: Apply ground limestone at a rate of 3 tons per acre (138 lbs/1,000 sq. ft.).

(4) Mulch: Mulch with hay or straw at 2.0 - 3.0 tons per acre, or 2-3 bales per 1,000 sq. ft.

Anchor mulch with mulch netting installed per manufacturer's recommendations.

(c) If permanent vegetated stabilization cannot be established due to the season of the year, all exposed and disturbed areas not to undergo further disturbance are to have dormant seeding applied and be temporarily mulched to protect the site. The following methods may be used to perform a dormant seeding:

(1) Prepare the seedbed, add the required amounts of lime and fertilizer, then mulch and anchor. After the first killing frost and before snow fall, broadcast or hydroseed the selected seed mixture. Double the regular seeding rates for this type seeding.

(2) When soil conditions permit, between the first killing frost and before snow fall, prepare the seedbed, lime and fertilize, apply the selected seed mixture, and mulch and anchor. Double the regular seeding rates for this type of seeding.

Dormant seedings need to be anchored extremely well on slopes, ditch bases and areas of concentrated flows.

Dormant seeding requires inspection and reseeding as needed in the spring. All areas where cover is inadequate must be immediately reseeded and mulched as soon as possible.

(3) Erosion Control Mats

(a) During the growing season (April 15-Sept 15) use mats indicated on drawings or, if not specified use North American Green S75 or equal or mulch with netting on:

* The base of grassed waterways
* Steep slopes (>15%)
* Any disturbed soil within 100 feet of lakes, streams and wetlands

During the late fall and winter (Sept 15-April 15) use heavy grade mats indicated on drawings or, if not specified use North American Green SC150 or equal on all areas noted above plus use lighter grade mats or mulch with netting on:

* Side slopes of grassed waterways
* Moderate slopes (>8%)

(b) Install mats in accordance with manufacturers' recommendations.

3. Lined Ditches

On designated ditches, use reinforced mats (North American Green as specified or approved equal) as permanent stabilization. Install mats in accordance with manufacturers' recommendations.

D. CONSTRUCTION SEQUENCE

It is anticipated that construction will commence upon receipt of all necessary permits and approvals. The following outlines the preliminary construction sequence:

a. Install silt fence and other temporary erosion control measures for the construction of Cell and accessory facilities such as detention ponds, berms, and service roads;

b. Construct upslope stormwater diversion berms, ditches, culvert outlets, and control structures:

c. Clear and grub Cell areas;

d. Construct service road;

e.. Construct Cell base grade and underdrain system;

f. Construct Cell liner system, and leachate collection system;

g. Operate Cell;

h. As permanent erosion control measures become stabilized, remove temporary measures (e.g., silt fence, stone check dams); and

i. Install intermediate and final cover on cells filled to capacity in areas shown in the Cell Development Plans - Appendix C of this application.

E. CONSTRUCTION INSPECTIONS

Inspections will be undertaken by qualified personnel to ensure that temporary and permanent erosion and sedimentation controls are properly installed and correctly functioning, and that additional erosion control measures are installed if needed. Such inspections will occur bi-weekly and after each significant rainfall event (1 inch or more within a 24 hour period) during construction until permanent erosion control measures have been properly installed and the site is stabilized.

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE
GUY H. COTE
No. 5013
LICENSED
PROFESSIONAL ENGINEER
2/19/16

JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

C-308

VOLUME III APPENDIX K CELL 11 DESIGN DRAWINGS

- **C-102 BASE GRADING PLAN**
- **C-104 LEACHATE COLLECTION PIPING PLAN**
- **C-200 FORCE MAIN AND LANDFILL GAS HEADER PLAN AND PROFILE**
- **C-300 SECTIONS AND DETAILS**
- **C-301 SECTIONS AND DETAILS**
- **C-302 SECTIONS AND DETAILS**
- **C-303 SECTIONS AND DETAILS**
- **C-304 SECTIONS AND DETAILS**
- **C-305 SECTIONS AND DETAILS**
- **C-306 SECTIONS AND DETAILS**

NOTES:

1. BASE MAP PREPARED BY AERIAL SURVEY & PHOTO INC., NORRIDGEWOCK, MAINE. PHOTO DATE 12/31/14. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE. STANDARD PRACTICE DICTATES THAT PLANS COMPILED IN THIS MANNER SHOULD BE FIELD VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION. SITE BENCHMARK INFORMATION:

   PLUG 1 IS A BRASS PLUG ON FORMER LEACHATE POND PUMP STATION LOCATED AT COORDINATES NORTHING 478242.05, EASTING 925376.35 ELEVATION 167.93

   PLUG 2 IS A BRASS PLUG AT THE ADMINISTRATION BUILDING LOCATED AT COORDINATES NORTHING 479497.17, EASTING 926131.46, ELEVATION 215.12

2. EXISTING TOPOGRAPHY SHOWN AT 2-FOOT INTERVALS. EXISTING TOPOGRAPHY SHOWN ON THE DRAWINGS REPRESENT GRADES AT THE TIME OF THE SURVEY. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

3. LOCATIONS OF EXISTING UNDERGROUND UTILITIES INCLUDING ELECTRICAL AND PIPING BASED ON FIELD SURVEY DURING CONSTRUCTION OF CELLS 1, 2, 3A, 3B, 4, 5, 6, 7, 8, LEACHATE POND AND LANDFILL GAS HEADER AND FLARE RE-LOCATIONS. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

4. LEACHATE COLLECTION PIPE SHOWN IN CELLS 9 AND 10 IS PROPOSED AS OF JULY 2015. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

5. WETLAND BOUNDARIES DELINEATED BY WOODLOT ALTERNATIVES, INC. IN 2004 AND STANTEC CONSULTING SERVICES 2008, 2014 AND 2015.

6. MONITORING WELLS AND OBSERVATION WELLS TO BE INSTALLED INCLUDE MW-501, OW-601A, OW-601B, OW-602A, OW-603B AND OW-604A.

7. GRUB AND STRIP IN ALL AREAS WHERE FILL WILL BE PLACED TO REMOVE ORGANIC AND OTHER DELETERIOUS MATERIAL OR A MINIMUM OF 8 INCHES.

8. CONTRACTOR SHALL SUBMIT TO THE ENGINEER A WRITTEN PLAN FOR EXCAVATION, EXCAVATION STABILITY AND STORMWATER RUNOFF CONTROL PRIOR TO ANY CELL 11 TO CELL 7 OR CELL 9 CONNECTION CONSTRUCTION ACTIVITIES.

9. CONTRACTOR TO SUPPLY STORMWATER MANAGEMENT PLAN FOR DIVERSION OF WATER DRAINAGE OFF THE GEOSYNTHETIC COVER LOCATED ON EXISTING SIDESLOPE OF CELLS ADJACENT TO CELL 11.

10. SILT FENCE SHALL BE INSTALLED IN LOCATION OF CLEARING LIMIT LINE WHERE THE LINES ARE SHOWN ADJACENT TO EACH OTHER (OFFSET SHOWN IS FOR CLARITY).

11. PROPOSED EXPANSION GRADES WITHIN THE PROPOSED SOLID WASTE LIMIT REPRESENT BASE GRADES PRIOR TO PLACEMENT OF THE IMPORTED CLAY AND CONSTRUCTION OF THE LINER SYSTEM. THE PROPOSED GRADES SHOWN OUTSIDE THE PROPOSED SOLID WASTE LIMIT ARE SUBBASE ROAD GRADES.

12. AREA IDENTIFIED SHALL BE UNDERCUT 1-FOOT BELOW BASEGRADE. THE SECONDARY LINER SHALL BE AUGMENTED IN THIS AREA AS DETAILED ON DRAWING C-300.

13. DETENTION POND 9 SHALL BE EXPANDED TO THE FINISH GRADES SHOWN. ALL DISTURBED AREAS AND GRADED SURFACES SHALL RECEIVE 4-INCHES OF TOPSOIL AND BE SEEDED AND MULCH.

BASE GRADE INTERIOR LAYOUT POINTS

POINT #	NORTHING	EASTING	ELEVATION
100	479660.3	926881.7	215.1
101	479643.3	926934.4	201.2
102	479647.9	926943.9	200.9
103	479651.9	926952.2	200.6
104	479644.1	926954.9	200.9
105	479640.1	926946.6	201.1
106	479139.3	927126.3	214.0
107	478668.2	927294.2	216.0
108	478281.7	927431.9	217.6
109	478148.5	927423.3	217.8
110	479058.2	926888.4	219.0
111	479053.5	926881.6	221.7
112	479098.7	926854.7	222.0
113	479381.9	926685.1	224.1
114	479444.5	926692.6	211.0
115	479504.8	926649.9	210.3
116	479501.0	926597.5	225.0
117	479509.2	926570.9	225.0
118	479531.7	926615.4	225.0

BASE GRADE EXTERIOR LAYOUT POINTS

POINT #	NORTHING	EASTING	ELEVATION
500	479759.4	926959.1	212.0
501	479760.7	926963.4	212.0
502	479761.6	926966.2	210.5
503	479774.7	927007.2	209.7
504	479661.4	926991.1	214.0
505	479662.8	926995.4	214.0
506	479663.6	926998.3	212.5
507	479515.0	927090.3	214.6
508	479502.9	927145.3	219.7
509	479476.2	927153.7	219.7
510	479428.0	927117.8	216.2
511	479328.7	927104.2	218.7
512	479148.6	927153.3	223.5
513	479150.0	927157.5	223.5
514	479151.0	927160.4	222.0
515	479165.7	927200.8	221.1
516	478762.3	927348.5	219.9
517	478760.0	927418.4	214.2
518	478738.3	927437.4	214.0
519	478676.0	927318.2	224.4
520	478677.4	927322.4	224.4
521	478679.5	927329.5	220.7
522	478680.6	927332.7	220.7

BASE GRADE EXTERIOR LAYOUT POINTS

POINT #	NORTHING	EASTING	ELEVATION
523	478679.1	927379.0	219.8
524	478424.8	927420.9	220.0
525	478426.2	927425.8	220.0
526	478284.8	927454.7	225.1
527	478285.4	927459.2	225.1
528	478294.5	927467.8	219.6
529	478296.5	927473.4	219.6
530	478307.6	927515.1	218.8
531	478215.9	927468.8	218.0
532	478123.7	927466.6	216.0
533	478099.1	927491.1	215.0
534	478089.3	927509.2	214.2
535	478097.6	927550.4	213.8
536	477995.4	927564.1	213.2
537	478062.8	927481.0	216.0
538	478108.7	927442.8	225.1
539	478104.9	927445.1	225.1
540	479515.7	926549.6	225.1
541	479540.7	926611.1	225.1
542	479669.6	926877.9	225.1
543	479716.5	926909.6	199.5
544	479726.0	926906.4	199.3
545	479735.0	926924.3	198.7

ROADWAY CENTERLINE TABLE

CL STATION	EASTING	NORTHING	BASEGRADE ELEVATION	ID
0+00.00	926337.3	479462.7	220.0	CL
0+50.00	926383.6	479481.4	220.0	CL AT PC
1+00.00	926426.3	479507.1	220.0	CL
1+38.42	926452.6	479535.0	219.5	CL AT PT
2+00.00	926489.0	479584.6	217.0	CL
3+00.00	926548.2	479665.2	212.9	CL
4+00.00	926607.4	479745.8	208.7	CL
5+00.00	926666.7	479826.4	204.5	CL
5+46.63	926694.3	479864.0	202.6	CL AT PC
6+00.00	926731.9	479901.4	200.4	CL
7+00.00	926826.1	479929.6	201.9	CL
8+00.00	926918.4	479895.9	207.7	CL
8+96.78	926971.4	479816.7	209.3	CL AT PT
9+00.00	926972.4	479813.7	209.4	CL
9+46.00	926986.8	749770.2	210.0	CL
9+65.27	926992.9	479751.7	210.4	CL
10+00.00	927003.6	479718.7	211.1	CL
10+71.29	927025.6	479650.9	212.4	CL
11+00.00	927034.2	479623.5	212.9	CL
12+00.00	927064.4	479528.1	214.7	CL
13+00.00	927094.5	479432.8	216.5	CL
14+00.00	927124.7	479337.4	218.2	CL
15+00.00	927154.8	479242.1	220.0	CL
15+87.17	927181.1	479159.0	221.6	CL
16+00.00	927185.5	479146.9	221.5	CL
17+00.00	927219.9	479053.0	221.3	CL
18+00.00	927254.3	478959.1	221.0	CL
19+00.00	927288.7	478865.2	220.8	CL
20+00.00	927323.1	478771.3	220.5	CL
21+00.00	927357.5	478677.4	220.2	CL
22+00.00	927391.9	478583.5	219.9	CL
23+00.00	927426.3	478489.6	219.4	CL
24+00.00	927460.6	478395.7	218.8	CL
24+99.59	927494.9	478302.2	218.3	CL
25+00.00	927495.0	478301.8	218.3	CL
26+00.00	927512.0	478203.3	216.6	CL
26+84.89	927526.5	478119.6	214.5	CL
27+00.00	927528.5	478104.6	214.2	CL
27+60.40	927536.5	478044.8	212.7	CL

ROADWAY CENTERLINE TABLE LEGEND
CL = CENTERLINE
PC = POINT OF CURVE
PT = POINT OF TANGENT

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE · LICENSED · PROFESSIONAL ENGINEER · No. 51...

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

BASE GRADING PLAN

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	5/2015
CHECKED BY:	
LMN:	C11-BASEGRADE
CTB:	SME-STD

JOB NO. 14101 DWG FILE CELL11-SITEPLAN C-102

NOTES:

1. BASE MAP PREPARED BY AERIAL SURVEY & PHOTO INC., NORRIDGEWOCK, MAINE. PHOTO DATE 12/31/14. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE. STANDARD PRACTICE DICTATES THAT PLANS COMPILED IN THIS MANNER SHOULD BE FIELD VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION. SITE BENCHMARK INFORMATION:

   PLUG 1 IS A BRASS PLUG ON FORMER LEACHATE POND PUMP STATION LOCATED AT COORDINATES NORTHING 478242.05, EASTING 925376.35 ELEVATION 167.93

   PLUG 2 IS A BRASS PLUG AT THE ADMINISTRATION BUILDING LOCATED AT COORDINATES NORTHING 479497.17, EASTING 926131.46, ELEVATION 215.12

2. EXISTING TOPOGRAPHY SHOWN AT 2-FOOT INTERVALS. EXISTING TOPOGRAPHY SHOWN ON THE DRAWINGS REPRESENT GRADES AT THE TIME OF THE SURVEY. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

3. LOCATIONS OF EXISTING UNDERGROUND UTILITIES INCLUDING ELECTRICAL AND PIPING BASED ON FIELD SURVEY DURING CONSTRUCTION OF CELLS 1, 2, 3A, 3B, 4, 5, 6, 7, 8, LEACHATE POND AND LANDFILL GAS HEADER AND FLARE RE-LOCATIONS. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

4. GRADES SHOWN WITHIN THE LIMITS OF CELLS 9 AND 10 REPRESENT TOP OF SUBGRADE PRIOR TO CELL LINER CONSTRUCTION (TOP OF THE 12-INCH UNDERDRAIN LAYER).

5. LEACHATE COLLECTION PIPE SHOWN IN CELLS 9 AND 10 IS PROPOSED AS OF JULY 2015. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

6. GRUB AND STRIP IN ALL AREAS WHERE FILL WILL BE PLACED TO REMOVE ORGANIC AND OTHER DELETERIOUS MATERIAL OR A MINIMUM OF 8 INCHES.

7. CONTRACTOR SHALL SUBMIT TO THE ENGINEER A WRITTEN PLAN FOR EXCAVATION, EXCAVATION STABILITY AND STORMWATER RUNOFF CONTROL PRIOR TO ANY CELL 11 TO CELL 7 OR CELL 9 CONNECTION CONSTRUCTION ACTIVITIES.

8. CONTRACTOR TO SUPPLY STORMWATER MANAGEMENT PLAN FOR DIVERSION OF WATER DRAINAGE OFF THE GEOSYNTHETIC COVER LOCATED ON EXISTING SIDESLOPE OF CELLS ADJACENT TO CELL 11.

9. PROPOSED EXPANSION GRADES WITHIN THE PROPOSED SOLID WASTE LIMIT REPRESENT BASE GRADES PRIOR TO PLACEMENT OF THE IMPORTED CLAY AND CONSTRUCTION OF THE LINER SYSTEM. THE PROPOSED GRADES SHOWN OUTSIDE THE PROPOSED SOLID WASTE LIMIT ARE SUBBASE ROAD GRADES.

10. THE CONTRACTOR SHALL SUPPLY PIPE AND FITTINGS TO THE OWNER AND PROVIDE INSTALLATION SUPPORT AS NECESSARY TO EXTEND CLEANOUTS. EXTENSIONS SHALL BE INSTALLED IN THE SOFT WASTE LAYER. PIPE SHALL BE HDPE SOLID WALL PIPE SLOPED 2 PERCENT MINIMUM DOWN TO CELL 7 AND CELL 9.

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE · LICENSED · PROFESSIONAL ENGINEER · No. 514

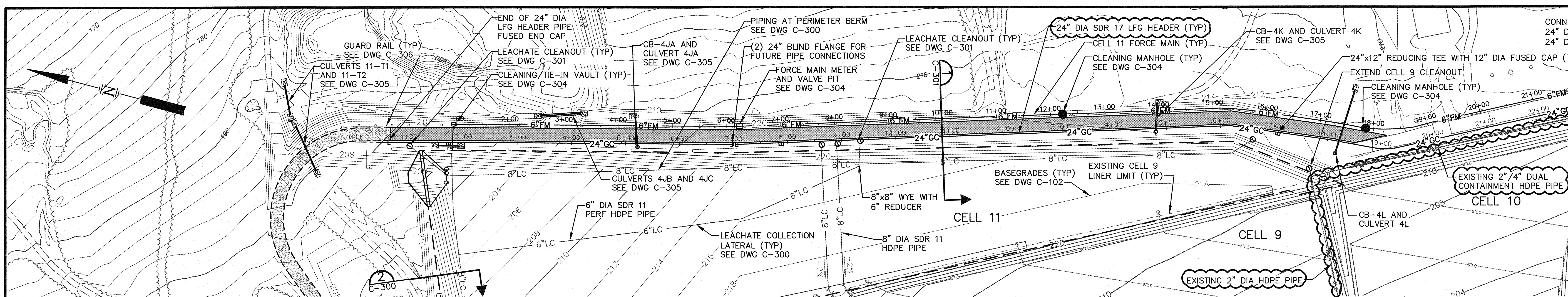
CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

LEACHATE COLLECTION PIPING PLAN

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 5/2015
CHECKED BY:
LMN: LCPIPING
CTB: SME-STD

JOB NO. 14101 DWG FILE CELL11-SITEPLAN C-104

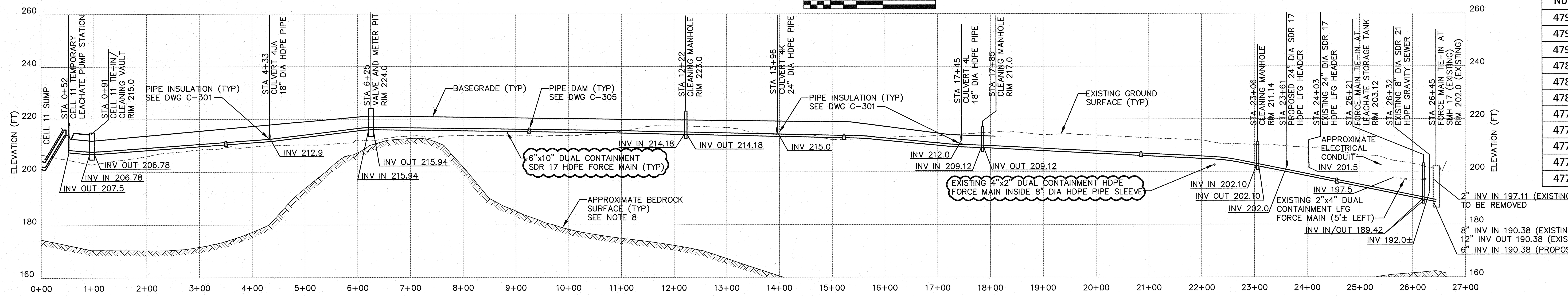


FORCE MAIN AND LFG HEADER PLAN

50 0 100 200 FEET

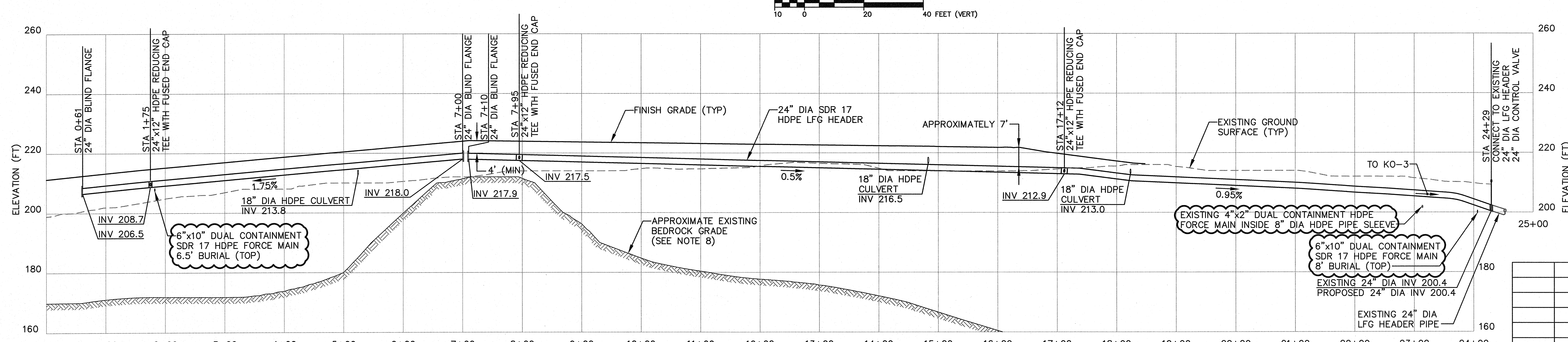
FORCE MAIN CENTERLINE LOCATION TABLE

Northing	Easting	Description
479638.3	926948.6	STA 0+00
479664.5	927040.1	FM VAULT
479160.8	927199.9	FM VAULT
478599.7	927404.8	CLEANING MH
478328.1	927504.8	STA 15+11
478059.4	927552.6	CLEANING MH
477601.5	927797.7	CLEANING MH
477580.4	927760.4	STA 23+49
477547.6	927665.7	STA 24+49
477544.1	927583.8	STA 25+31
477524.2	927496.2	CLEANING MH



FORCE MAIN PROFILE

50 0 100 200 FEET (HORZ)
10 0 20 40 FEET (VERT)



LANDFILL GAS HEADER PROFILE

50 0 100 200 FEET (HORZ)
10 0 20 40 FEET (VERT)

LANDFILL GAS HEADER CENTERLINE LOCATION TABLE

Northing	Easting	Description
479828.2	926951.6	STA 0+00
479770.3	926970.7	STA 0+61
479163.3	927163.8	STA 7+00
479157.5	927165.7	STA 7+10
479078.7	927194.5	STA 7+95
478298.3	927480.3	STA 16+19
478021.0	927525.0	STA 19+00
477629.5	927744.9	STA 23+50
477553.5	927746.5	STA 24+29

NOTES:

1. BASE MAP PREPARED BY AERIAL SURVEY & PHOTO INC., NORRIDGEWOCK, MAINE. PHOTO DATE 12/31/14. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE. STANDARD PRACTICE DICTATES THAT PLANS COMPILED IN THIS MANNER SHOULD BE FIELD VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION. SITE BENCHMARK INFORMATION:

   PLUG 1 IS A BRASS PLUG ON FORMER LEACHATE POND PUMP STATION LOCATED AT COORDINATES NORTHING 478242.05, EASTING 925376.35 ELEVATION 167.93

   PLUG 2 IS A BRASS PLUG AT THE ADMINISTRATION BUILDING LOCATED AT COORDINATES NORTHING 479497.17, EASTING 926131.46, ELEVATION 215.12

2. EXISTING TOPOGRAPHY SHOWN AT 2-FOOT INTERVALS. EXISTING TOPOGRAPHY SHOWN ON THE DRAWINGS REPRESENT GRADES AT THE TIME OF THE SURVEY. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

3. LOCATIONS OF EXISTING UNDERGROUND UTILITIES INCLUDING ELECTRICAL AND PIPING BASED ON FIELD SURVEY DURING CONSTRUCTION OF CELLS 1, 2, 3A, 3B, 4, 5, 6, 7, 8, LEACHATE POND AND LANDFILL GAS HEADER AND FLARE RE-LOCATIONS. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

4. GRADES SHOWN WITHIN THE LIMITS OF CELLS 9 AND 10 REPRESENT TOP OF SUBGRADE PRIOR TO CELL LINER CONSTRUCTION (TOP OF THE 12-INCH UNDERDRAIN LAYER).

5. LEACHATE COLLECTION PIPE SHOWN IN CELLS 9 AND 10 IS PROPOSED AS OF JULY 2015. CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTRUCTION.

6. GRUB AND STRIP IN ALL AREAS WHERE FILL WILL BE PLACED TO REMOVE ORGANIC AND OTHER DELETERIOUS MATERIAL OR A MINIMUM OF 8 INCHES.

7. PROPOSED EXPANSION GRADES WITHIN THE PROPOSED SOLID WASTE LIMIT REPRESENT BASE GRADES PRIOR TO PLACEMENT OF THE IMPORTED CLAY AND CONSTRUCTION OF THE LINER SYSTEM. THE PROPOSED GRADES SHOWN OUTSIDE THE PROPOSED SOLID WASTE LIMIT ARE SUBBASE ROAD GRADES.

8. EXISTING BEDROCK SURFACE INTERPRETED FROM SITE EXPLORATIONS (BORINGS, PROBES AND TEST PITS). CONTRACTOR SHOULD FIELD VERIFY THE EXISTING CONDITIONS PRIOR TO CONSTURCTION.

9. LANDFILL GAS INFRASTRUCTURE FROM PLANS ENTITLED "LFG SYSTEM EXPANSION MASTER PLAN" BY SANBORN HEAD, DATED JUNE 2015. THE PROFILE OF THE LFG HEADER WAS TAKEN FROM DRAWING ENTITLED "PERIMETER LFG HEADER PIPE PROFILE". THE LFG HEADER PROFILE SHOWN ON THE CELL 11 DRAWING SHOWS THE PIPE FROM STATION 9+61 TO STATION 33+29 AS ON THE REFERENCED DRAWING. STATION 0+00 ON THE CELL 11 LFG PROFILE IS EQUAL TO STATION 9+00 ON THE REFERENCED DRAWINGS.

REV.	BY	DATE	STATUS
	Pcm	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE
GUY H. COTE, JR.
No. 5144
LICENSED PROFESSIONAL ENGINEER
2/19/16

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

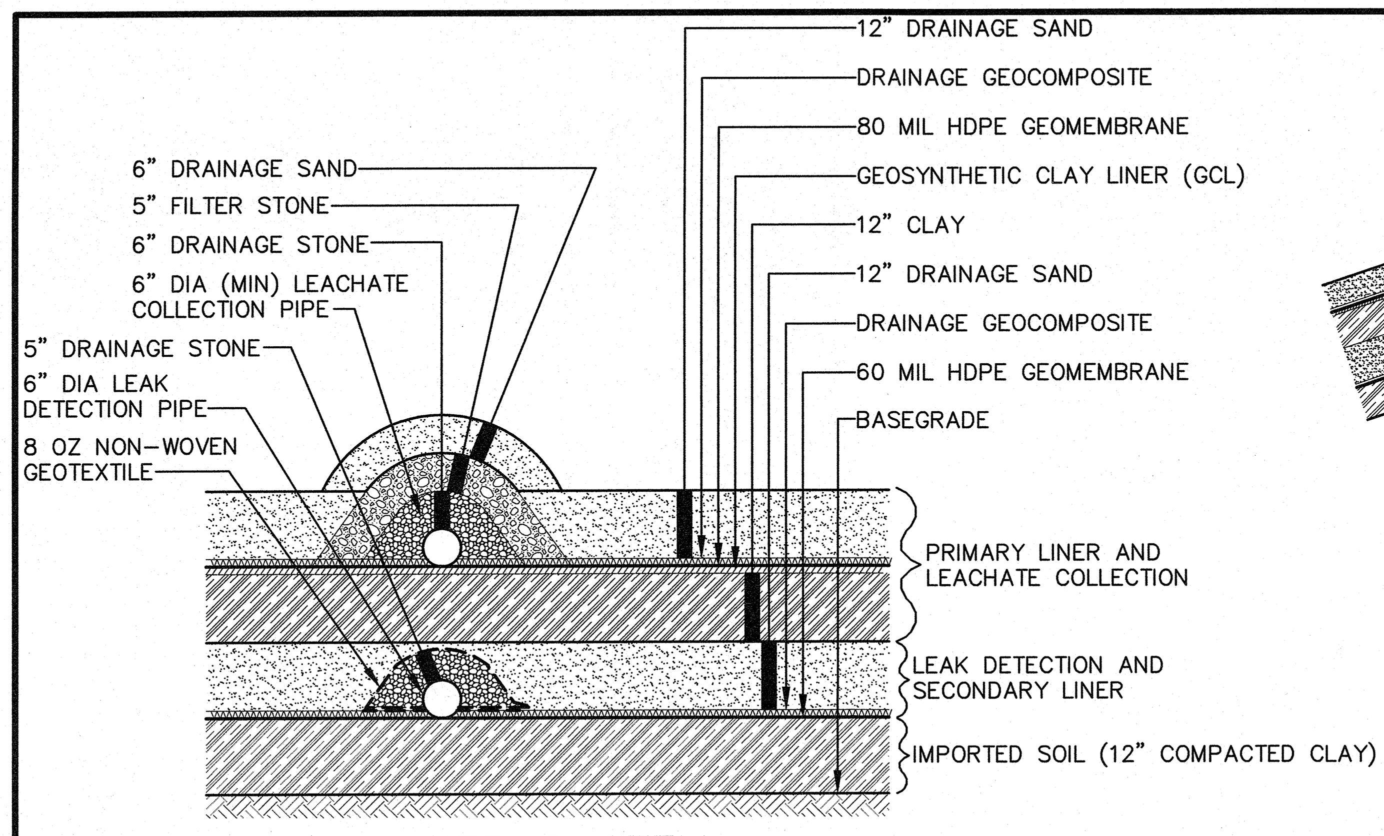
FORCE MAIN AND LANDFILL GAS HEADER PLAN AND PROFILE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

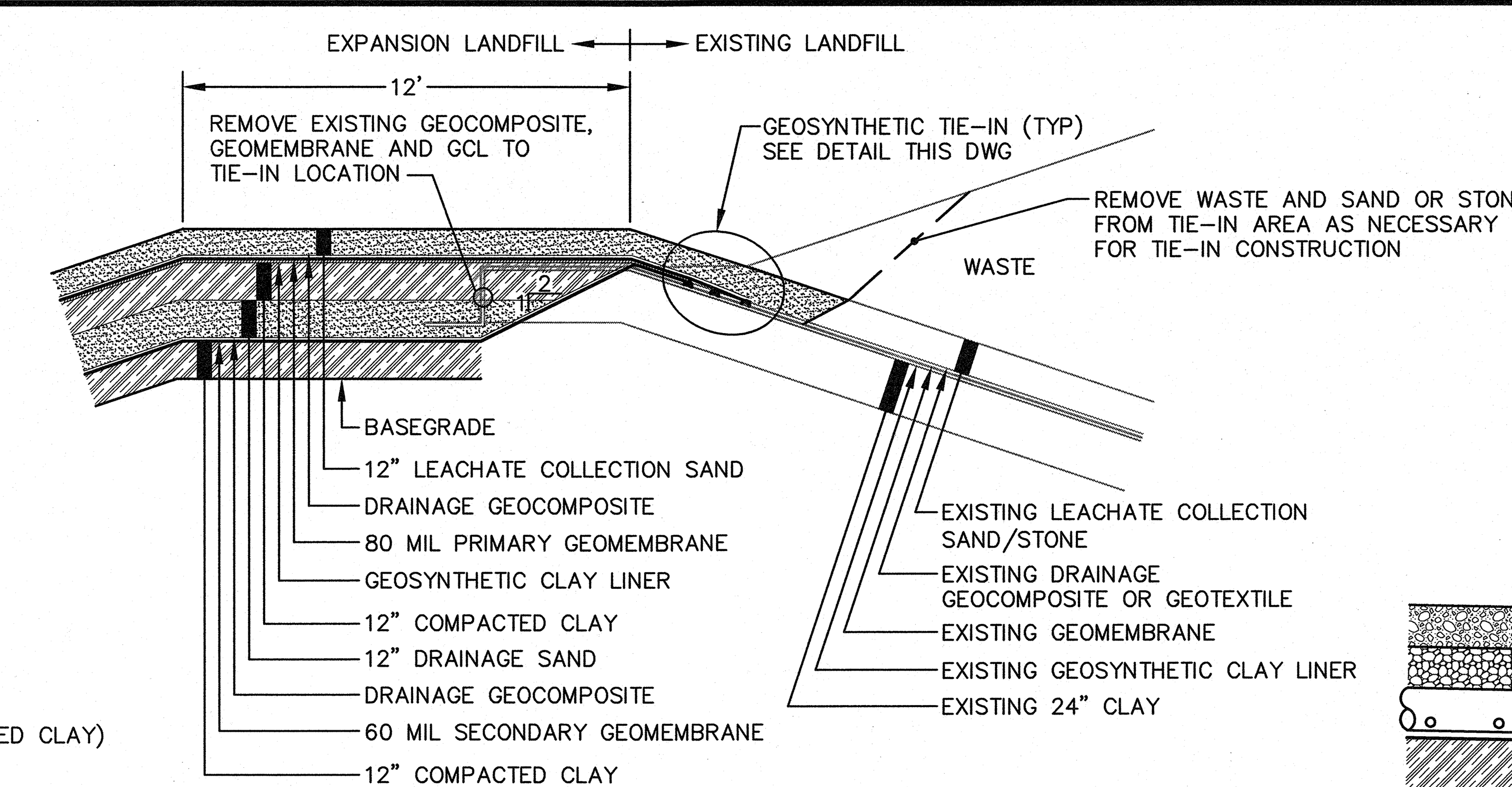
DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	5/2015
CHECKED BY:	
LMN:	FM PLANPROF
CTB:	SME-STD

JOB NO. 14101 DWG FILE CELL11-SITEPLAN

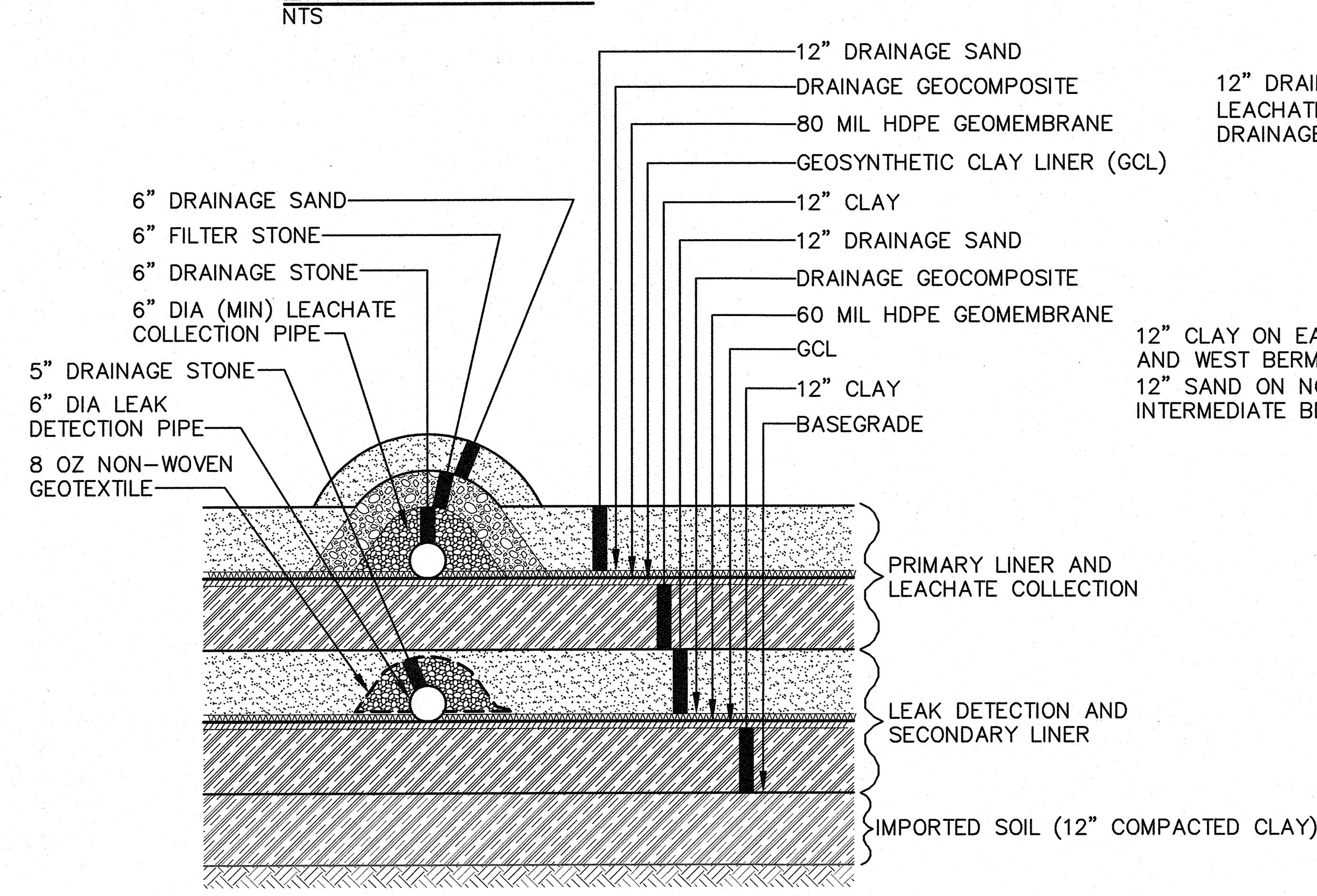
C-200



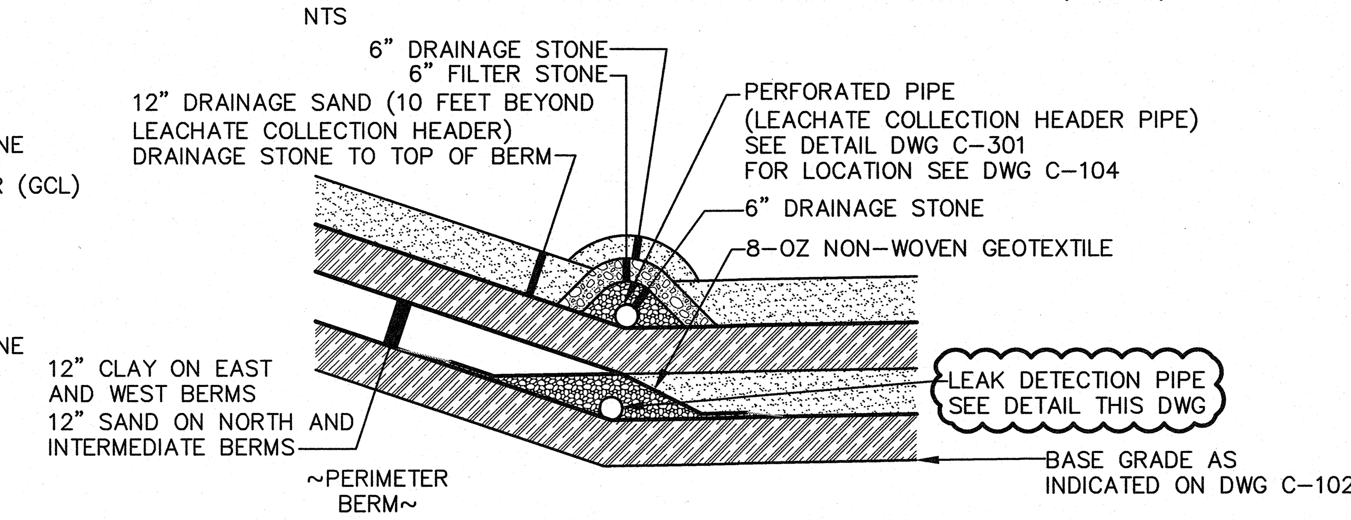
LINER SYSTEM
NTS



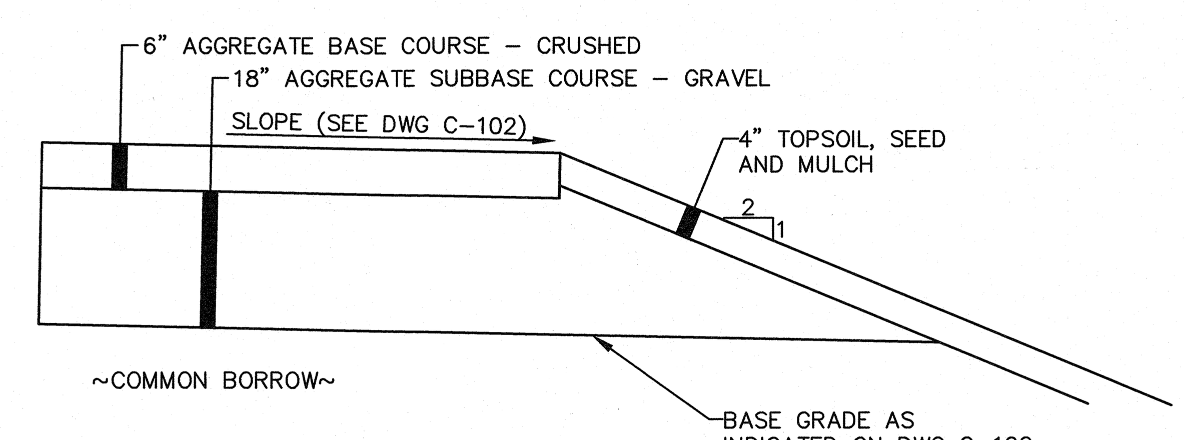
LANDFILL LINER SYSTEM TIE-IN (TYP)
NTS



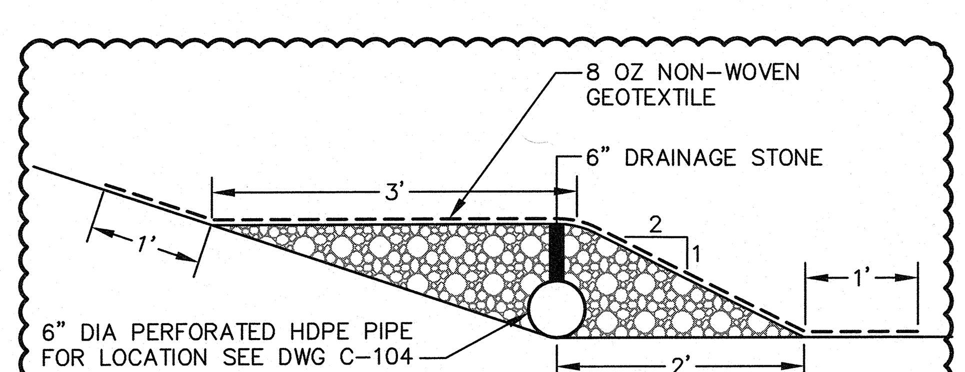
LINER SYSTEM WITH AUGMENTED SECONDARY LINER
NTS



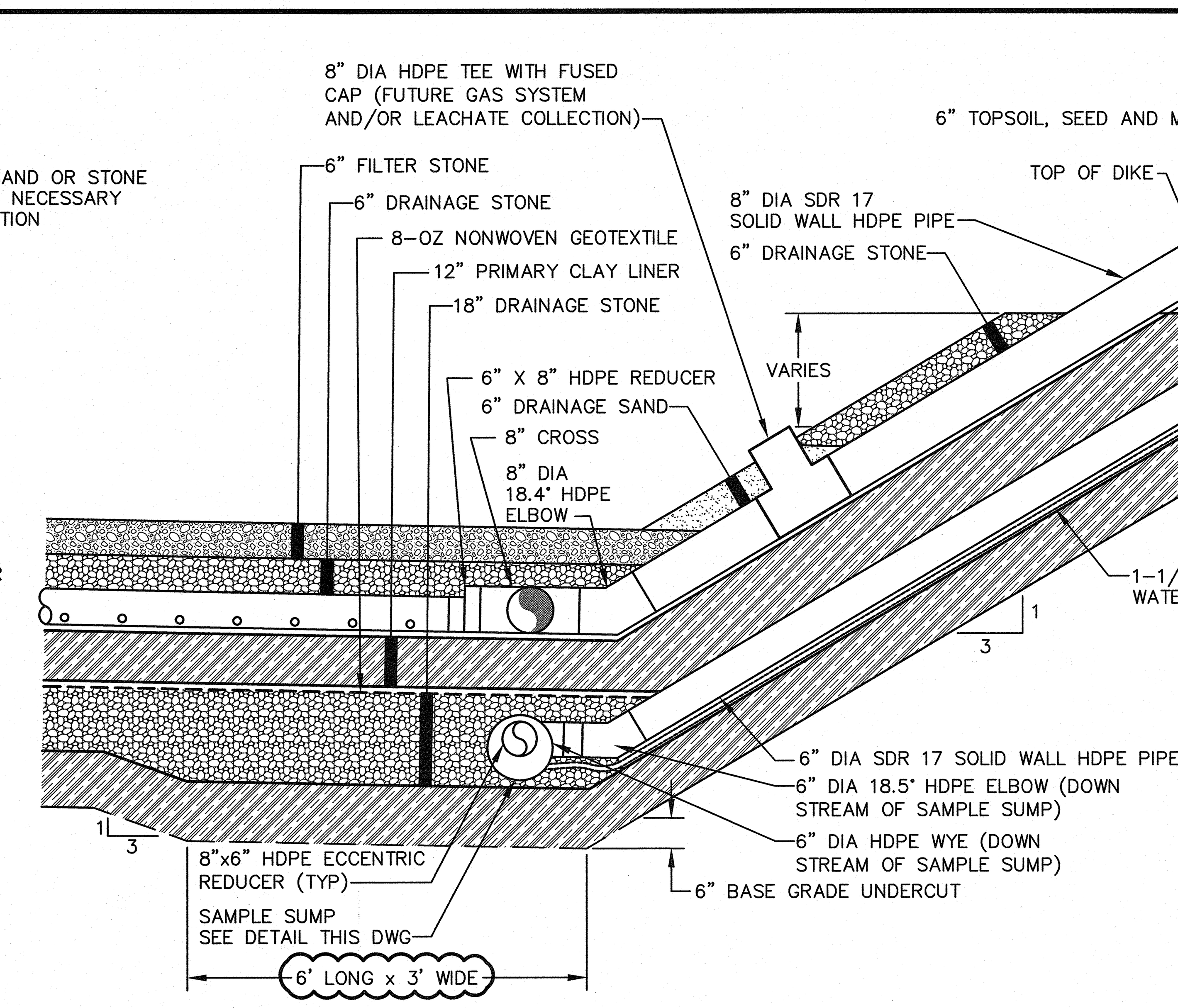
PIPING AT PERIMETER BERM
NTS



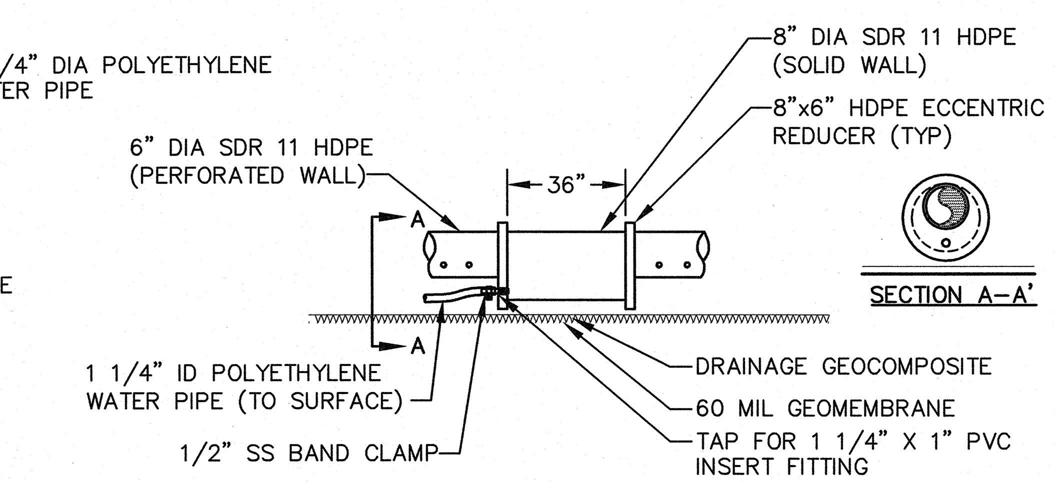
GRAVEL ROAD BUILD-UP
NTS



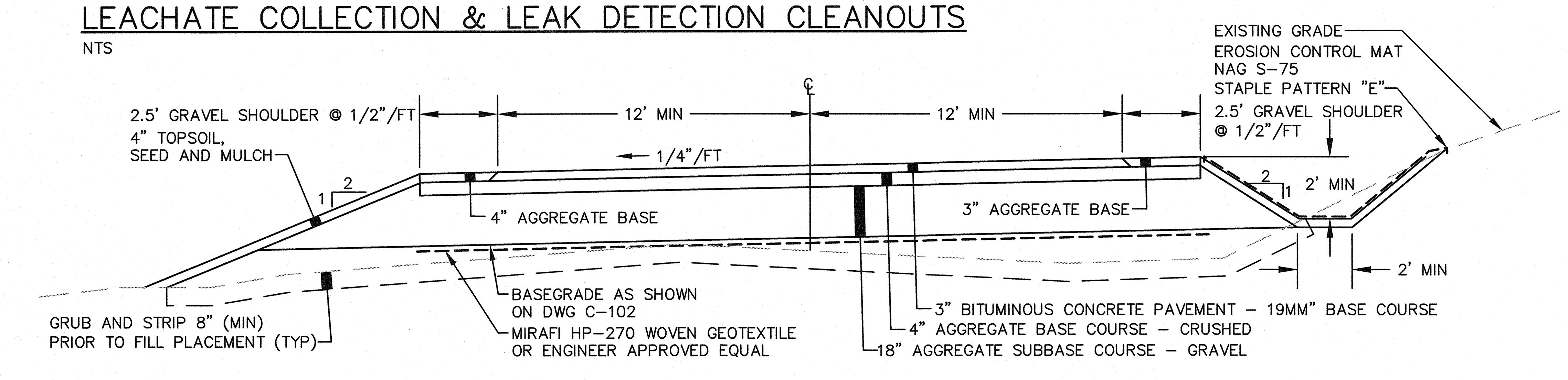
LEAK DETECTION PIPE
NTS



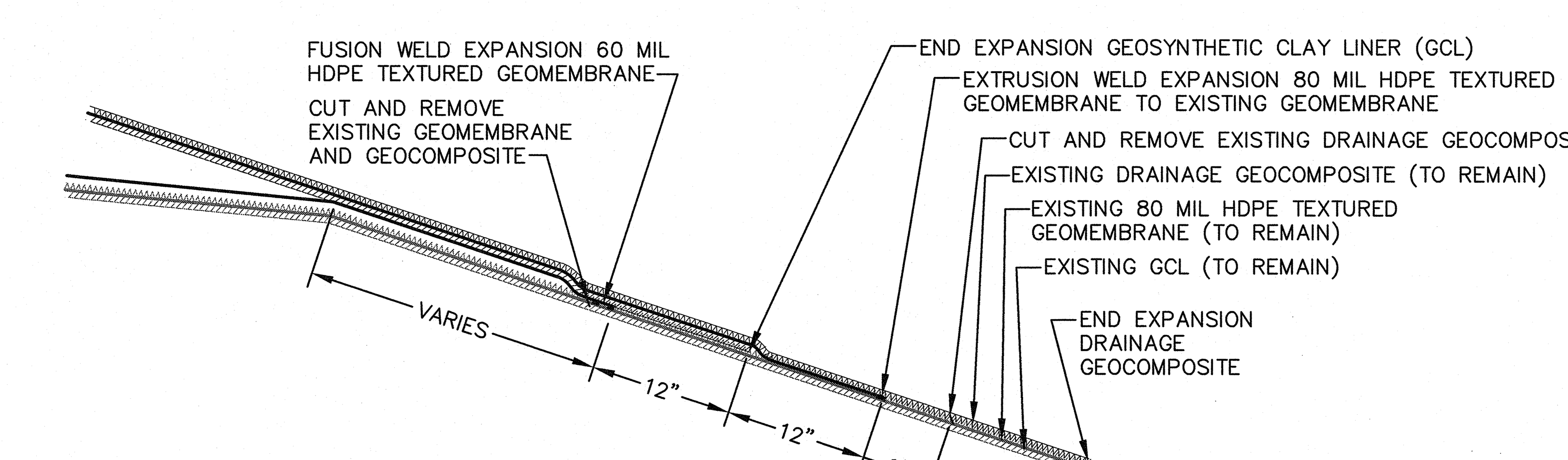
LEACHATE COLLECTION & LEAK DETECTION CLEANOUTS
NTS



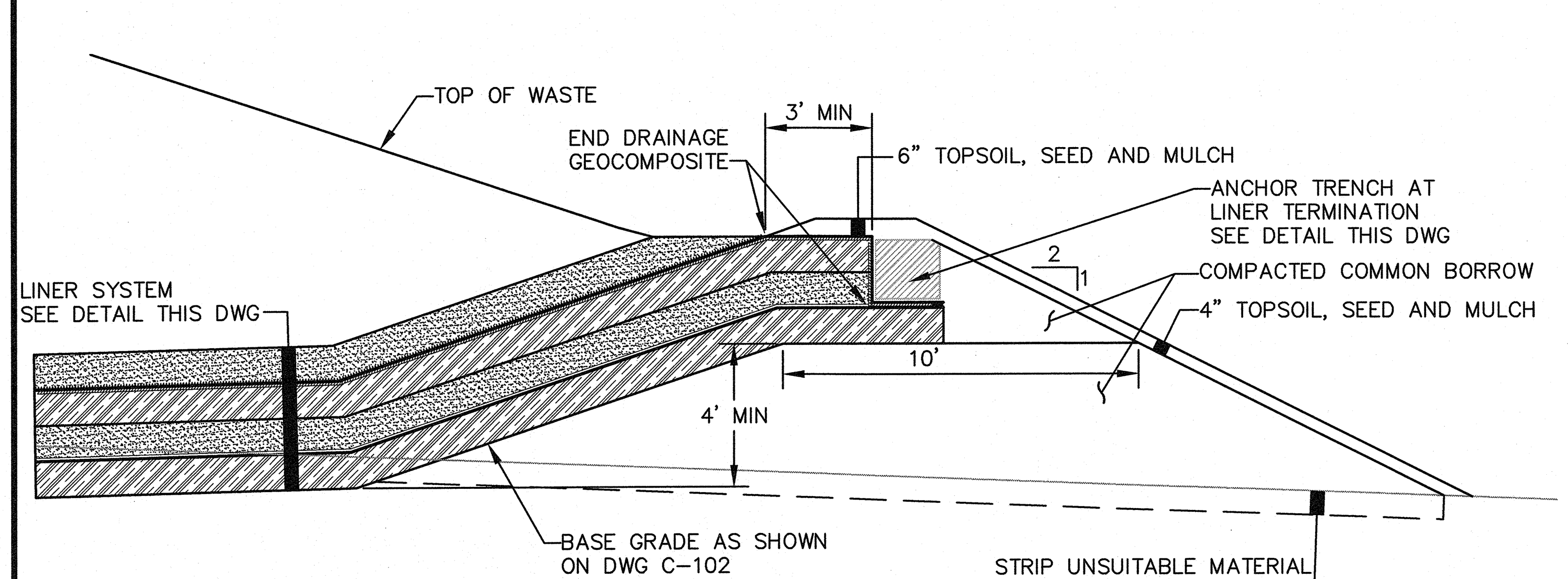
SAMPLE SUMP
NTS



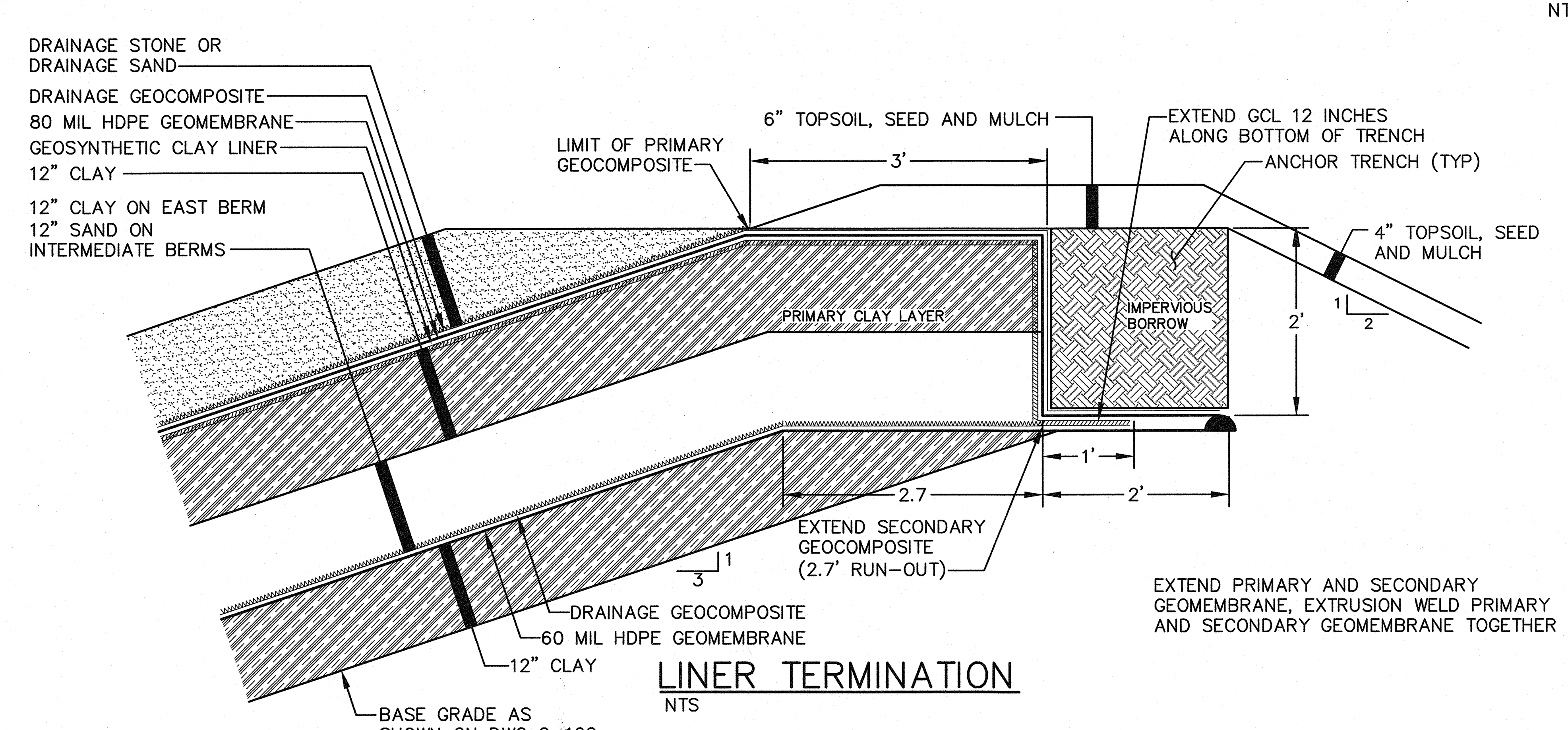
24'-WIDE TEMPORARY PAVED ACCESS ROAD
NTS



GEOSYNTHETIC TIE-IN
NTS



INTERMEDIATE CELL BERM (2)
NTS
C-102
C-103
C-104
C-200



LINER TERMINATION
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

SECTIONS AND DETAILS

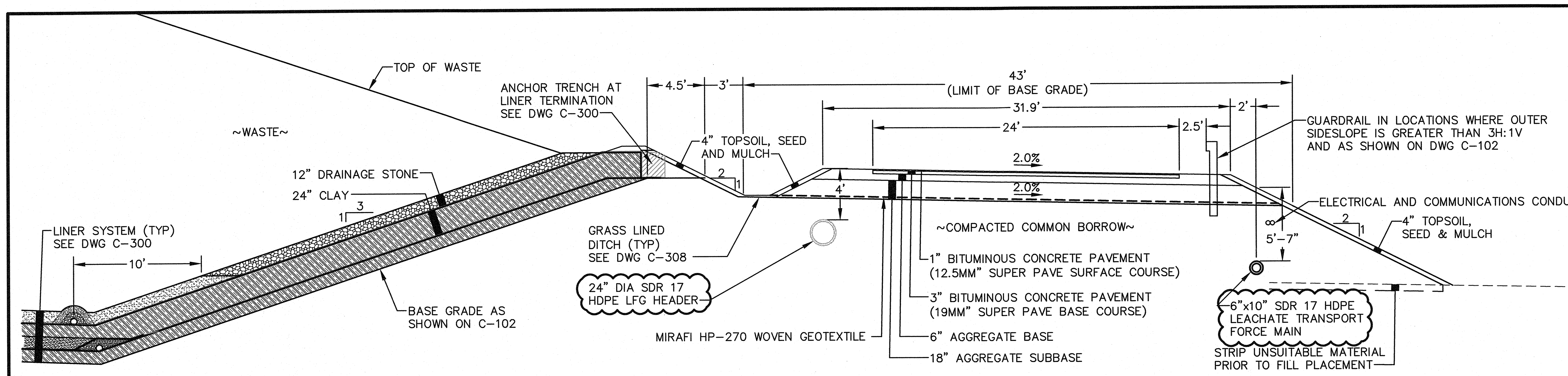
STATE OF MAINE ... No. 5144 LICENSED PROFESSIONAL ENGINEER

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

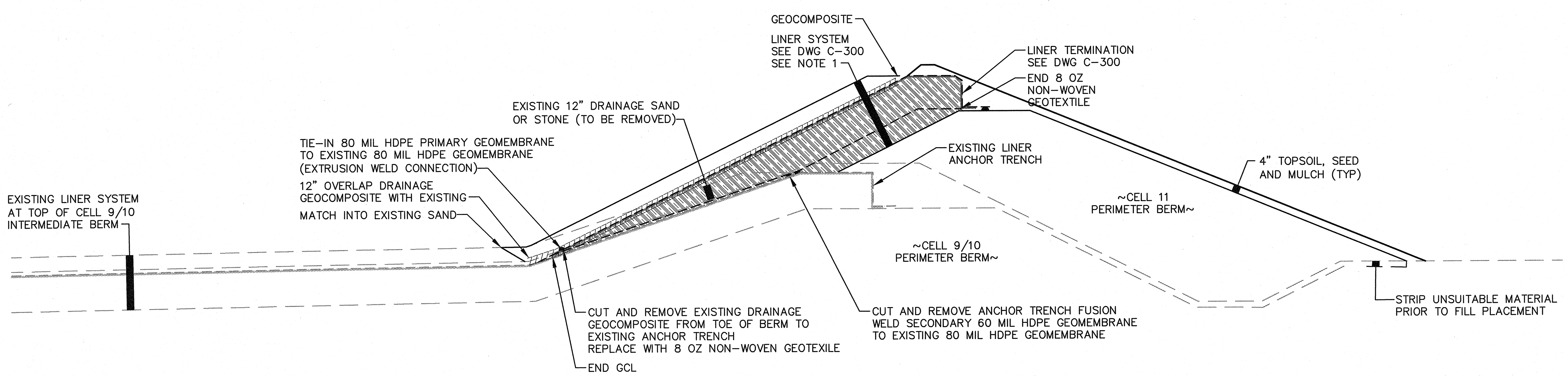
JOB NO. 14101.00 DWG FILE DETAILS

C-300



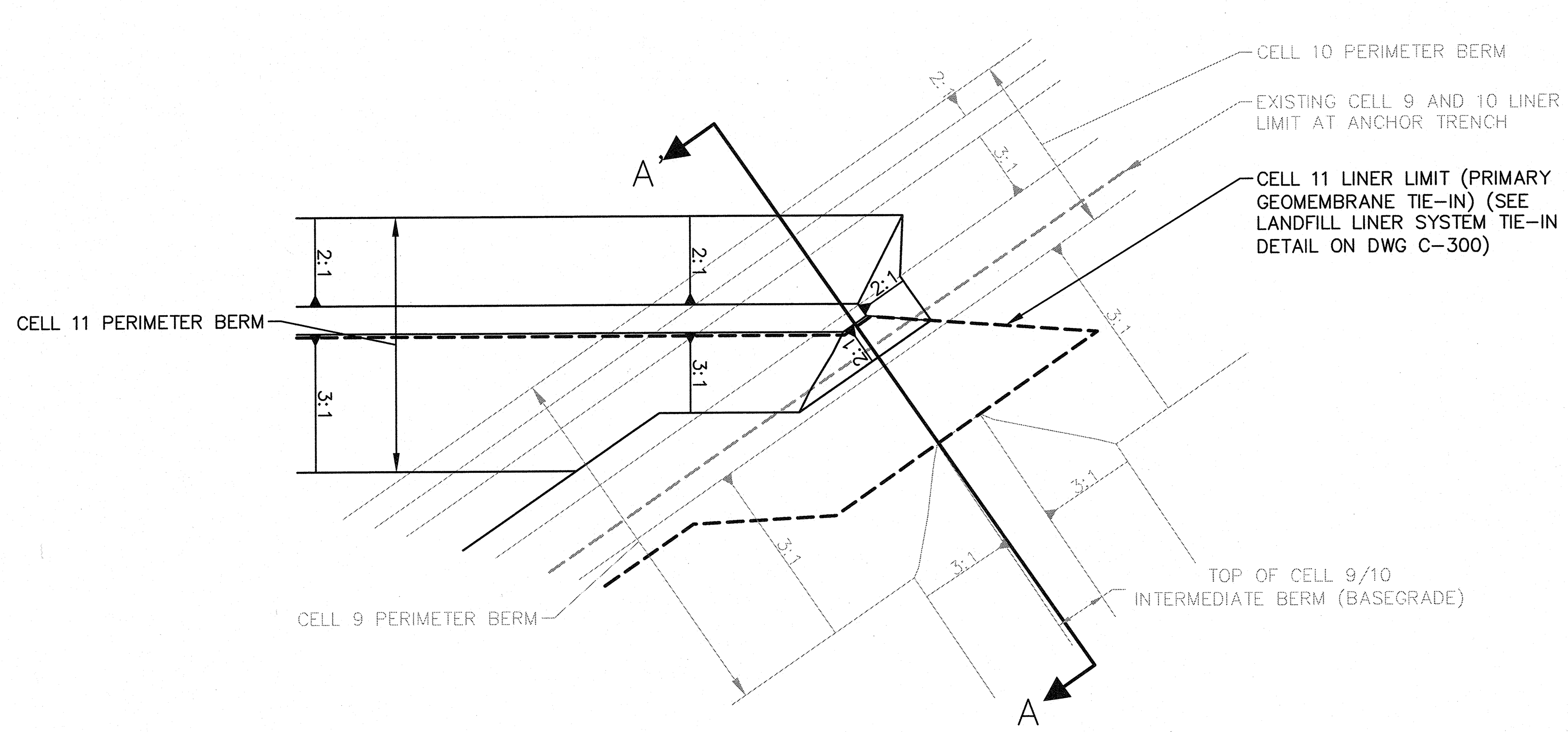
EAST SIDE PERIMETER BERM AND ACCESS ROAD (1)
NTS

C-102
C-103
C-104
C-105
C-107
C-108

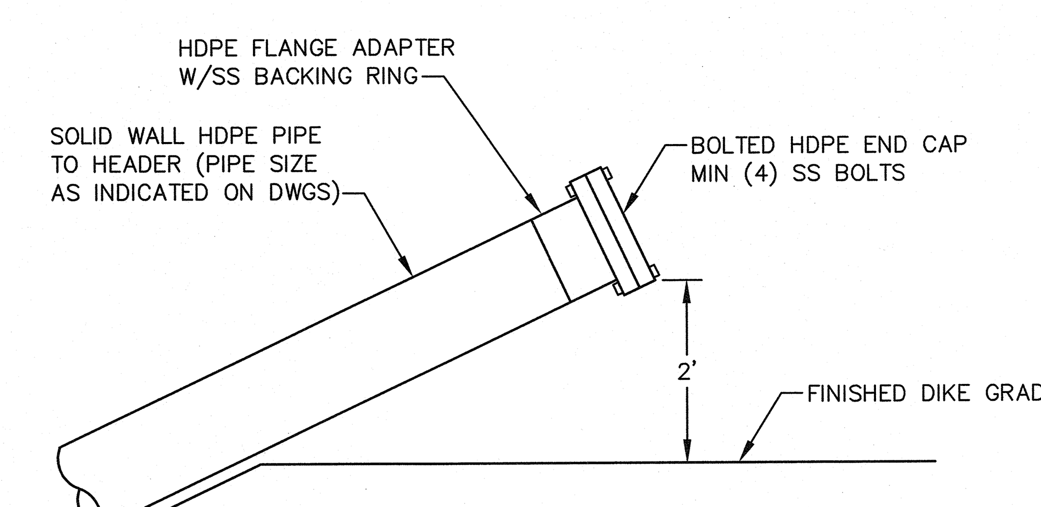


NOTE:
1. ON THE SIDESLOPE OF THE PERIMETER BERM THE LEAK DETECTION SAND LAYER IS REPLACED WITH 12-INCHES OF CLAY.

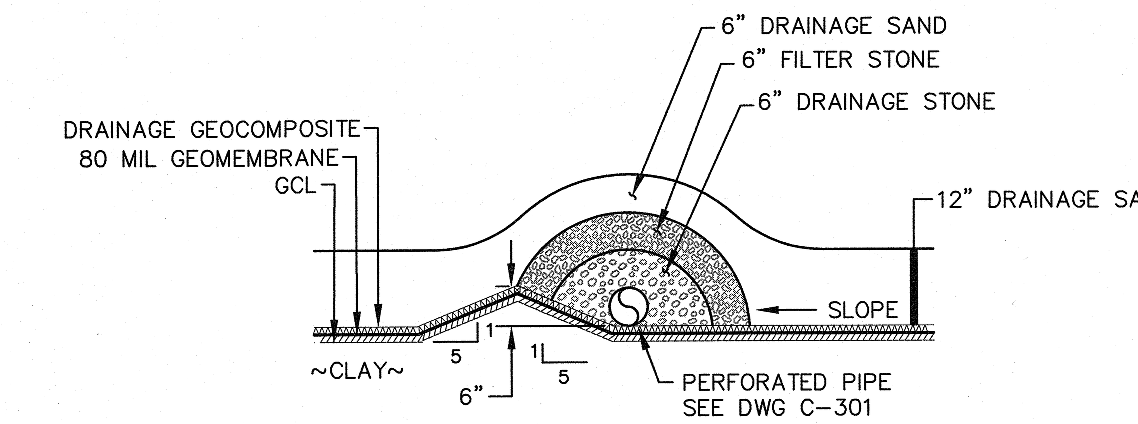
SECTION A–A'
NTS



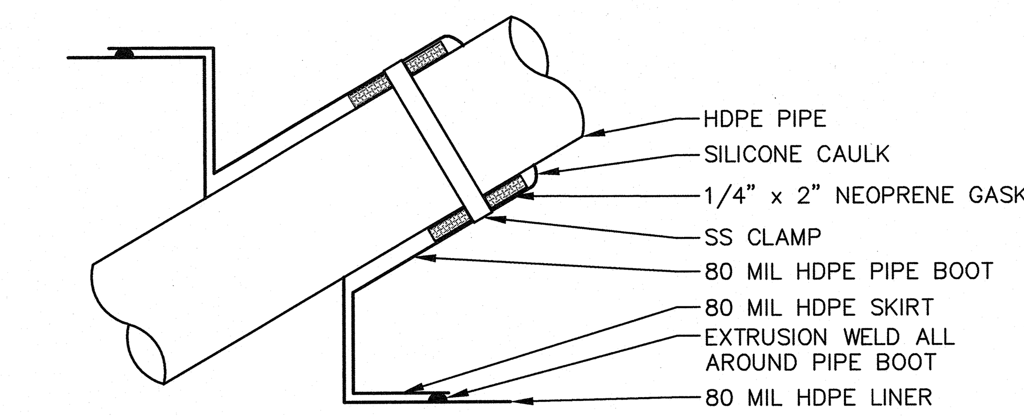
PERIMETER BERM TIE-IN AT SOUTH END OF CELL 11
NTS



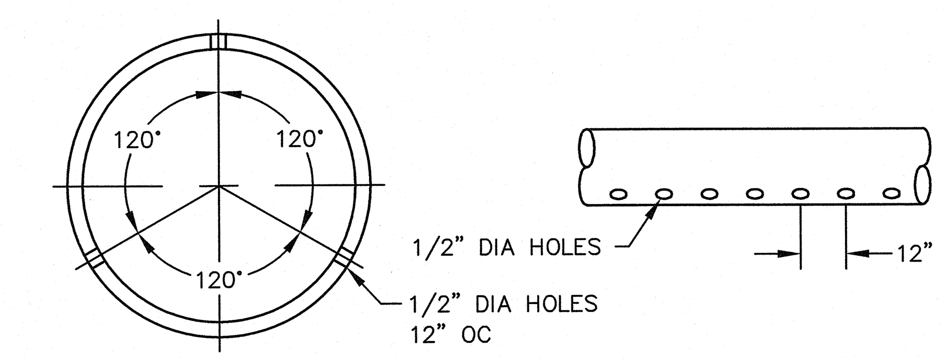
FLANGED END CAP
NTS



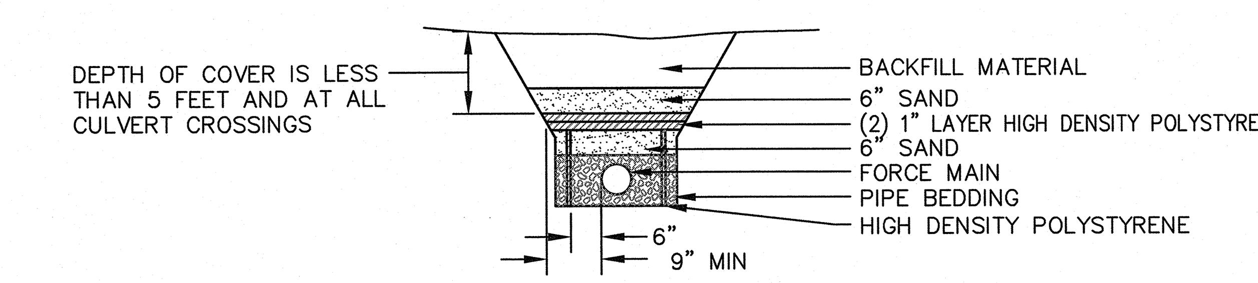
LEACHATE COLLECTION LATERAL (TYP)
NTS



PIPE BOOT
NTS



PERFORATED PIPE
NTS



PIPE INSULATION
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE ... LICENSED PROFESSIONAL ENGINEER

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

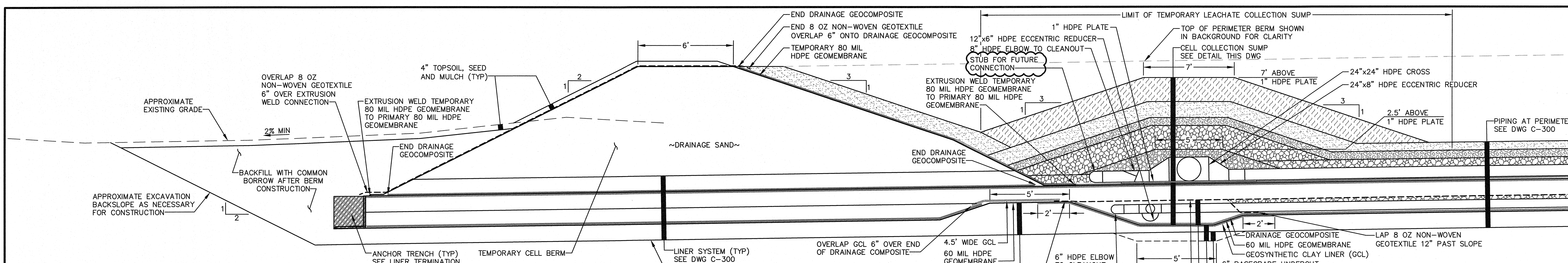
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	12/5/2014
CHECKED BY:	
LMN:	NONE
CTB:	SME-STD

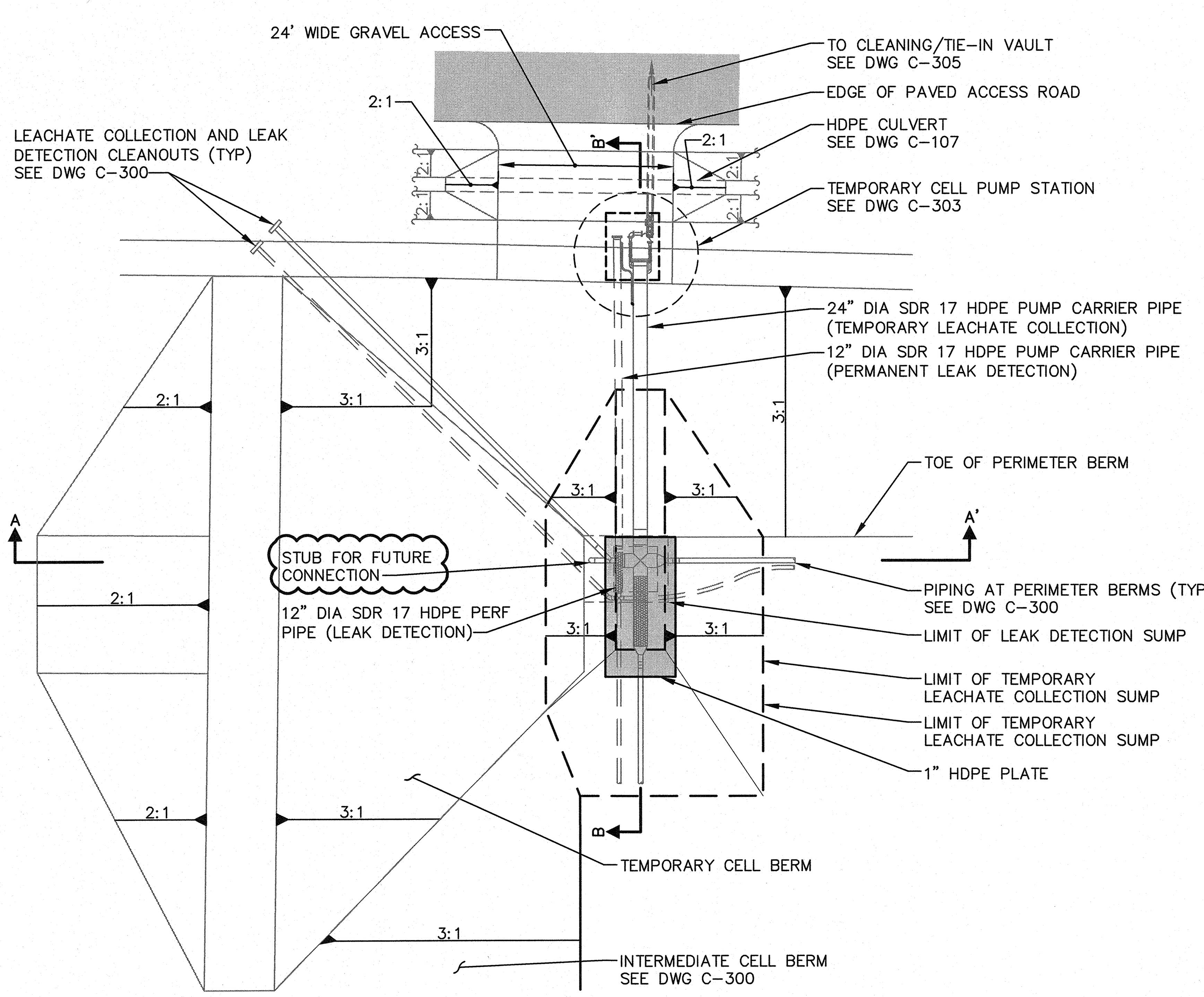
JOB NO. 14101.00 DWG FILE DETAILS

C-301

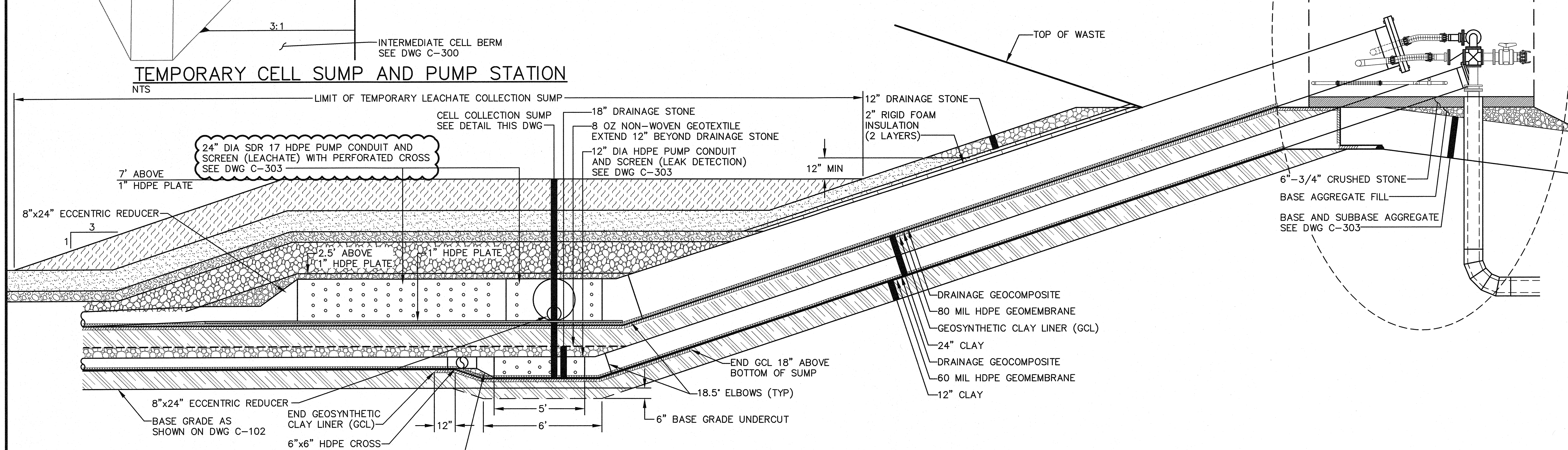


SECTION A-A'
NTS

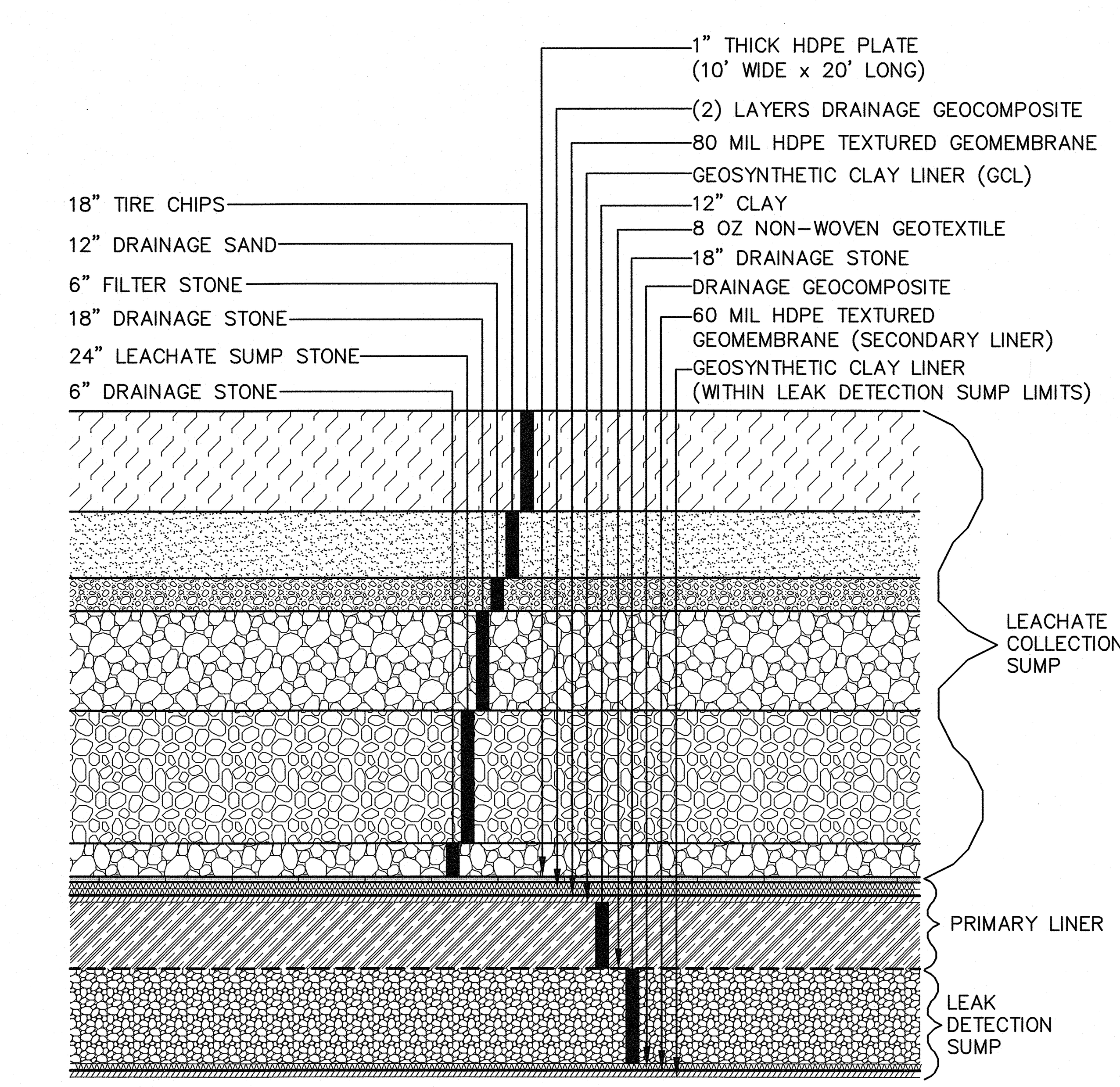
NOTES:
1. UNDERDRAIN AND AUGMENTED SECONDARY LINER ARE NOT SHOWN IN DETAIL. BASE GRADE SHALL BE UNDERCUT AN ADDITIONAL 1-FOOT IN THOSE LOCATIONS.
2. LIMIT OF LEACHATE COLLECTION SUMP IS DEFINED BY THE LIMIT OF THE 18" TIRE CHIPS LAYER



TEMPORARY CELL SUMP AND PUMP STATION
NTS



SECTION B-B'
NTS



CELL COLLECTION SUMP
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE — GUY COTE — No. 5… — LICENSED — PROFESSIONAL ENGINEER — 2/19/16

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

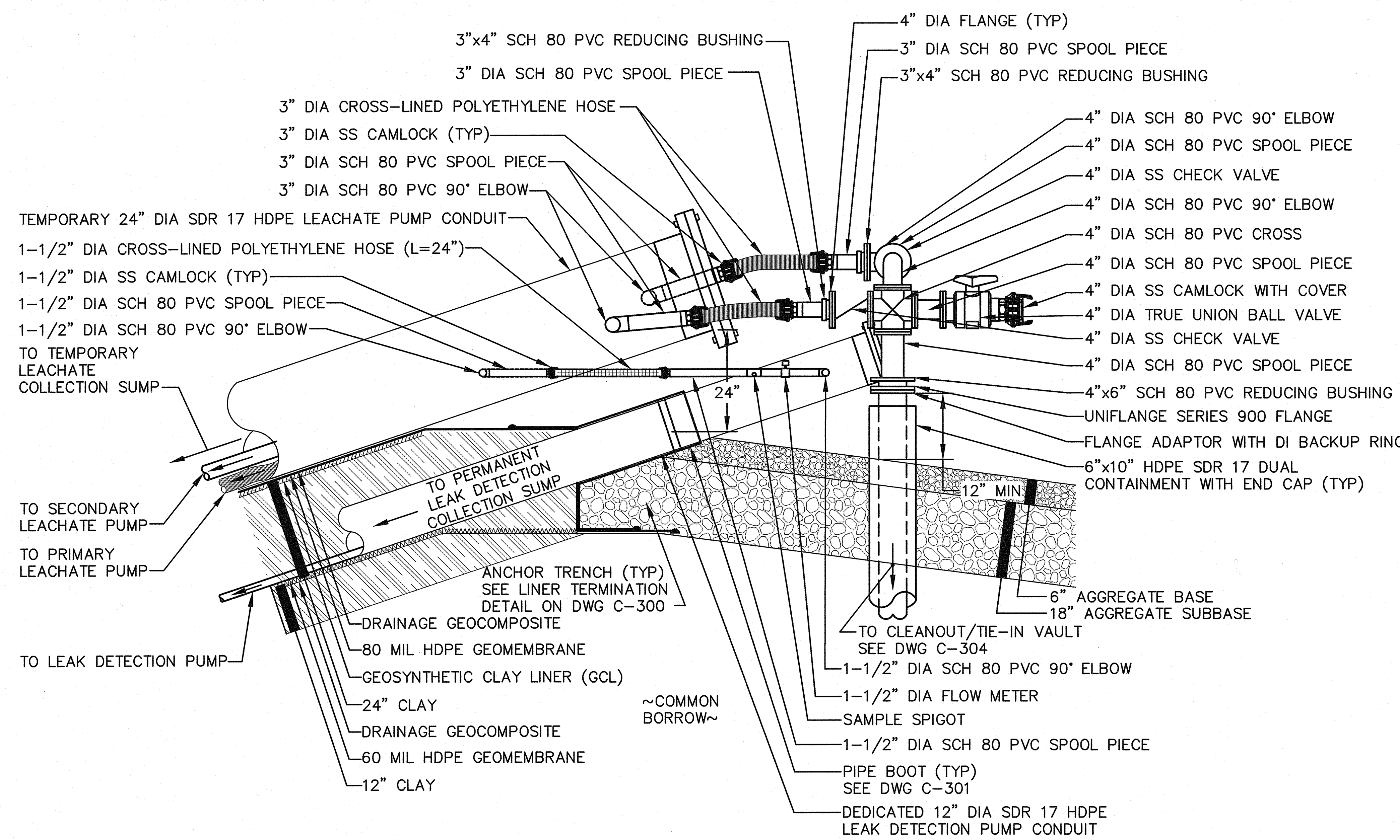
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

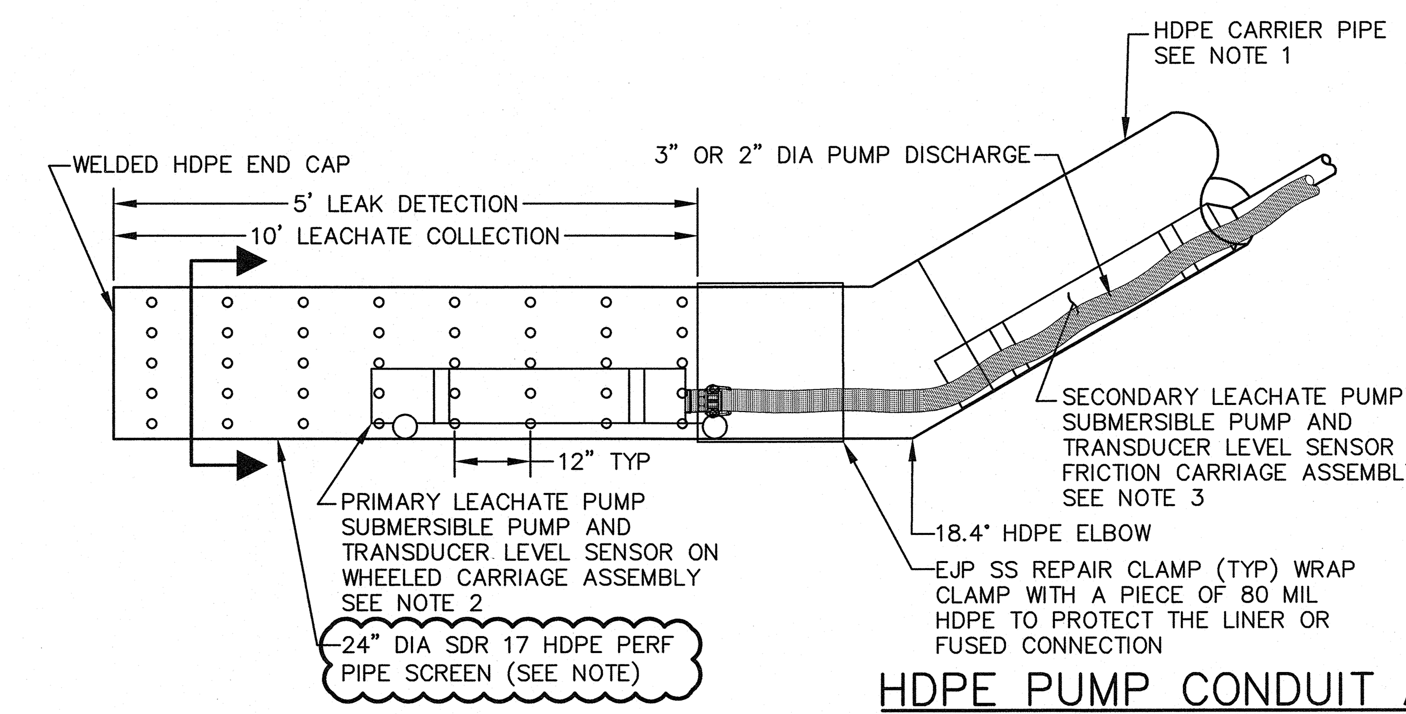
JOB NO. 14101.00 DWG FILE DETAILS

C-302

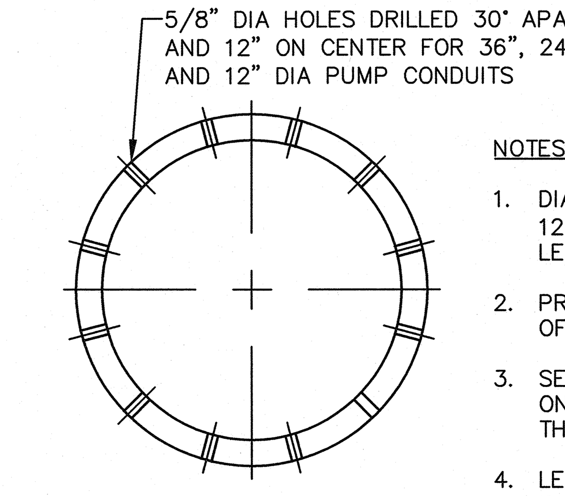


NOTE:
ALL LEACHATE COLLECTION PUMPS AND TRANSPORT PIPING SHOWN ARE TEMPORARY AND SHALL BE REMOVED OR ABANDONED AS SUBSEQUENT DOWNSTREAM CELL IS CONSTRUCTED. THE 24" PUMP CARRIER PIPE SHALL REMAIN IN PLACE AS A CLEANOUT. THE LEAK DETECTION PUMP AND PIPING IS PERMANENT AND SHALL REMAIN IN PLACE.

SECTION A-A'



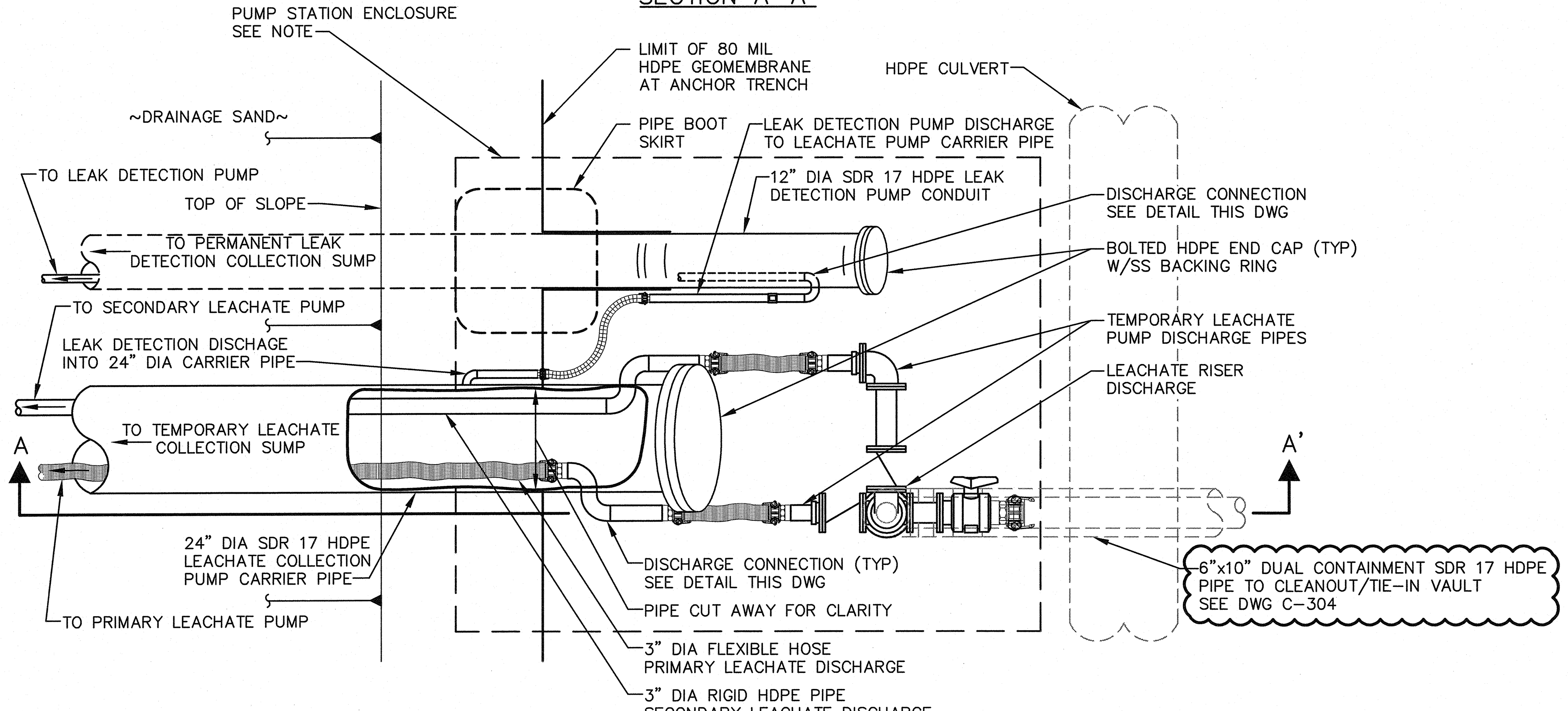
HDPE PUMP CONDUIT AND SCREEN
NTS



SECTION

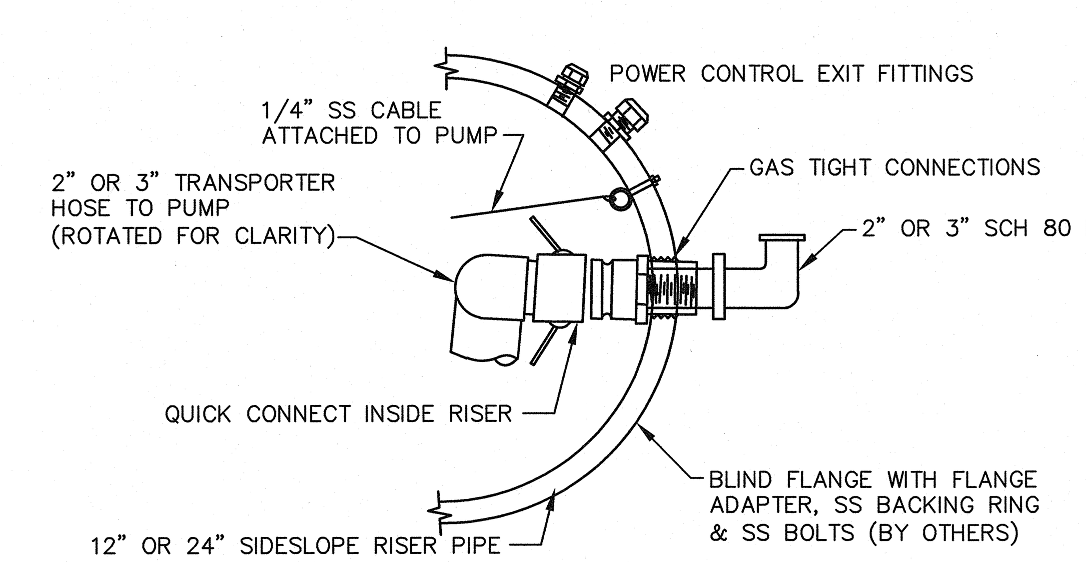
NOTES:

1. DIAMETER OF PUMP CONDUIT AND SCREEN IS EITHER 12" FOR LEAK DETECTION SYSTEM OR 24" FOR LEACHATE COLLECTION SYSTEM.
2. PRIMARY LEACHATE PUMP SHALL BE AT THE BOTTOM OF SUMP AND SIT HORIZONTALLY ON SUMP FLOOR.
3. SECONDARY LEACHATE PUMP SHALL BE POSITIONED ON THE INCLINED SIDESLOPE AND BE LOWERED TO THE TOW OF SLOPE.
4. LEAK DETECTION PUMP SHALL BE POSITIONED AT THE BOTTOM OF THE LEAK DETECTION SUMP AND SIT HORIZONTALLY ON SUMP FLOOR. LEAK DETECTION DISCHARGE HOSE IS 2-INCH DIAMETER.



NOTE:
THE PUMP STATION ENCLOSURE IS TO BE A WOOD FRAMED BUILDING MOUNTED TO A PRESSURE TREATED WOOD FRAMED SKID. THE CONTRACTOR SHALL SUBMIT SHOP DRAWINGS OF THE PUMP STATION ENCLOSURE TO THE ENGINEER AND OWNER FOR APPROVAL. THE PUMP STATION SHALL INCLUDE BUT NOT BE LIMITED TO, 2"x6" WOOD FRAMING, INSULATION, METAL ROOF, T-111 SIDING, ELECTRICAL LIGHTING, GFIC RECEPTACLE, INSULATED DOUBLE STEEL DOORS, PASSIVE VENTILATION AND GAS MONITORING SYSTEM.

TEMPORARY CELL PUMP STATION
NTS



DISCHARGE CONNECTION
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE LICENSED PROFESSIONAL ENGINEER No. 514 2/3/16

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

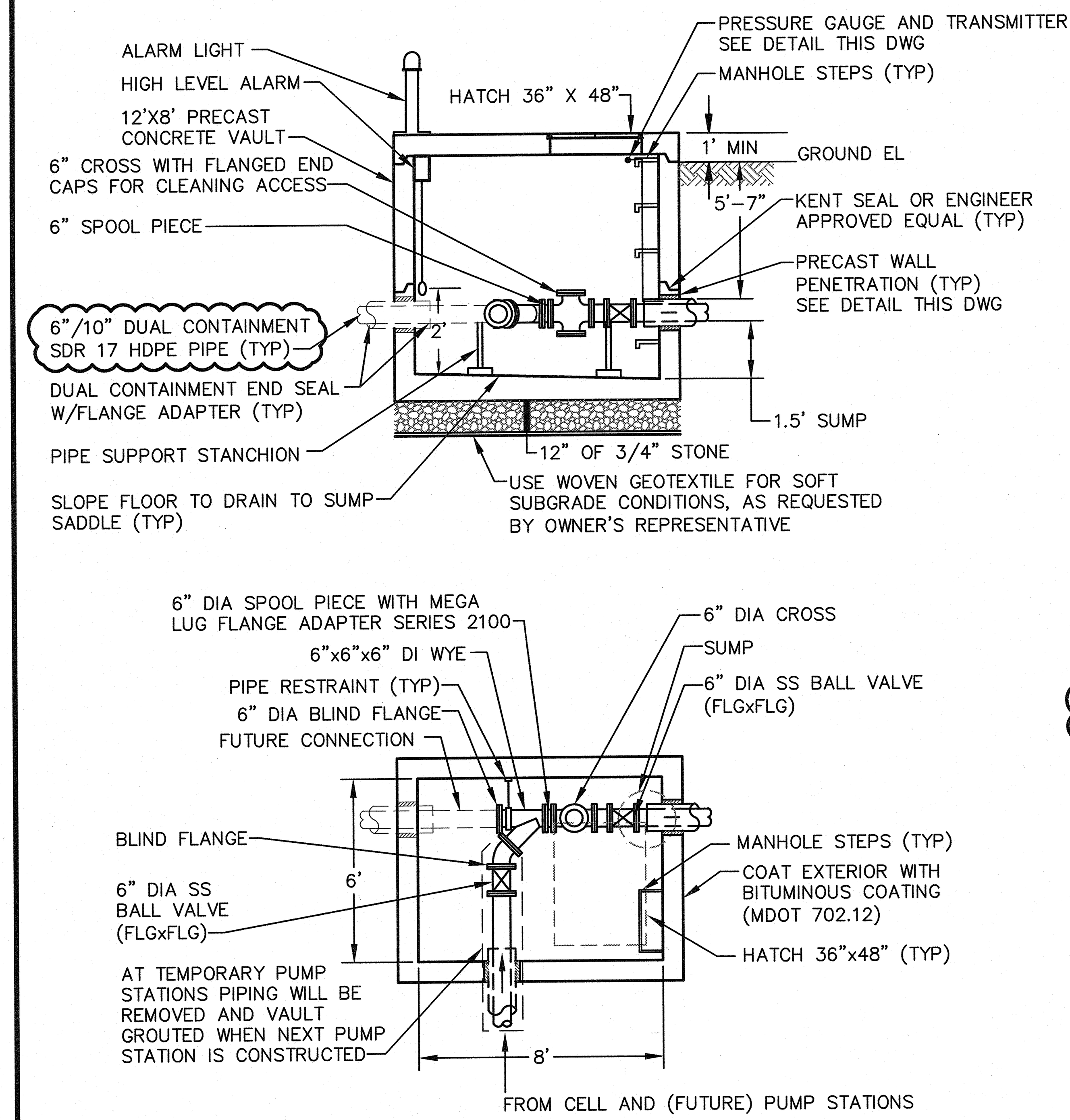
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

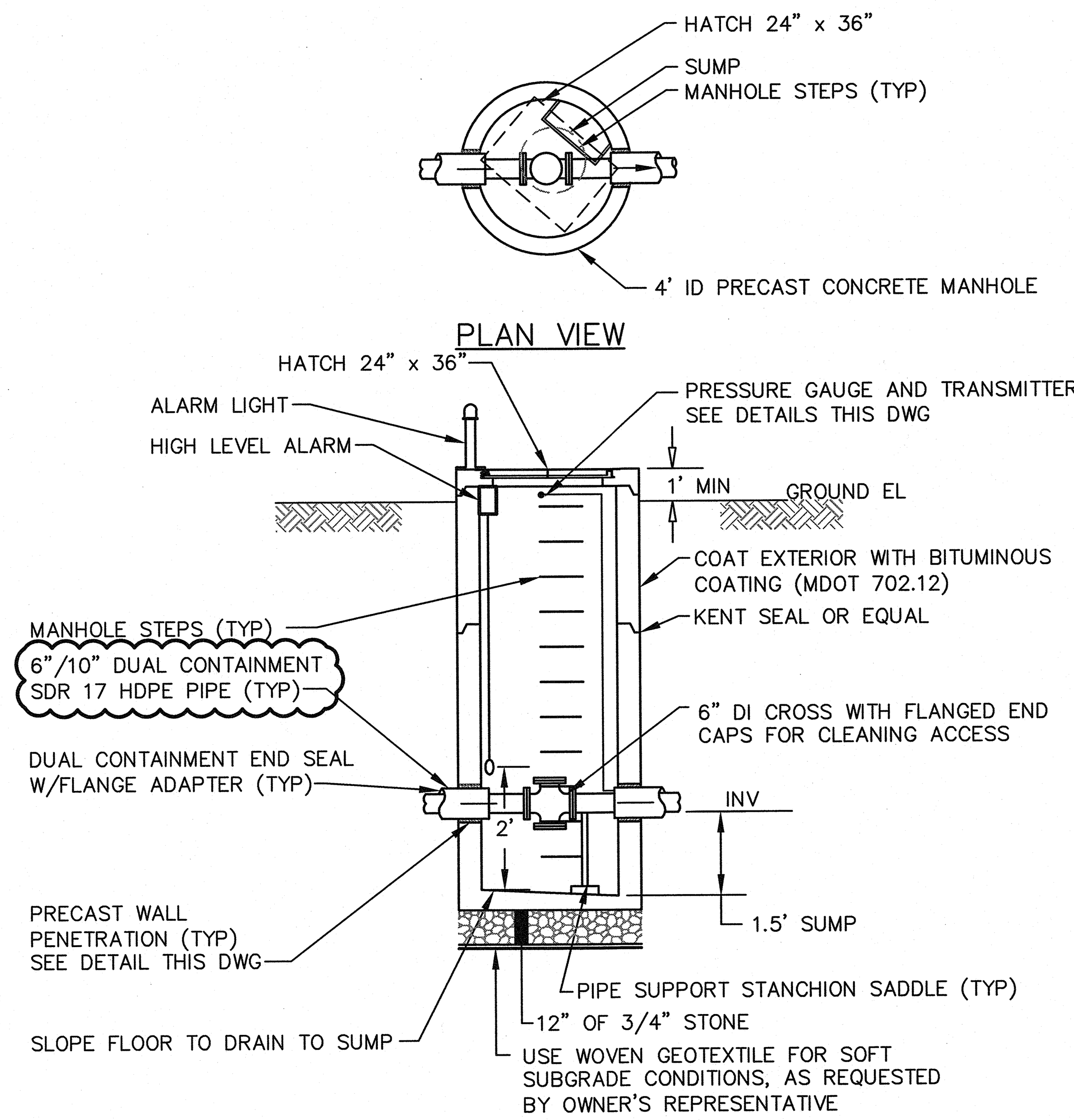
DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

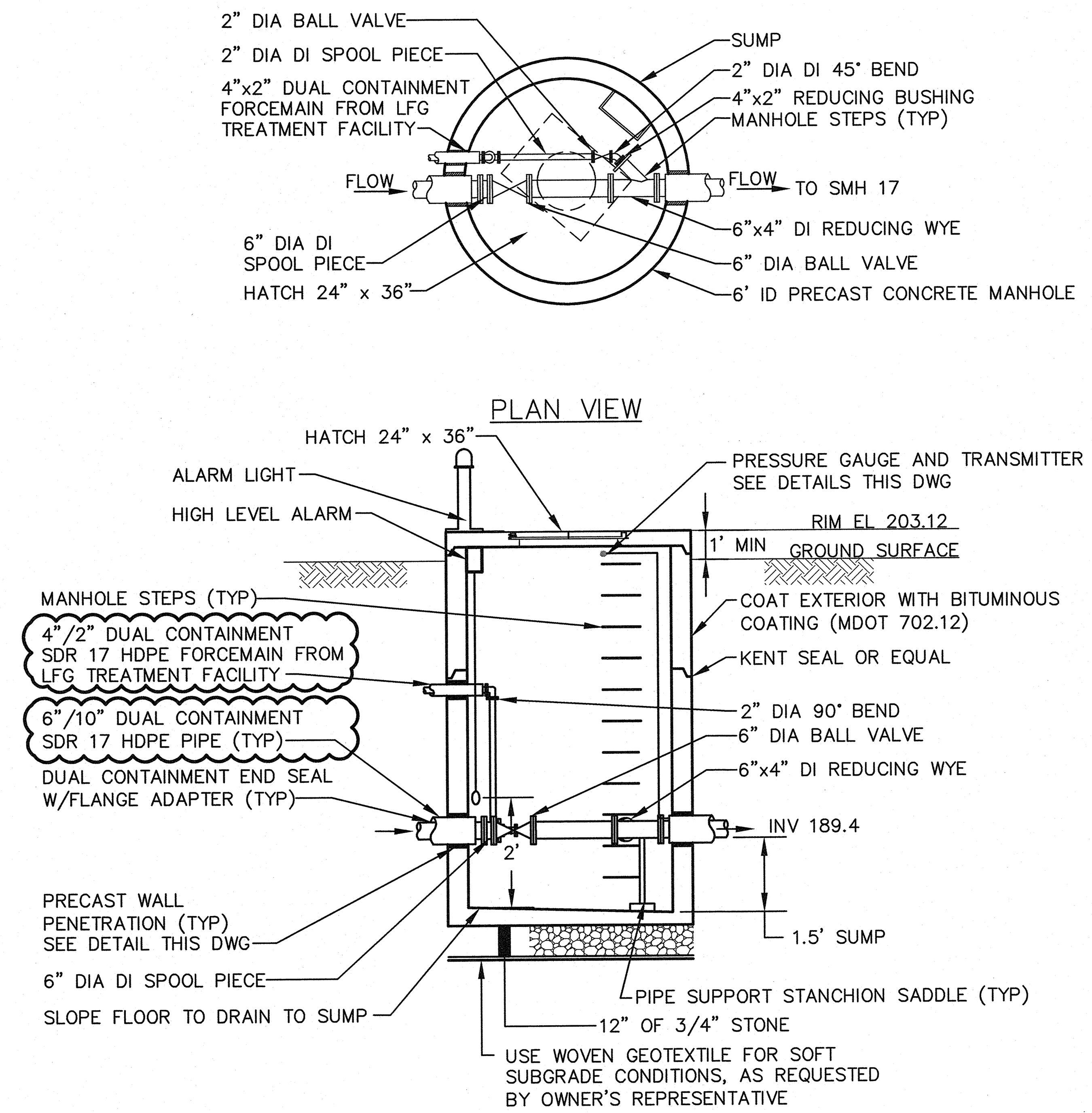
C-303



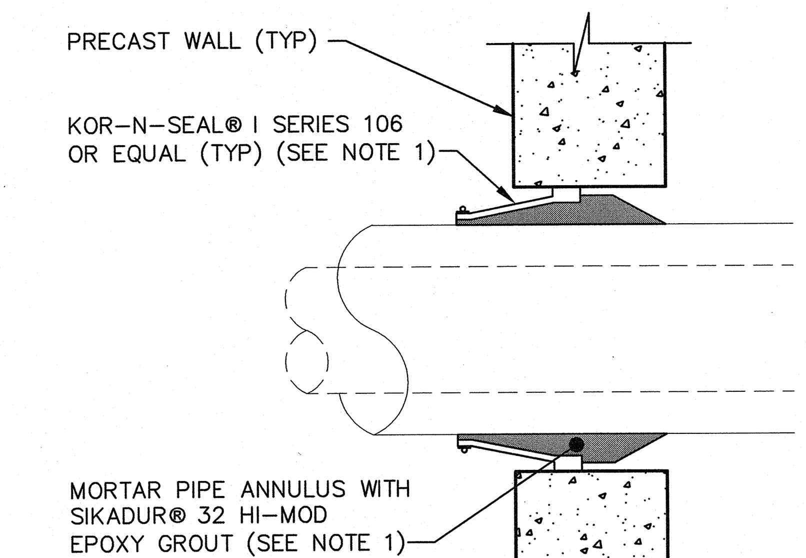
CLEANING/TIE-IN VAULT
NTS



CLEANING MANHOLE
NTS



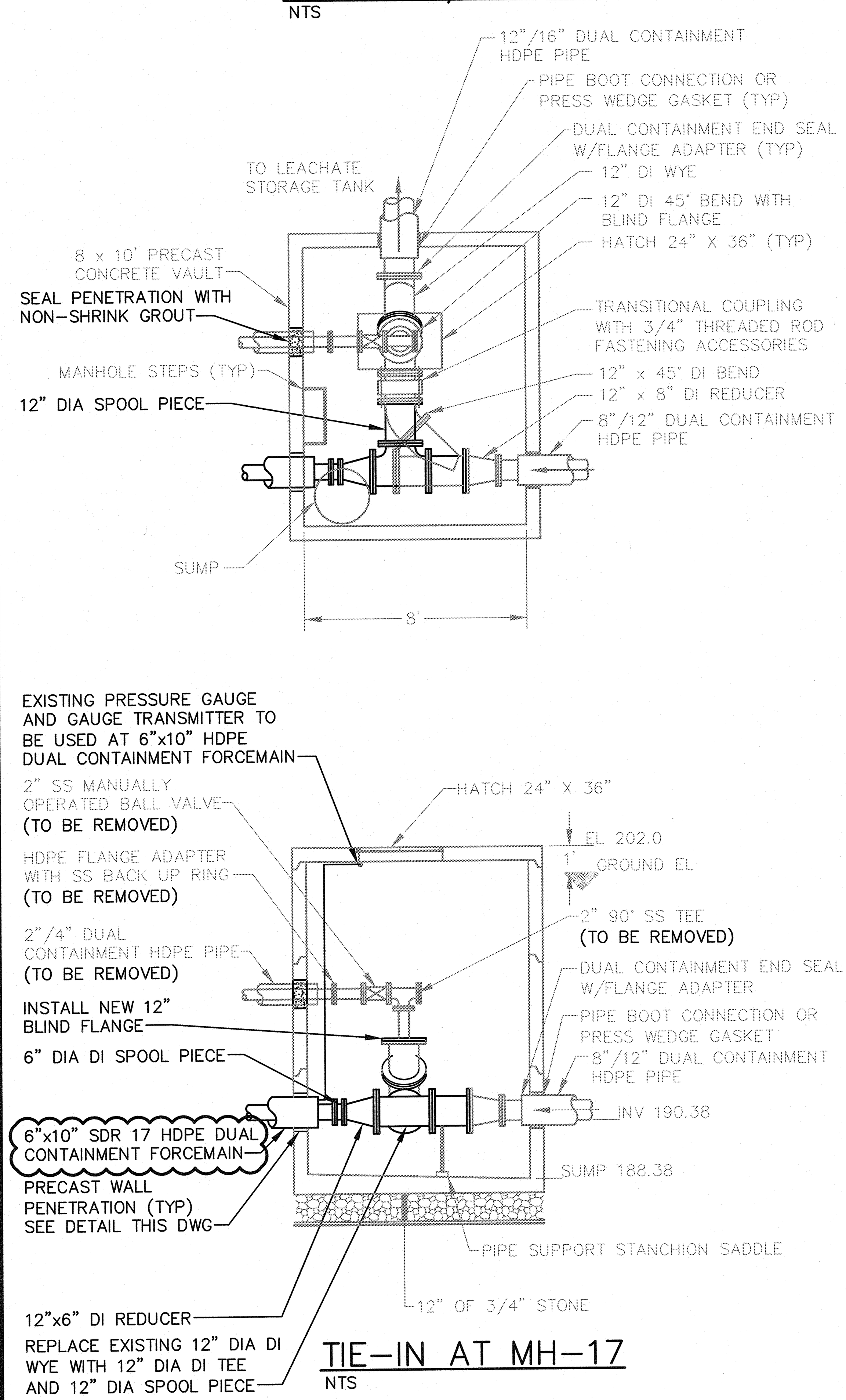
FORCEMAIN TIE-IN AT LEACHATE STORAGE TANK
NTS



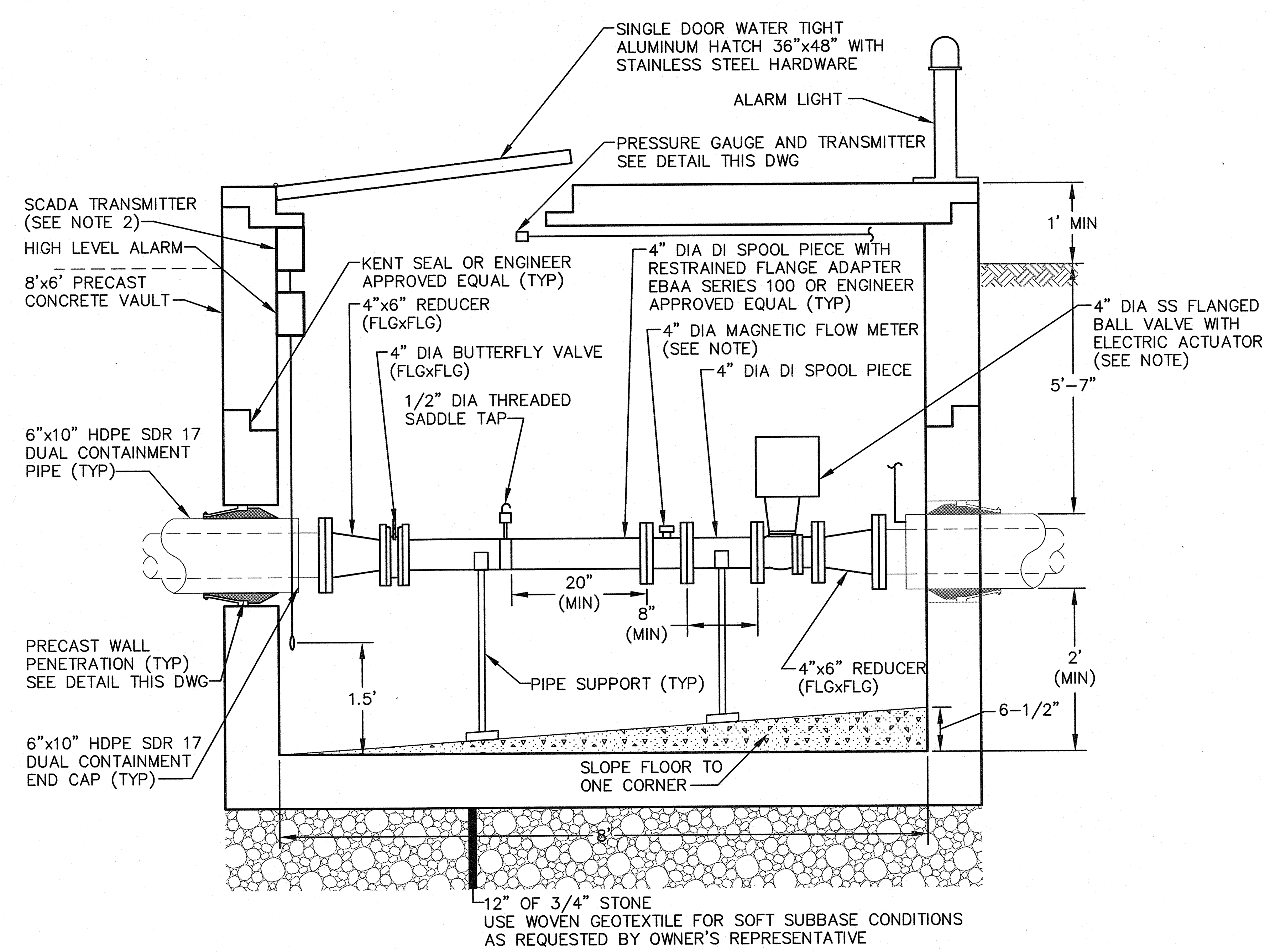
NOTE:

THE JOINTS FOR THE PIPES THROUGH PRECAST WALLS SHALL BE WEDGE STYLE KOR-N-SEAL®I SERIES 106 AS MANUFACTURED BY TRELLEBORG OR EQUAL. THE ANNULUS SHALL BE MORTARED WITH SIKADUR®32 HI-MOD GROUT AS MANUFACTURED BY SIKA CORPORATION. THE CONTRACTOR SHALL FOLLOW THE MANUFACTURERS INSTRUCTIONS TO MIX AND APPLY THE EPOXY GROUT. THE FINAL MORTARED PLUG SHALL YIELD A STRONG AND RIGID SEAL AROUND THE PIPE. COMPLETE VACUUM TESTING PRIOR TO THE INSTALLATION OF GROUT.

PRECAST WALL PENETRATION
NTS



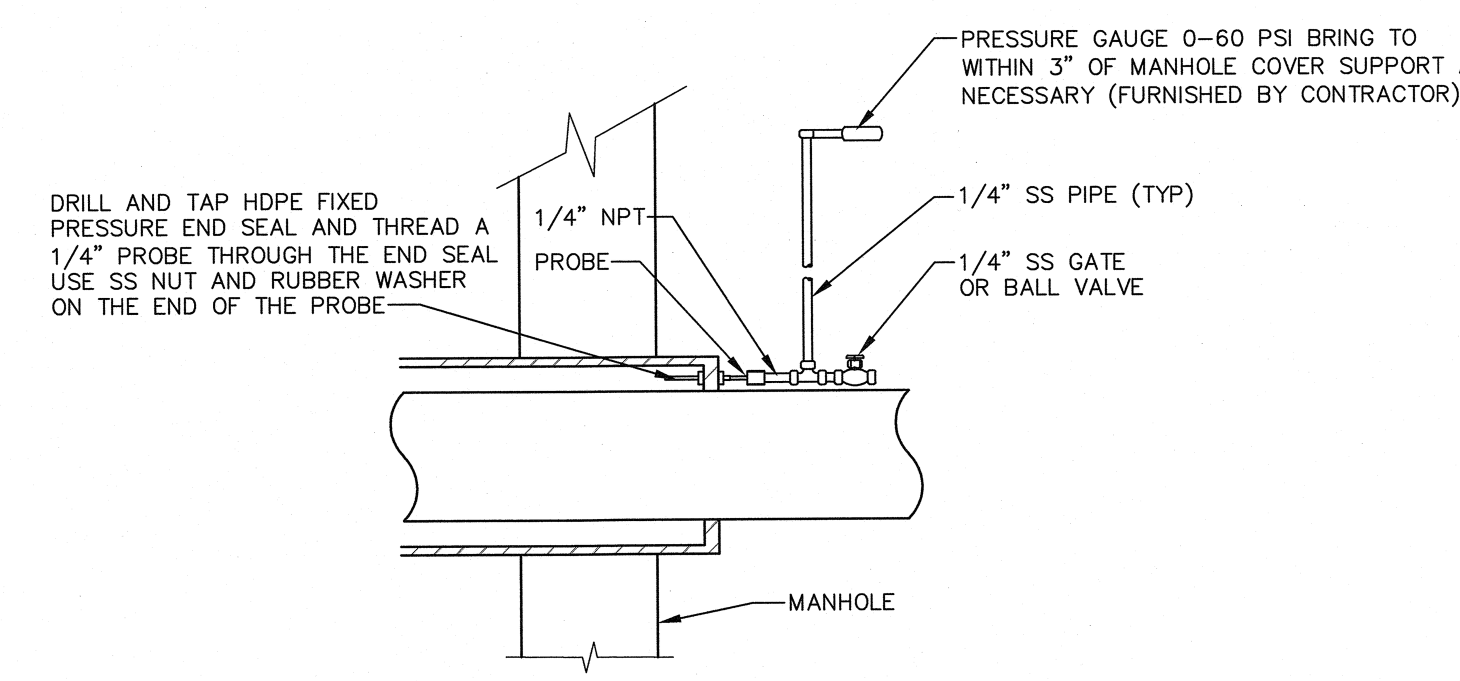
TIE-IN AT MH-17
NTS



NOTE:

FLOW METER AND BALL VALVE WITH ELECTRIC ACTUATOR SHALL BE CONNECTED TO SCADA TRANSMITTER.

FORCE MAIN METER AND VALVE PIT
NTS



PRESSURE PROBE AND GAUGE
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

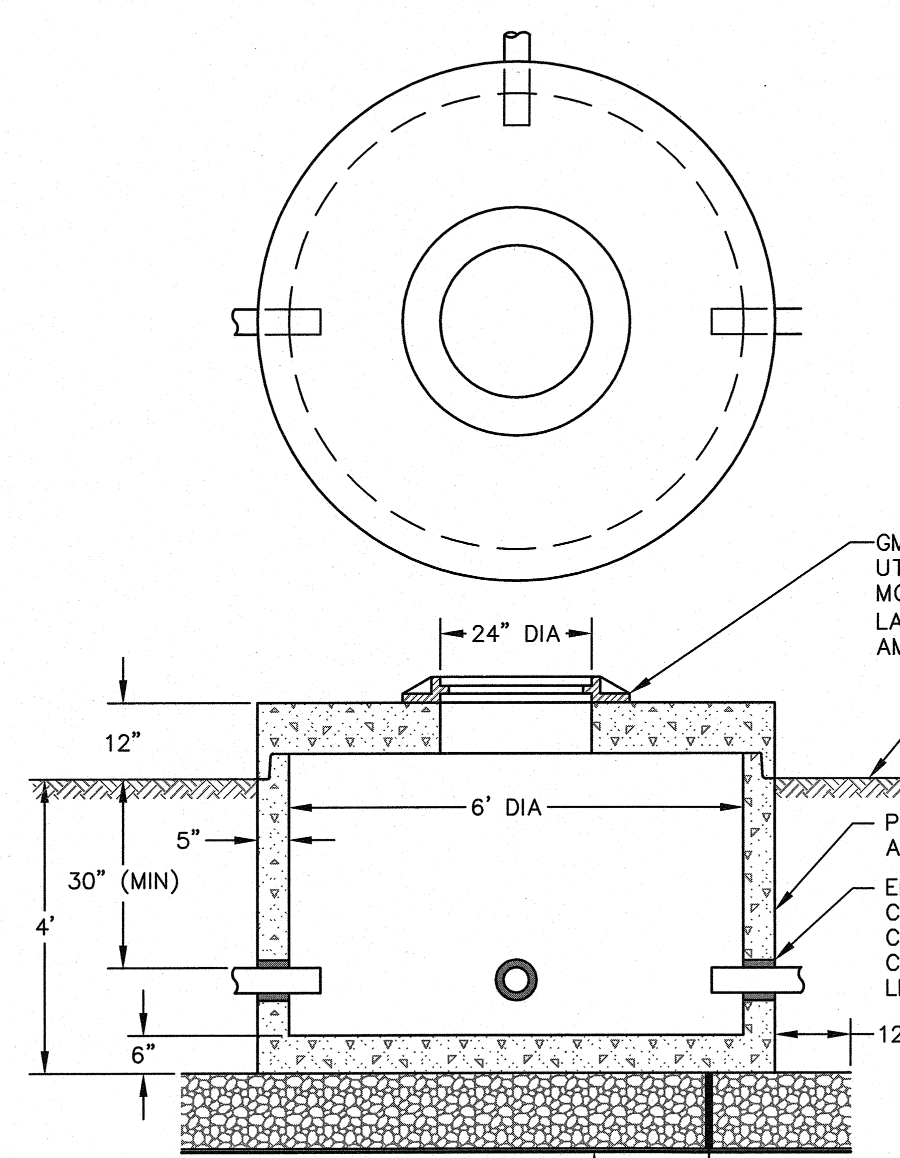
CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

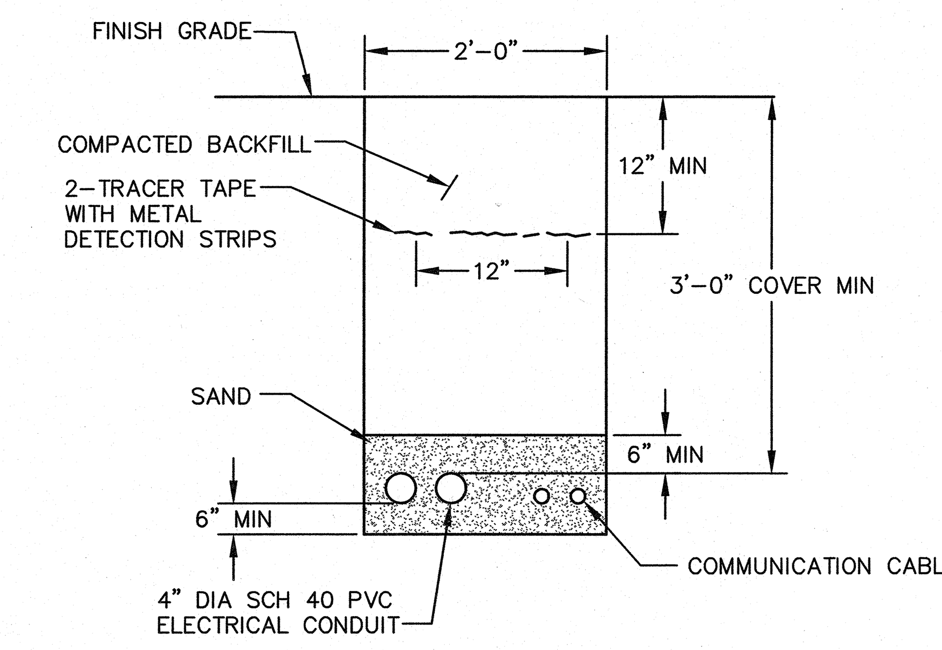
JOB NO. 14101.00 DWG FILE DETAILS
C-304



NOTES:

1. 6' DIA MANHOLE AS MANUFACTURED BY AMERICAN CONCRETE INDUSTRIES OR ENGINEER APPROVED EQUAL.
2. 4000 PSI CONCRETE AT 28 DAYS.
3. DESIGNED FOR H-20 WHEEL LOADING.
4. CONFORMS TO ASTM C-478 SPECIFICATIONS.
5. REINFORCED TO 0.12 IN SQ/LF.
6. SHIPLAP JOINTS SEALED WITH BUTYL RUBBER.
7. EXTERIOR COATED WITH ASPHALTIC PROTECTIVE DAMPROOFING.
8. BOTTOM MIN 5'-0" BELOW FINISH GRADE.
9. PRECAST CONCRETE VAULT MANUFACTURER TO PROVIDE ANTI-FLOATATION EXTENDED BASE SLAB AS NECESSARY. ANTI-FLOATATION DESIGN AND SHOP DRAWINGS SHALL BE PREPARED BY THE MANUFACTURER AND SUBMITTED TO THE ENGINEER FOR APPROVAL.

ELECTRIC UTILITY MANHOLE
NTS

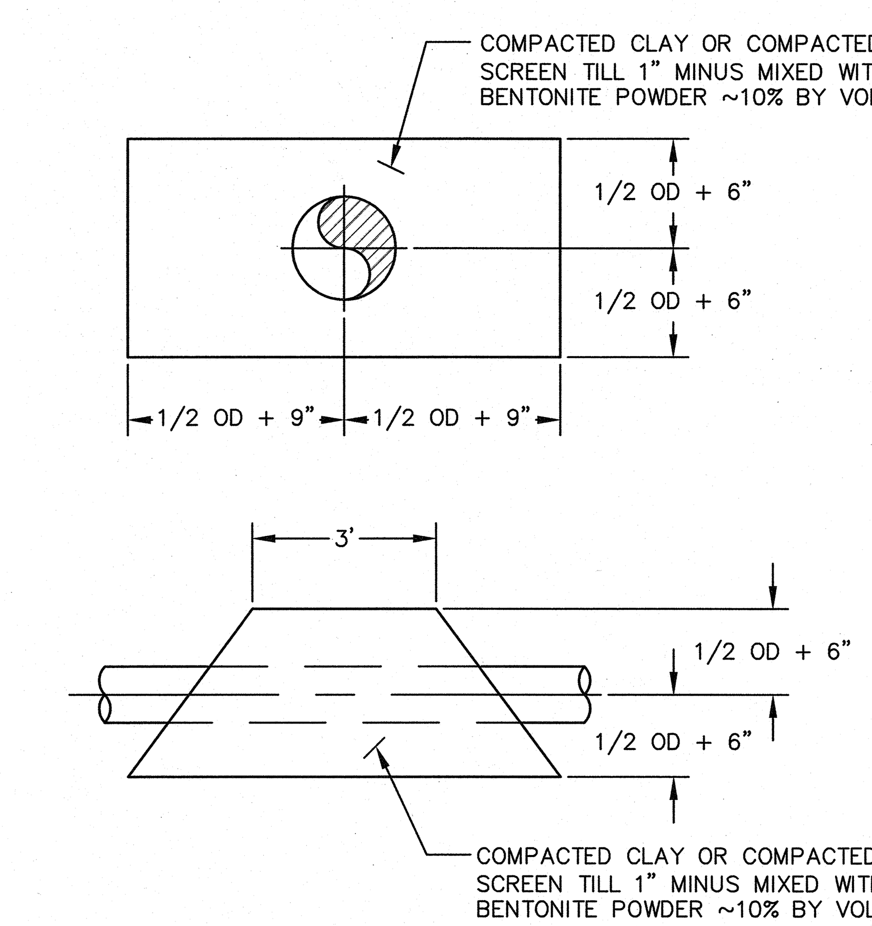


UNDERGROUND ELECTRIC AND COMMUNICATION TRENCH
NTS

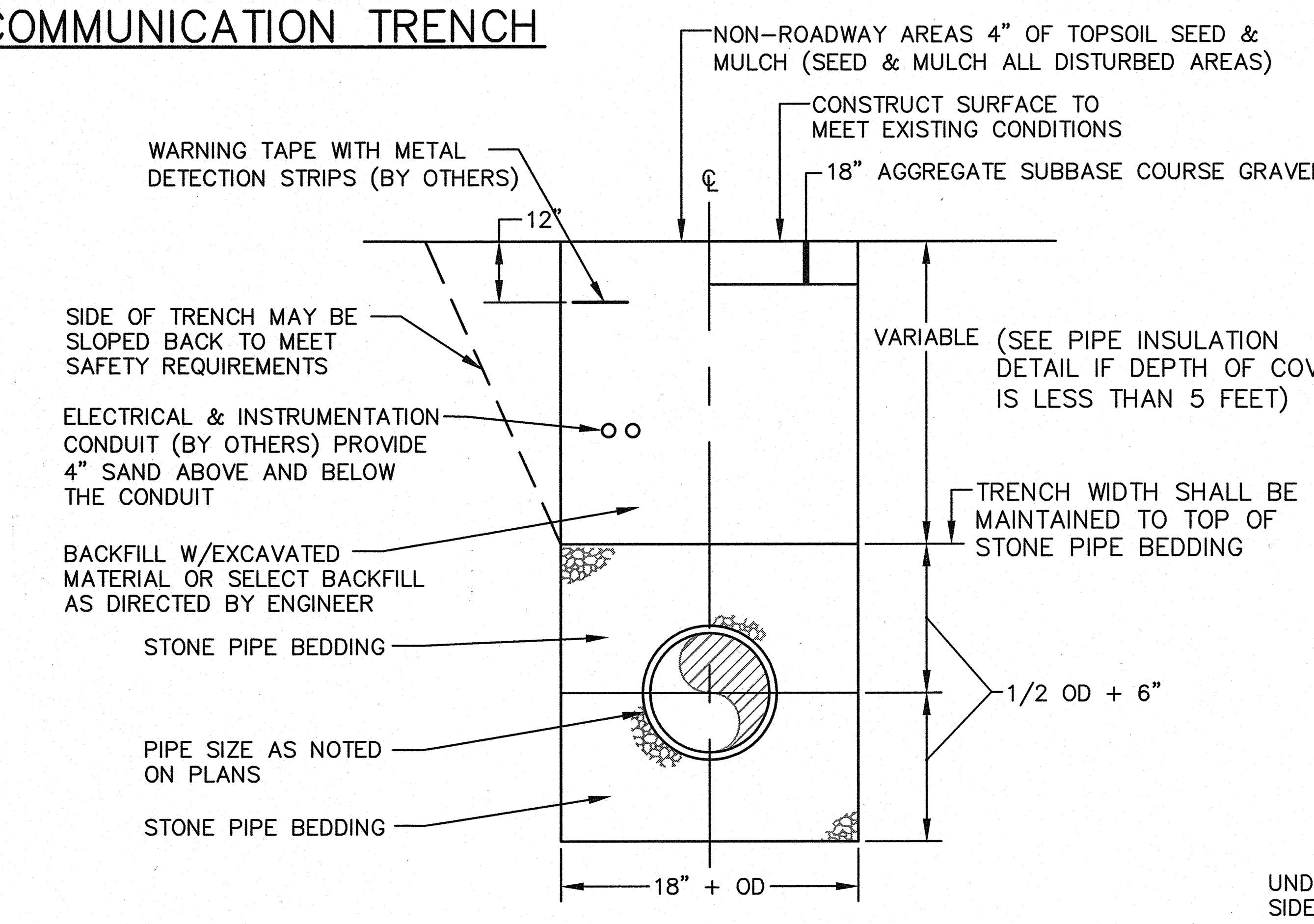
CULVERTS	DIAMETER (IN)	LENGTH (FT)	SLOPE (FT/FT)	INV IN (FT)	INV OUT (FT)
C-4BA	24	78	0.01	204.4	203.7
C-4BB	24	78	0.01	204.4	203.7
C-11-T1	18	90	0.07	198.0	192.0
C-11-T2	24	50	0.06	192.0	189.0
C-4I	18	80	0.13	202.5	192.0
C-4IA	18	40	0.02	212.9	212.2
C-4JA	18	60	0.03	214.0	212.3
C-4JB	24	73	0.02	211.5	210.0
C-4JC	24	73	0.02	211.5	210.0
C-4K	24	51	0.04	216.5	214.3
C-4L	18	121	0.02	213.0	211.0

CULVERT SCHEDULE
NTS

NOTE: DAMS SPACED AT 150' ALONG FORCE MAIN AND AT LOCATION SHOWN ON PLAN DRAWINGS

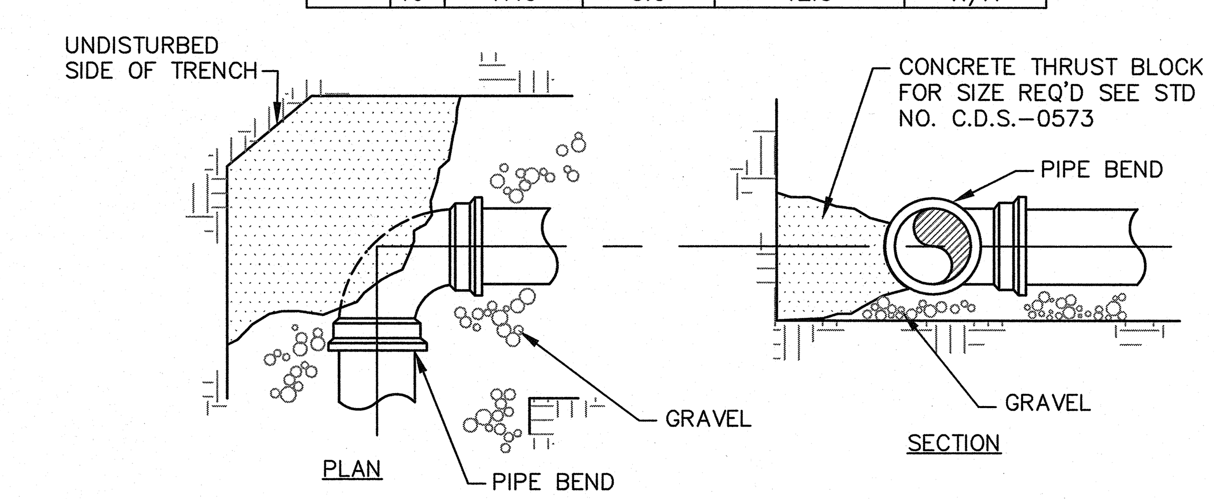


PIPE DAM
NTS



PIPE TRENCH
NTS

SIZE REQUIREMENTS FOR CONCRETE THRUST BLOCKS					
	BEARING ON UNDISTURBED SOIL (SQ FT)				
FITTINGS	90° BENDS	45° BENDS	TEES & PLUGS	HYDRANTS	
PIPE SIZE 4"	2.0	1.0	1.0	N/A	
6"	3.0	2.0	2.0	6.0	
8"	5.0	3.0	4.0	N/A	
10"	7.0	4.0	5.0	N/A	
12"	10.0	6.0	7.0	N/A	
14"	13.0	7.0	10.0	N/A	
16"	17.0	9.0	12.0	N/A	



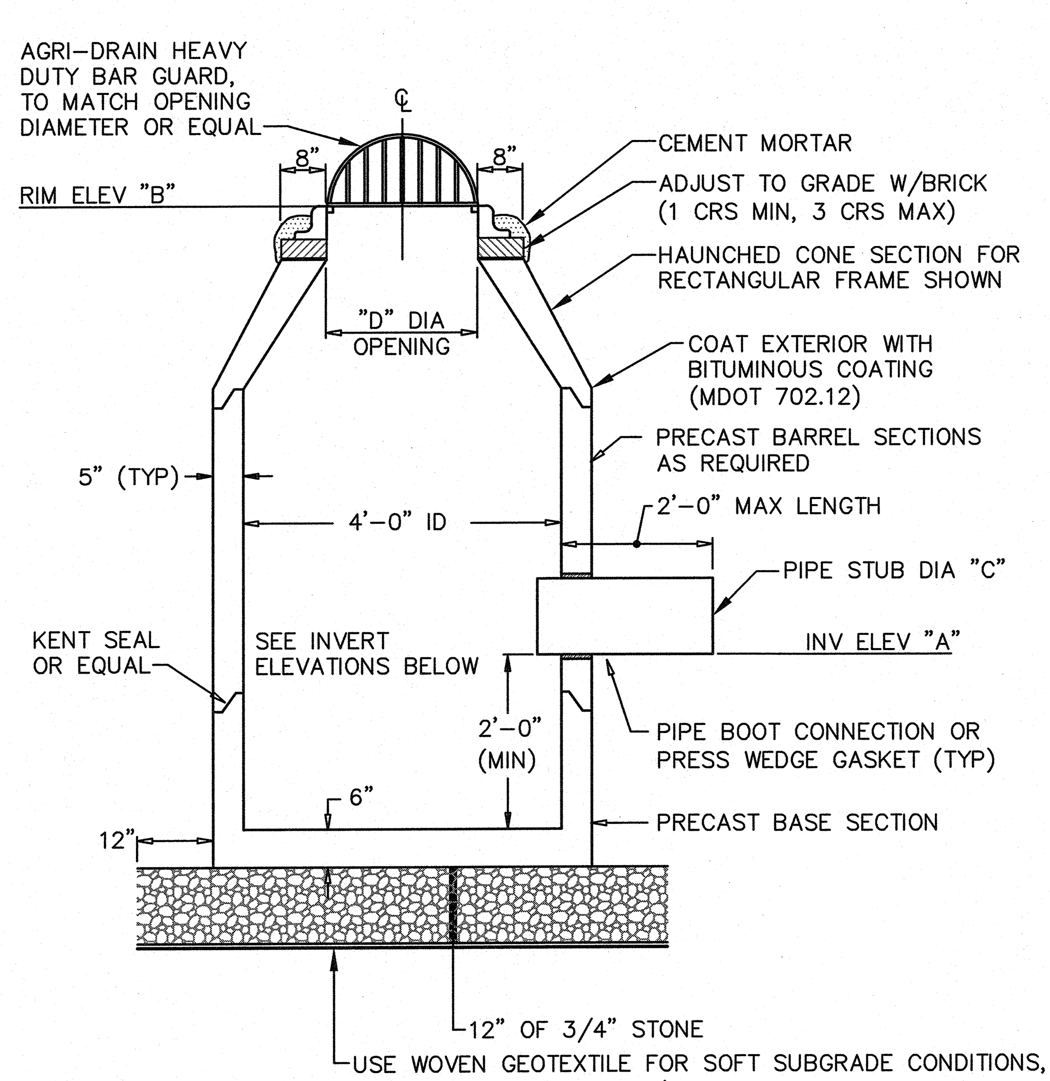
TYPICAL THRUST BLOCK PLACEMENT ON BENDS
NTS

CATCH BASIN SCHEDULE A

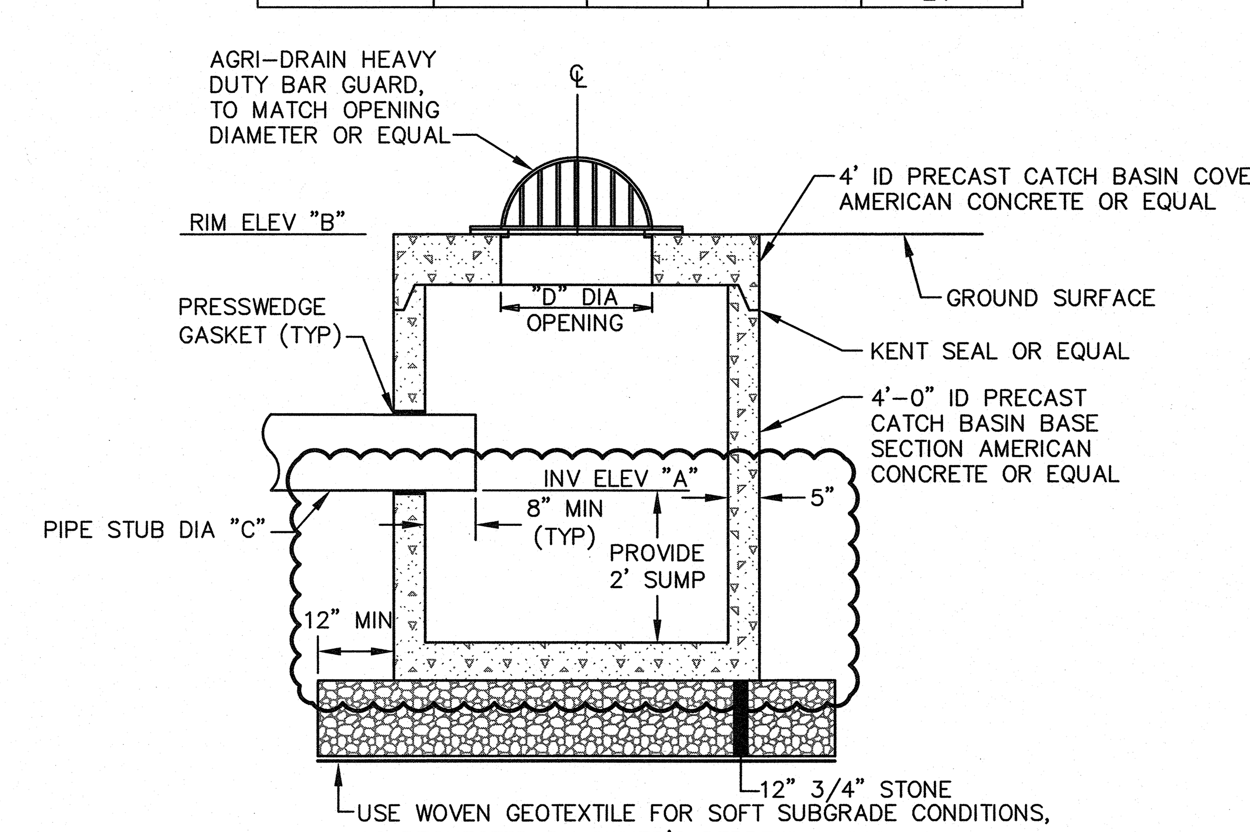
CATCH BASIN DESIGNATION	PIPE INV EL "A" (FT)	RIM EL "B" (FT)	PIPE DIA "C" (IN)	TOP OPENING DIA "D" (IN)
CB-4I	202.5	207.6	18"	24"
CB-4JA	214.0	218.7	18"	30"

CATCH BASIN SCHEDULE B

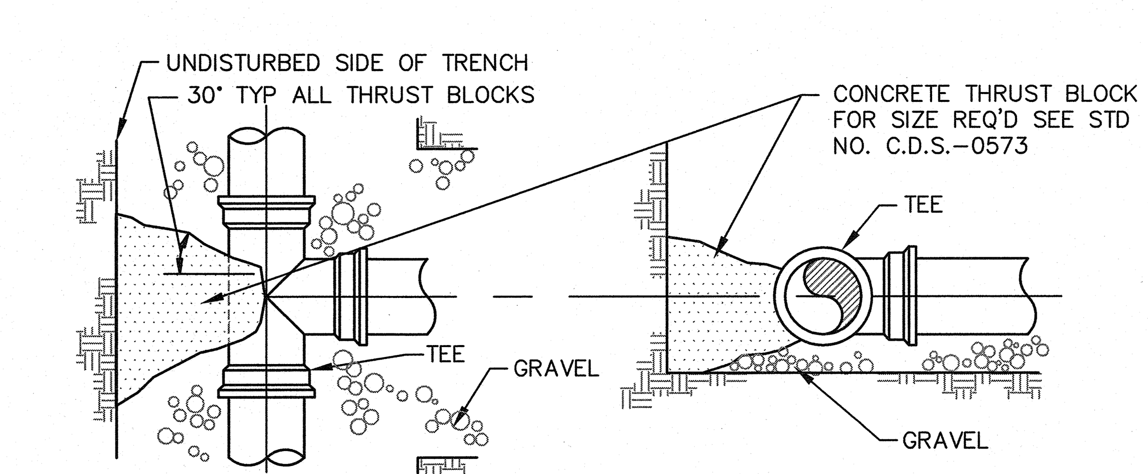
CATCH BASIN DESIGNATION	PIPE INV EL "A" (FT)	RIM EL "B" (FT)	PIPE DIA "C" (IN)	TOP OPENING DIA "D" (IN)
CB-4K	216.5	220.0	24"	30"
CB-4L	213.0	215.0	18"	24"



CATCH BASINS 4I AND 4JA
NTS

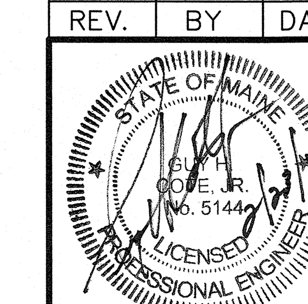


CATCH BASINS 4K & 4L
NTS



TYPICAL THRUST BLOCK PLACEMENT ON TEES
NTS

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION



CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

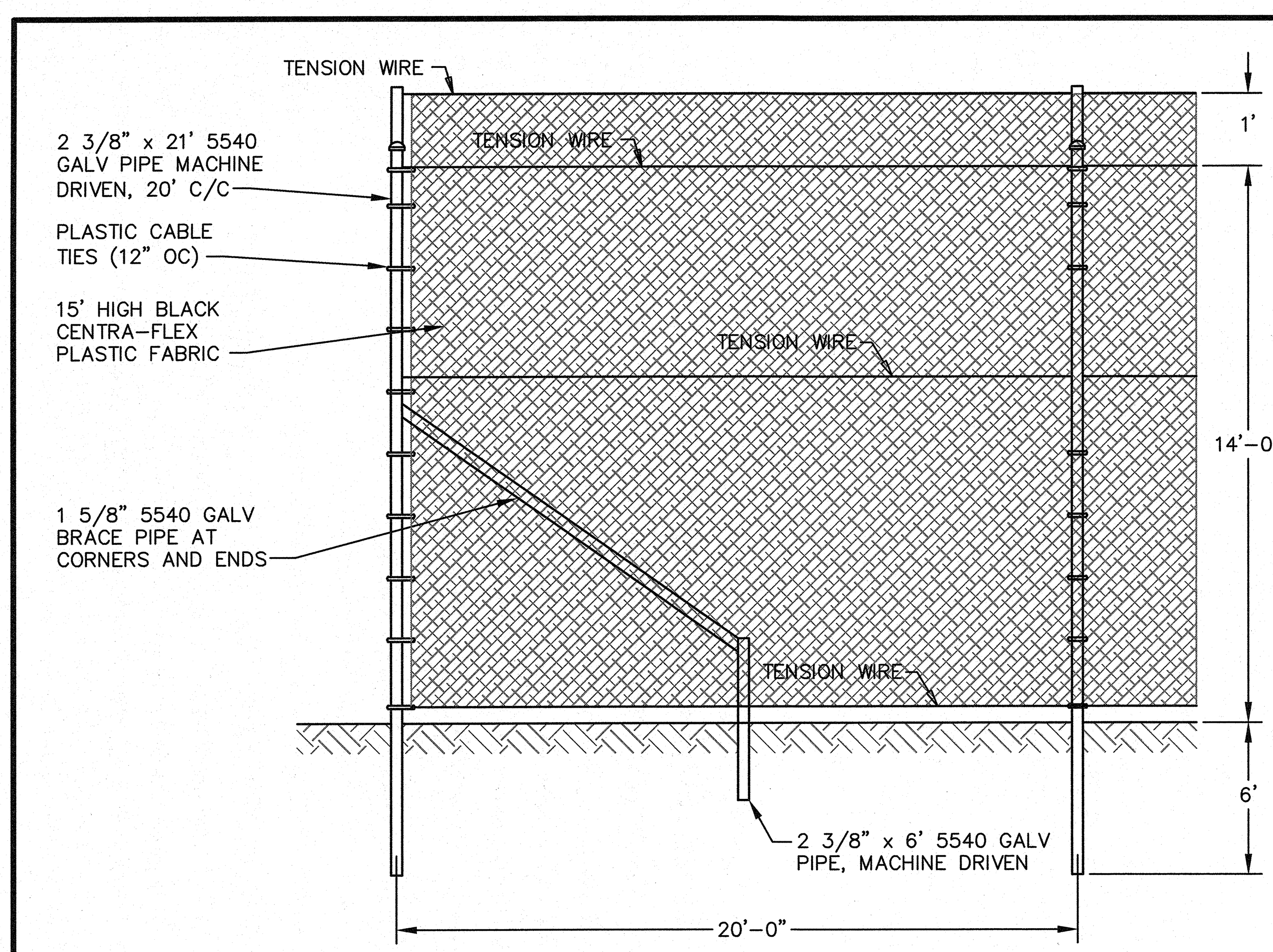
SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

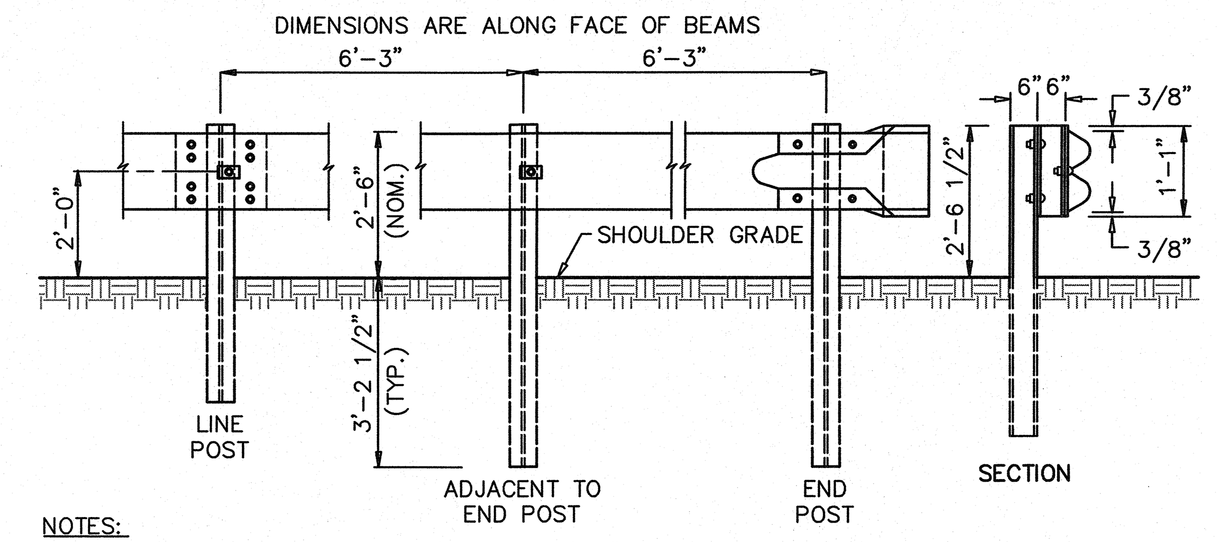
DESIGN BY: PCM
DRAWN BY: SJM
DATE: 12/5/2014
CHECKED BY:
LMN: NONE
CTB: SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

C-305

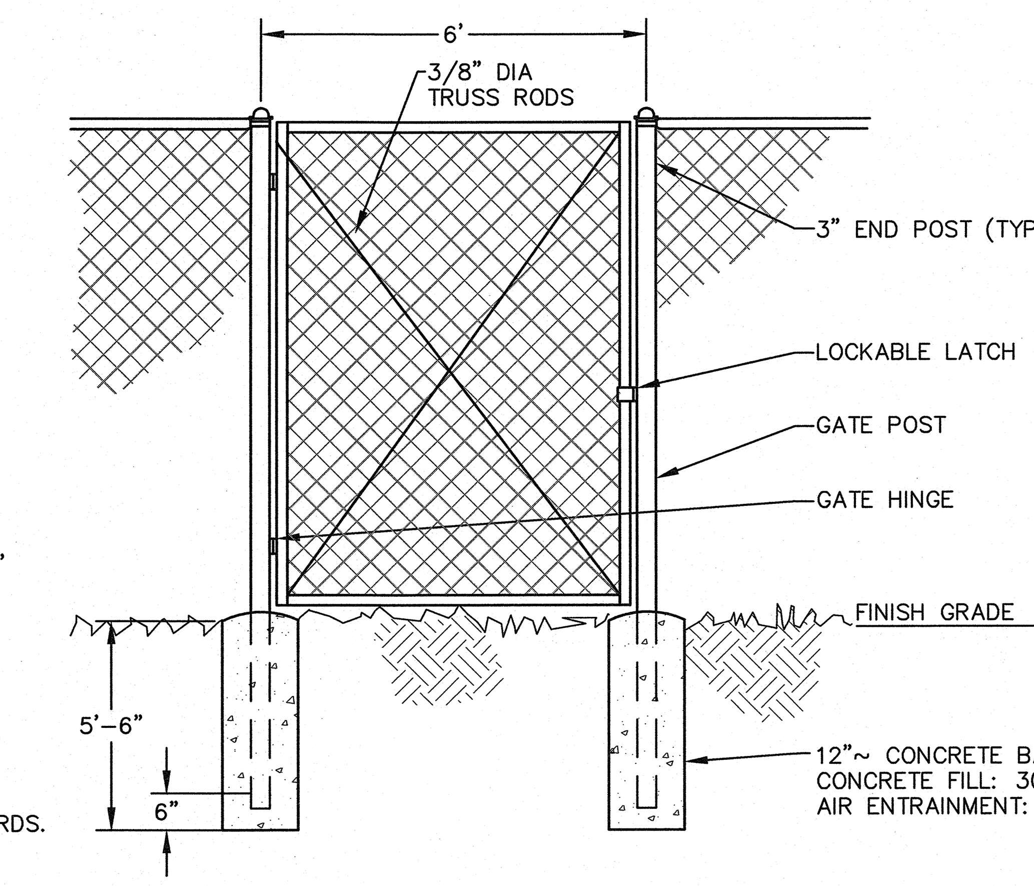


LITTER FENCE
NTS

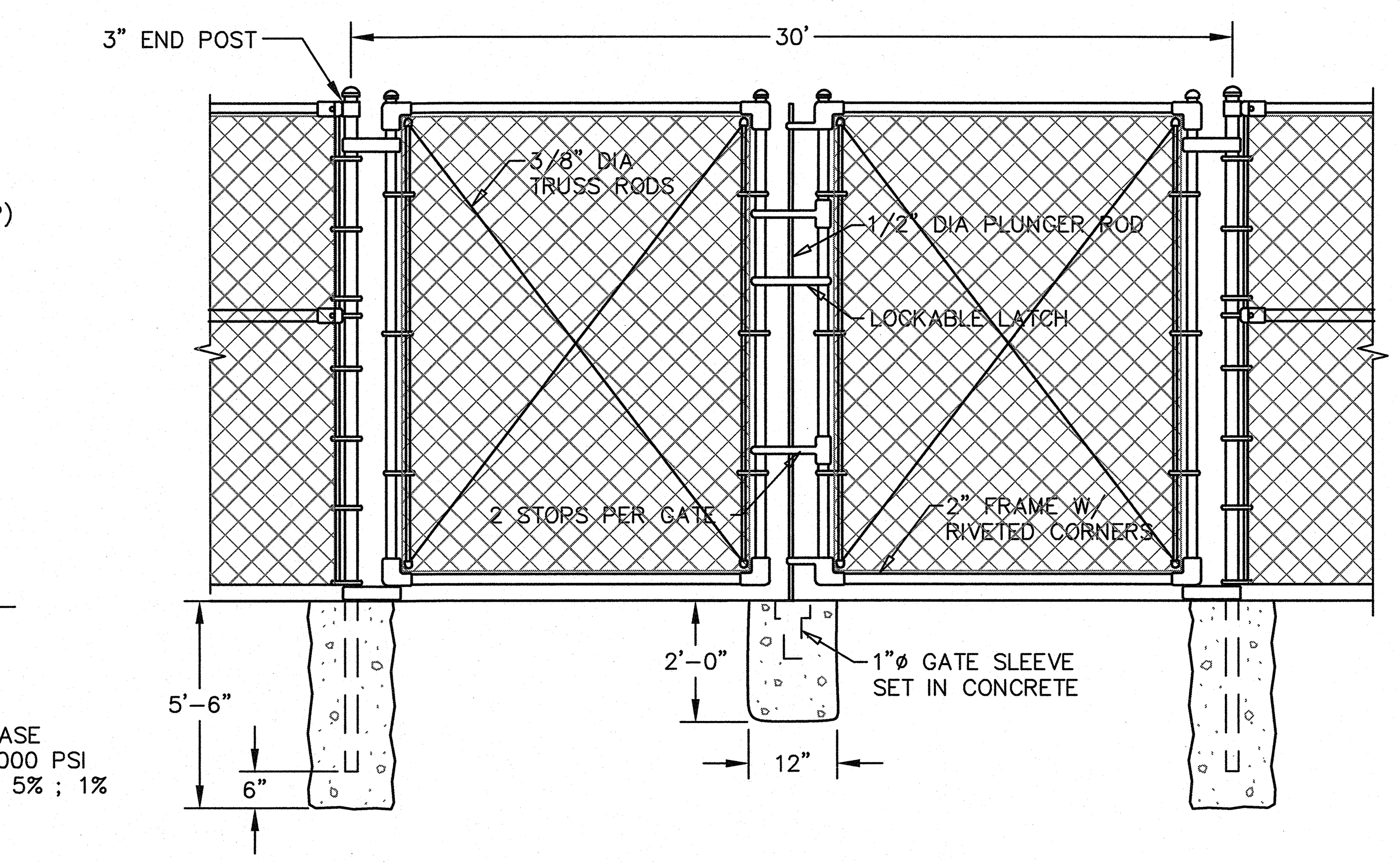


NOTES:
1. INTERMEDIATE POST SPACING SHALL BE 6'-3" UNLESS OTHERWISE SHOWN.
2. POST AND OFFSET BRACKETS SHALL BE 4"x 6" I-BEAM 8.5 OR 9.0 LBS/FT. LENGTH OF 5'-9" ATTACHED WITH 5/8" DIA BOLTS WITH HEX NUTS.
3. ALL HOLES IN BEAM TO BE SHOP-PUNCHED PRIOR TO GALVANIZING.
4. RAIL PANELS AND END SECTIONS TO BE 12 GAUGE STEEL.
5. BACK-UP PLATE TO BE PLACED BEHIND RAIL ELEMENTS AT INTERMEDIATE STEEL POSTS (NON-SPLICE POSTS).
6. ALL PARTS SHALL CONFORM TO CURRENT MAINE DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS.
7. WHEN GUARDRAIL IS CONSTRUCTED AT UP TO FOUR FEET FROM THE EDGE OF PAVEMENT, THE GUARDRAIL WILL BE SET FROM THE GRADE AT THE FACE OF RAIL.
8. END SECTIONS TO BE IN ACCORDANCE WITH MAINE DEPARTMENT OF TRANSPORTATION STANDARDS.
9. GUARDRAIL SET ON A RADIUS OF 150 FEET OR LESS TO BE CIRCULAR.

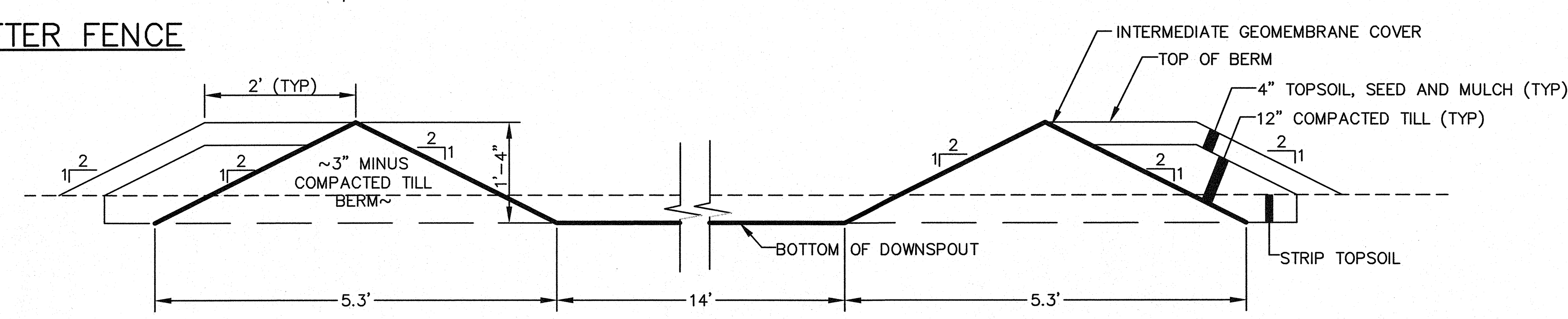
GUARDRAIL
NTS



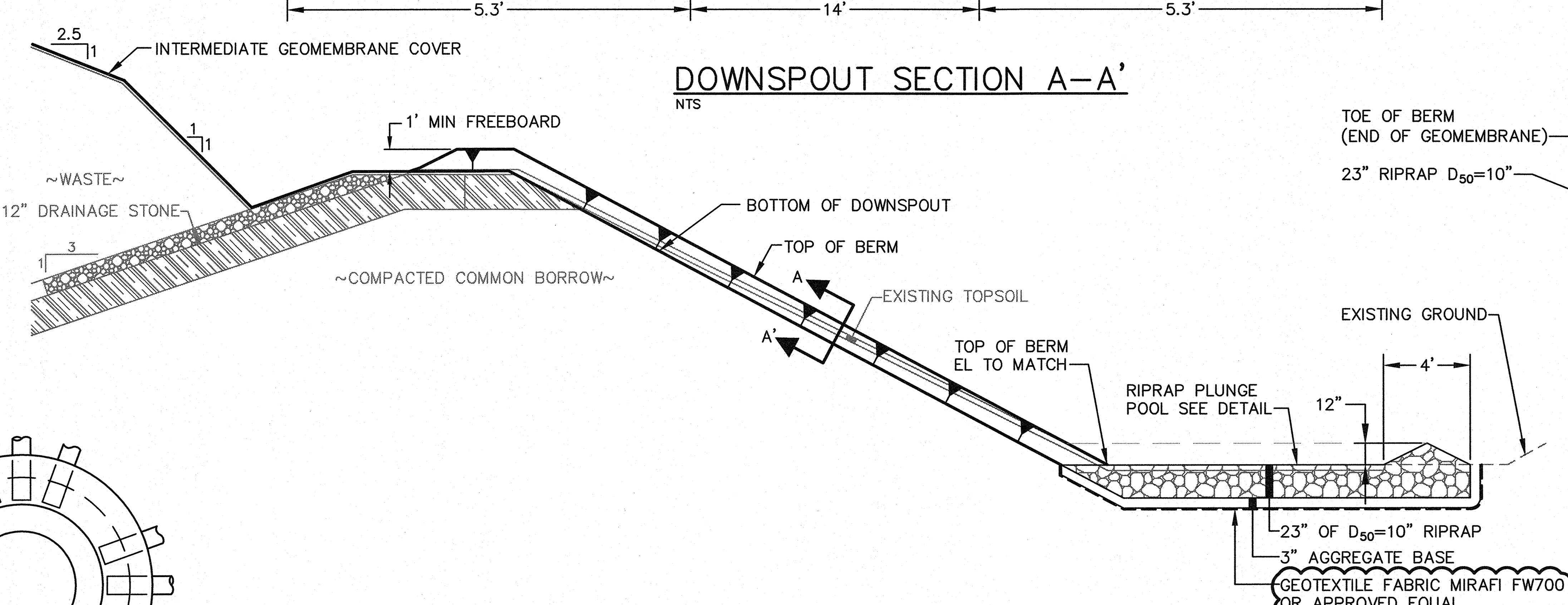
CHAIN LINK FENCE PEDESTRIAN GATE
NTS



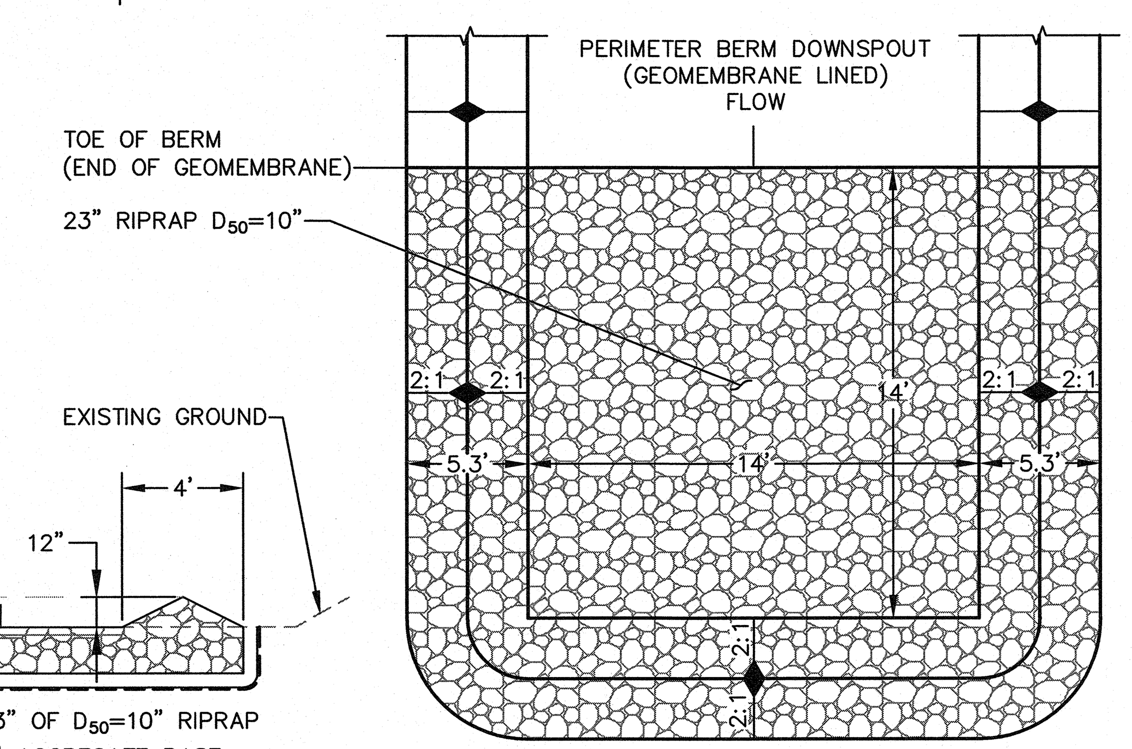
DOUBLE SWING GATE
NTS



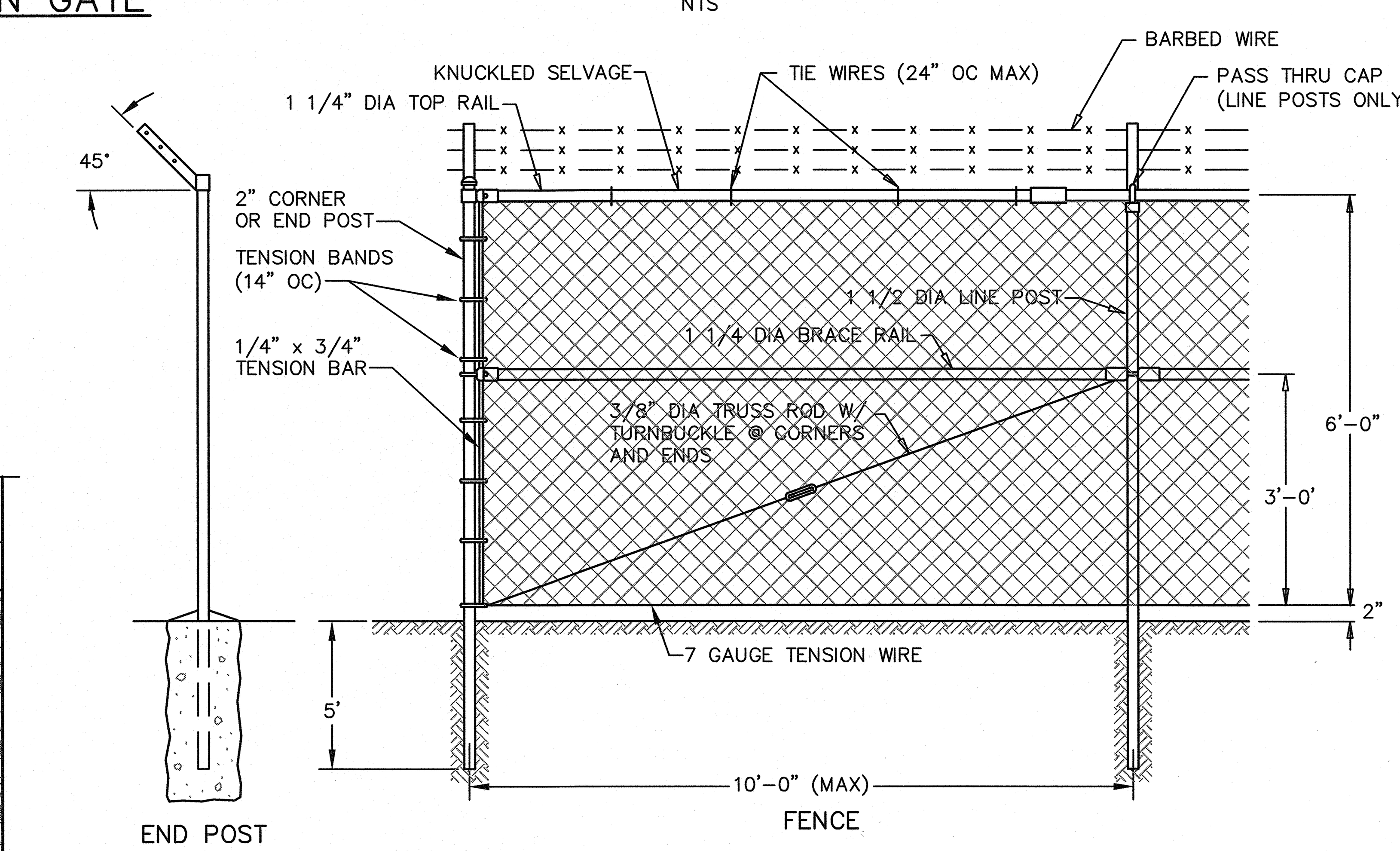
DOWNSPOUT SECTION A-A'
NTS



PERIMETER BERM DOWNSPOUT
NTS



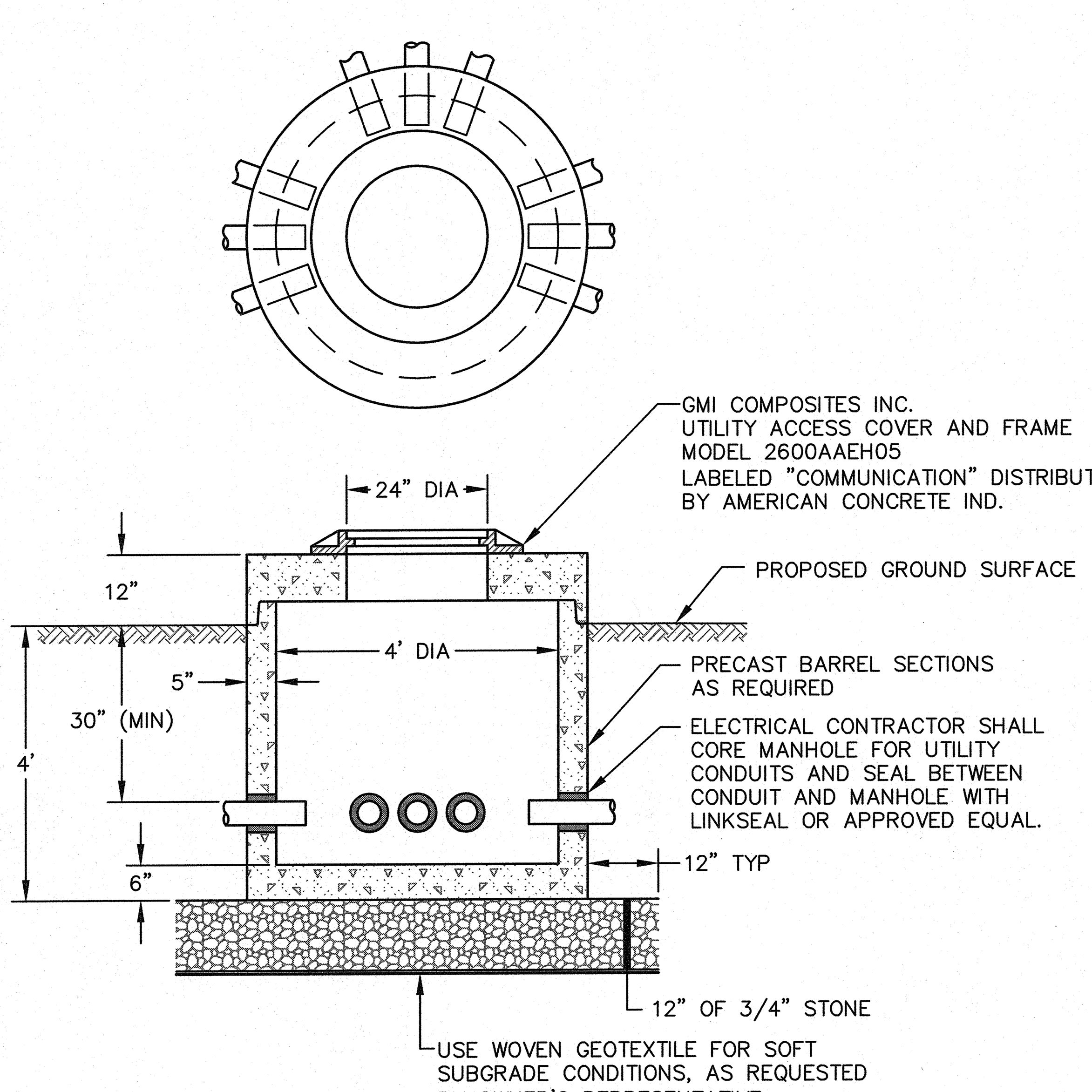
PLUNGE POOL
NTS



NOTES:

1. END OR CORNER POSTS: NOM 2" DIA GALV STL PIPE, MIN 3.65 LB/LF OR 2 1/4" x 2" "H" SECTION, 4.10 LB/LF, OR 3 1/2" x 3 1/2" "L" SECTION WITH INTEGRAL FABRIC LOOPS 5.14 LB/LF.
2. LINE POSTS: NOM 1 1/2" DIA GALV STL PIPE, MIN 2.72 LB/LF OR 1 7/8" x 1 5/8" "H" SECTION, 2.70 LB/LF, OR 1 7/8" x 1 5/8" "C" SECTION, 2.28 LB/LF.
3. TOP & BRACE RAILS: NOM 1 1/4" DIA GALV STL PIPE, MIN 2.27 LB/LF OR 1 5/8" x 1 1/4" "⊓" SHAPED, ROLLED FORMED SECTION.
4. STRETCHER BARS: LENGTH TO BE 1" LESS THAN FULL HEIGHT OF FABRIC. ONE STRETCHER BAR FOR EACH GATE AND END POST, AND TWO STRETCHER BARS FOR CORNER AND BRACING.
5. CLEARING LIMIT 10 FEET EACH SIDE OF FENCE.

CHAIN LINK FENCE
NTS



COMMUNICATION UTILITY MANHOLE
NTS

NOTES:

1. 4' DIA MANHOLE AS MANUFACTURED BY AMERICAN CONCRETE INDUSTRIES OR ENGINEER APPROVED EQUAL.
2. 4000 PSI CONCRETE AT 28 DAYS.
3. DESIGNED FOR H-20 WHEEL LOADING.
4. CONFORMS TO ASTM C-478 SPECIFICATIONS.
5. REINFORCED TO 0.12 IN SQ/LF.
6. SHIPLAP JOINTS SEALED WITH BUTYL RUBBER.
7. EXTERIOR COATED WITH ASPHALTIC PROTECTIVE DAMPROOFING.
8. BOTTOM MIN 5'-0" BELOW FINISH GRADE.
9. PRECAST CONCRETE VAULT MANUFACTURER TO PROVIDE ANTI-FLOATATION EXTENDED BASE SLAB AS NECESSARY. ANTI-FLOATATION DESIGN AND SHOP DRAWINGS SHALL BE PREPARED BY THE MANUFACTURER AND SUBMITTED TO THE ENGINEER FOR APPROVAL.

REV.	BY	DATE	STATUS
	PCM	2/2016	REVISED PER MEDEP COMMENTS
	PCM	7/2015	ISSUED FOR MEDEP SOLID WASTE PERMIT APPLICATION

STATE OF MAINE · LICENSED · PROFESSIONAL ENGINEER · No. 5144

CELL 11 CONSTRUCTION
JUNIPER RIDGE LANDFILL
NEWSME LANDFILL OPERATIONS LLC
OLD TOWN, MAINE

SECTIONS AND DETAILS

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE
4 Blanchard Road, PO Box 85A, Cumberland Center, Maine 04021
Phone 207.829.5016 • Fax 207.829.5692 • www.smemaine.com

DESIGN BY:	PCM
DRAWN BY:	SJM
DATE:	12/5/2014
CHECKED BY:	
LMN:	NONE
CTB:	SME-STD

JOB NO. 14101.00 DWG FILE DETAILS

C-306

SME 3

VOLUMES I AND III UPDATED APPLICATION TABLES

- **VOLUME I APPENDIX J SWMP**
  - **TABLE 7 SUMMARY OF STORMWATER CULVERTS, STORM DRAINS, CATCH BASINS, DITCHES**
- **VOLUME III**
  - **TABLE 3-5 GAS MANAGEMENT SYSTEMS**
- **VOLUME III**
  - **TABLE 3-7 POTENTIAL FAILURE MODES AND SIGNIFICANCE OF FAILURES IN ENGINEERED SYSTEMS**

VOLUME I APPENDIX J SWMP

TABLE 7 SUMMARY OF STORMWATER CULVERTS, STORM DRAINS, CATCH BASINS, DITCHES

TABLE 7-1

SUMMARY OF STORMWATER CULVERTS, STORM DRAINS, CATCH BASINS, DITCHES

Culverts	Diameter (in.)	Material	Length (ft.)	Slope (%)	Inv. In.	Inv. Out
EC-D-1G	24 (2)	CMP	56	0.018	183.0	182.0
C-2BA	36	HDPE	40	0.008	203.2	202.9
C-2BB	24	HDPE	96	0.010	195.0	194.0
C-4BA	24	HDPE	78	0.009	204.4	203.7
C-4BB	24	HDPE	78	0.009	204.4	203.7
C-4F	18	HDPE	78	0.04	165.0	162.0
C-4G	24	HDPE	36	0.028	175.0	174.0
C-4HA	18	HDPE	40	0.025	201.9	200.9
C-4HB	18	HDPE	101	0.025	178.5	176.0
C-4I	18	HDPE	80	0.131	202.5	192.0
C-4IA	18	HDPE	40	0.023	212.9	212.2
C-4JA	18	HDPE	60	0.028	214.0	212.3
C-4JB	24	HDPE	73	0.021	211.5	210.0
C-4JC	24	HDPE	73	0.021	211.5	210.0
C-4K	24	HDPE	51	0.043	216.5	214.3
C-4L	18	HDPE	121	0.017	213.0	211.0
C-4N	18	HDPE	33	0.030	184.0	183.0

Catch Basin	Basin Dia. (ft)	Grate Opening (in.)	Depth (ft)	Culvert Dia. (in.)
CB-2BB	4	30	7.2	24
CB-4G	4	24	8	24
CB-4HB	4	24	6.9	18
CB-4I	4	24	7.1	18
CB-4JA	4	24	6.7	18
CB-4K	4	30	5.5	24
CB-4L	4	24	4	18

Ditch	Base Width (ft.)	Depth (ft.)	Sideslope Z-Value ('/')	Lining
Ditch to Detention Pond 10	2	2	2	Segments 1&2: NAG S75 erosion Mat Segment 3: Riprap (D50=4", t=9")
Detention Pond 10 Emergency Spillway	10	2	2	Riprap (D50=4", t=9")
Perimeter (toe)	2'	2'	2 '/'	NAG S75 erosion mat
Maintenance Road Ditch	2'	3'	2 '/'	NAG S75 erosion mat
Terrace Drain	0' - V-ditch	1.5'	2 '/'	NAG C125BN erosion mat
Downspouts	4'	2'	2 '/'	Riprap (D50=8", t=18")

Notes:
1. Existing culverts to remain for Post Development condition.
2. Location of structures shown on Drawing C-107 contained in Appendix D.

VOLUME III

TABLE 3-5 GAS MANAGEMENT SYSTEMS

TABLE 3-5

GAS MANAGEMENT SYSTEMS

Engineered System	Design Selection	Site-Specific Factors	Basis for Engineering Design	Proposed Construction Procedure
Horizontal Gas Collection Trenches	• HDPE perforated pipes bedded in stone or tire chips • spaced 40 feet vertical, 100 feet horizontal • gas wells equipped with flow control values	• Placed in waste during filling operations connected to transport piping	• To assist controlling odor during waste filling	• Install in waste as waste filling occurs • Slope a minimum slope of 5 percent • As-built locations surveyed after installation
Vertical Gas Extraction Wells	• 8-Inch PVC perforated pipe screened section encased in stone or other porous media • gas wells equipped with flow control valves	• Wells installed with 18 months of waste reaching interim cell grades space for 100-foot radius of influence	• To extract landfill gas generated by the facility to assist in controlling odors and reduce green-house gas emissions	• Drill gas wells with 30-inch bucket auger • Install PVC pipe and backfill annulus with porous media • Connect to gas transmission piping
Gas Collection Transport and Header Pipes	• Gas Headers 24-inch HDPE transport pipes 6- to 12-inch HDPE	• Headers located in perimeter berm connected to existing headers • Transport pipes run up & down waste slope at perimeter of landfill • Condensate collection provided in transport and headers.	• Headers sized for projected gas flow based on site specific gas flow modeling	• Gas headers installed during berm construction • Gas transport pipes install as waste in placed in cells

VOLUME III

TABLE 3-7 POTENTIAL FAILURE MODES AND SIGNIFICANCE OF FAILURES IN ENGINEERED SYSTEMS

TABLE 3-7
(revised 2/2016)
POTENTIAL FAILURE MODES AND SIGNIFICANCE OF FAILURES IN ENGINEERED SYSTEMS

System I.D.	System Components	Design Considerations	Potential Failure Modes	Impact of Failure	Design Contingencies	Failure Significance Limiting Factors
Underdrain system	• Sand/stone (12-inch thick layer) • HDPE perforated pipe 6-inch diameter • Geotextiles (for separation)	• Pipe size • Pipe strength • Pipe spacing • Filter design (sand/stone) • Separation geotextile design	• Pipe clogging • Pipe collapse	• The groundwater level in the location of the underdrain would rise.	• Use of conservative material specification (i.e., K $\geq 1\times10^{-2}$ cm/sec) • Slope of base will provide positive drainage even with collapsed pipes • Pipe bedding with stone will provide drainage even if pipe is clogged or collapsed. • Pipe burial with semi-trench condition and compacted stone backfill will increase pipe strength and minimize collapse	• Performance of underdrain in short-term (i.e. protect bedrock, dewater site) will not be affected • Performance of underdrain in long-term (tertiary back-up of liner system) would be diminished but would remain functional
Secondary and Augmented Liner Systems	• 60-mil textured HDPE liner • GCL (augmented only) • 12-inch thick clay barrier soil (augmented only)	• Interface friction angle • Internal friction angle • Anchor trench design • Connection details • GRI GM-13 geomembrane standards • Hydraulic conductivity (1×10^{-7} cm/sec min) • In-place density 90 percent maximum dry-density • Moisture-content 0 to 4 percent above optimum • Place in 9-inch lifts	• Installation damage during construction • Manufacturing defects • Premature hydration of GCL • Stability/settlement issues • Clods, voids, poor bonding of lifts • Freeze/thaw cracking • Desiccation cracking	• Impact of failure of the secondary liner system would result in any fluid immediately above the failed liner flowing vertically through the underlying barrier soils. The flow rate would be controlled by the hydraulic conductivity of the barrier soil layer and liquid level above the barrier soil layer.	• Strict construction QA/QC to provide proper installation and repairs to manufacturer's or construction related defects • Proper timing of liner placement to avoid premature hydration of GCL • Use of proper anchor trench and cell development techniques to avoid stability issues with liner system • Use of composite liner with good intimate contact (HDPE/GCL) to minimize leakage if liner gets damaged during construction or operation • Use of imported barrier soil as a tertiary back-up to liner system	• Significance of failure is greatly diminished due to redundant composite liner and back-up barrier soil

TABLE 3-7 (cont'd)

System I.D.	System Components	Design Considerations	Potential Failure Modes	Impact of Failure	Design Contingencies	Failure Significance Limiting Factors
Leak Detection System	• Sand/stone (12-inch thick) • HDPE perforated pipe (6-inch diameter) • Geonet drainage composite	• Capacity and detection time, action leakage rate • Pipe size, strength, spacing • Filter design and separation design • Connection details • Dynamic and static loading • Monitoring and maintenance	• Clogging • Settlement • Pipe collapse	• Failure of the leak detection system would result in liquid levels increasing in the leak detection layer. Higher liquid levels would result in higher hydraulic gradients for flow emanating from the leak detection system.	• Use of redundant design components to collect and transport leachate (i.e. pipe and geonet) • Use of filter design criteria at critical material interfaces • Use of conservative material specifications and design factors of safety to account for nonperformance (i.e. K $\geq 1\text{x}10^{-2}$ cm/sec sand) • Relatively steep drainage slopes reduce clogging potential of system	• Significance of failure minimized by design conservatism and redundancy • Slope of liner system reduces the potential of system clogging • Pipe bedding with stone will provide drainage even if pipe is clogged or collapsed. • Pipe burial with semi-trench condition and compacted stone backfill will increase pipe strength and minimize collapse
Primary Composite Liner System	• 80-mil textured HDPE liner • GCL • Clay 12 inches	• Hydraulic conductivity ($1\text{x}10^{-7}$ cm/sec) max. particle size 3", 35% fines, 90% maximum density, 0-4% moisture content, 9" lifts • GRI GM-13 geomembrane standards • Interface/internal friction angle • Anchor trench design • Connection details	• Freeze/thaw • Desiccation • Stability/settlement • Installation damage during construction • Manufacturing defects • Premature hydration of GCL	• Failure of the primary composite liner system would allow leachate to flow into the leak detection which would collect the leachate above the secondary liner and direct it to the leak detection pumping system.	• Strict construction QA/QC to provide proper installation and repairs to manufacturer's or construction related defects • Proper timing of liner placement to avoid premature hydration of GCL • Use of proper anchor trench and cell development techniques to avoid stability issues with liner system • Use of composite liner with good intimate contact (HDPE/GCL) to minimize leakage if liner gets damaged during construction or operation • Use of composite secondary liner and leak detection system as secondary back-up to primary liner system • Use of imported barrier soil and underdrain system as a tertiary back-up to liner system • Use of frost protection layer over leachate collection system to minimize freeze/thaw effects	• Significance of failure is greatly diminished due to redundant composite primary liner, composite secondary liner with leak detection layer, and back-up barrier soil/underdrain system

TABLE 3-7 (cont'd)

System I.D.	System Components	Design Considerations	Potential Failure Modes	Impact of Failure	Design Contingencies	Failure Significance Limiting Factors
Leachate Collection Systems	• Sand/stone (12" thick) • HDPE perforated pipe 6"-12" dia. • Geonet drainage composite	• Capacity/hydraulic head on liner system • Pipe size, strength, spacing • Filter design • Connection details • Hydraulic conductivity • Dynamic and static loading • Monitoring and maintenance	• Clogging of collection system components • Pipe failure	• Failure of the leachate collection system would allow leachate to collect over the primary liner increasing the leachate elevation, and hence the hydraulic gradient above the primary liner.	• Use of redundant design means to collect and transport leachate (i.e. pipe and geonet) • Use of filter design criteria at critical material interfaces • Use of conservative material specifications and design factors of safety to account for nonperformance (i.e. $K \geq 1x10^{-2}$ cm/sec sand) • Slopes of liner reduce clogging potential of system	• Significance of failure minimized by design conservatism and redundancy • Slopes of liner system reduces the potential of system clogging • Pipe bedding with stone will provide drainage even if pipe is clogged or collapsed. • Pipe burial with semi-trench condition and compacted stone backfill will increase pipe strength and minimize collapse
Leachate Transport System – Sump and Pump	• HDPE dual containment force main (6"x10") • Dual containment pipe leak detection • Pump station • Valve pit • Control panel • Electrical supply • Back-up power • Alarm system • Sump area	• Capacity of sump area • Monitoring and maintenance • Longevity of components • Connection to existing transport line • Pump sizing (leachate, leak detection, and underdrain)	• Clogging of sump area • Power outage (long-term) • Power surges • Pipe failure • Clogging of transport pipe • Pump failure	• Failure of the leachate transport sump and pump system would result in leachate being stored in the landfill until it could be pumped out.	• Redundant design of system including: - Use of (2) sump pumps for leachate transport - Use of back-up generator - Use of dual-containment pipe • Pump leachate at minimum velocity of 2 fpm to prevent clogging of transport pipe • Use of pressure gauges to detect leaks in force main • Use of flow meter to monitor performance of system	• Significance of failure minimized by redundant design of system and ability to detect and respond to failures immediately
Gas Collection	• Sand/stone • HDPE pipe	• Capacity • Pipe size, strength, spacing	• Stability/settlement • Clogging • Pipe collapse • Water saturation	• Gas generated in the landfill either would be stored in the landfill or would migrate through the cover.	• Use of pipe strength design criteria and bedding installation methods to prevent pipe collapse • Install gas header pipes in perimeter berm to limit settlement of pipes • Install gas transport pipes up and down waste sideslopes to limit plugging of pipes	• Failure potential minimized by design contingencies • Significance of failure minimized by redundant design of system

TABLE 3-7 (cont'd)

System I.D.	System Components	Design Considerations	Potential Failure Modes	Impact of Failure	Design Contingencies	Failure Significance Limiting Factor
Final Cover Systems	• 12-inch medium for growing grass • 12-inch sand drainage layer • 40-mil geomembrane (textured) • 24 inches barrier soil (K <u><</u>1x10-7 cm/sec)	• Interface and internal friction angles • Long-term creep • Desiccation • Freeze/thaw	• Stability/settlement • Erosion	• The failure of the cover would result in precipitation entering the waste mass. This precipitation would be collected by the leachate collection system	• Use of sand layer on sideslopes to over design drainage of cover system and minimize stability issues • Use of composite cover system to provide redundant barrier to precipitation infiltration • Use of material components designed to provide the maximum internal and interface friction angles to minimize stability issues • Use of thick soil cover (24 inches) over the barrier soil component of cover system to minimize effects of freeze/thaw and desiccation • Use of strict post-closure maintenance to minimize erosion issues	• Failure potential minimized by design contingencies • Significance of failure minimized by redundant design of system

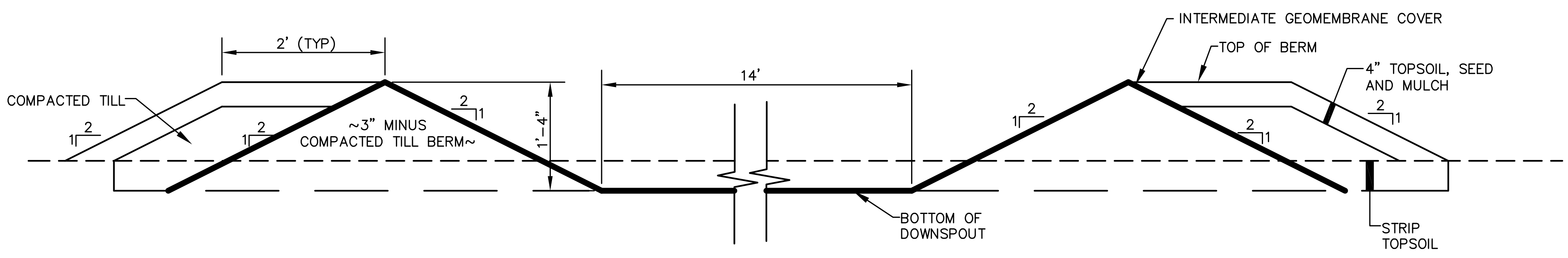
SME 4

VOLUME III AND IV UPDATED AND SUPPLEMENTAL APPLICATION FIGURES AND CALCULATION

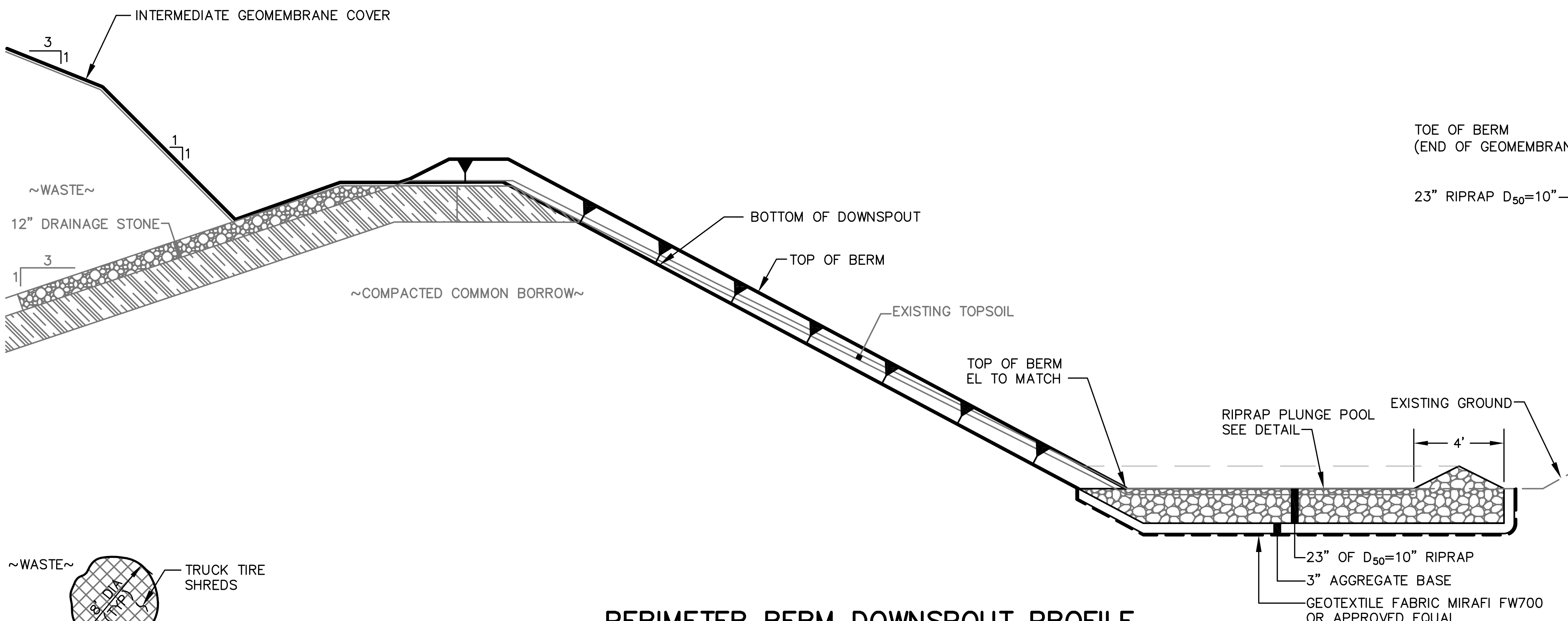
- **VOLUME IV**
  - o **FIGURE E-3 TYPICAL DEVELOPMENT DETAILS**
- **VOLUME III APPENDIX F-8**
  - o **FIGURE 1 SETTLEMENT EVALUATION POINTS**
- **VOLUME III APPENDIX E SUPPLEMENTAL FIGURES AND CALCULATION**
  - o **FIGURE 1 FINAL COVER TERMINATION**
  - o **FIGURE 2 DOWNSPOUT TERMINATION**
  - o **FIGURE 3 PIPE BOOT**
  - o **FIGURE 4 FINAL COVER TO PUMP STATION CONNECTION**
  - o **FIGURE 5 FUTURE CONNECTION TO TEMPORARY LEACHATE SUMP**
  - o **PLUNGE POOL CALCULATION**
- **VOLUME IV APPENDIX E SUPPLEMENTAL FIGURES**
  - o **FIGURE 6 LANDFILL ACCESS ROAD WITH INTERMEDIATE COVER**
  - o **FIGURE 7 SIDESLOPE INTERMEDIATE DOWNSPOUT SECTION**

VOLUME IV

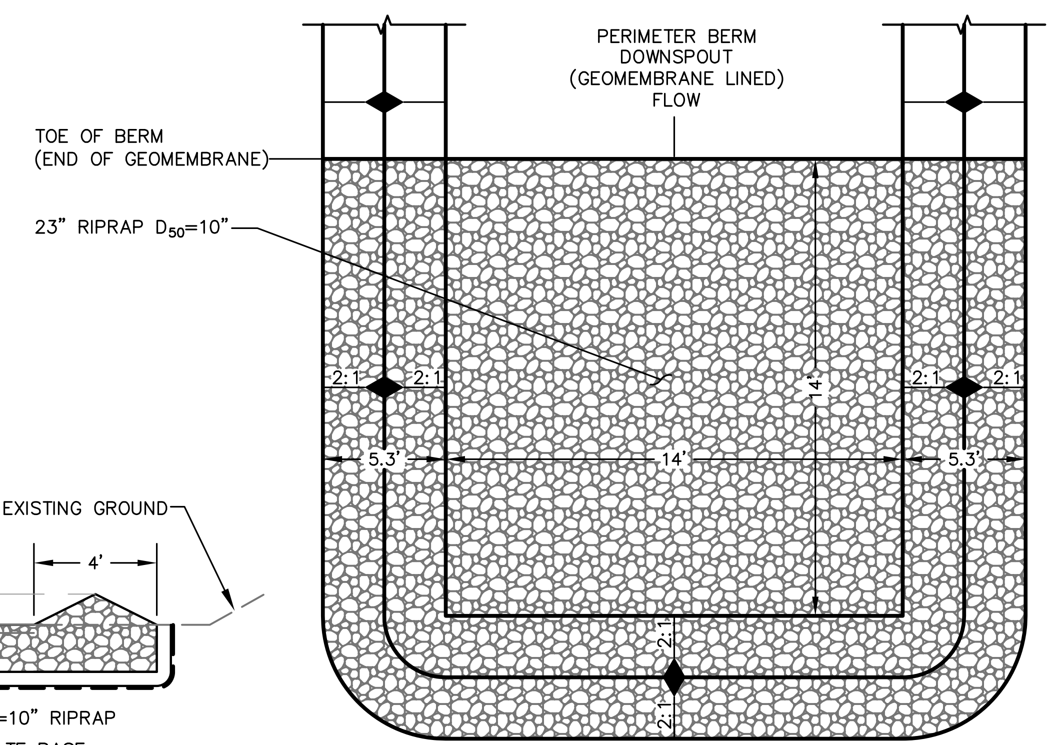
- **FIGURE E-3 TYPICAL DEVELOPMENT DETAILS**



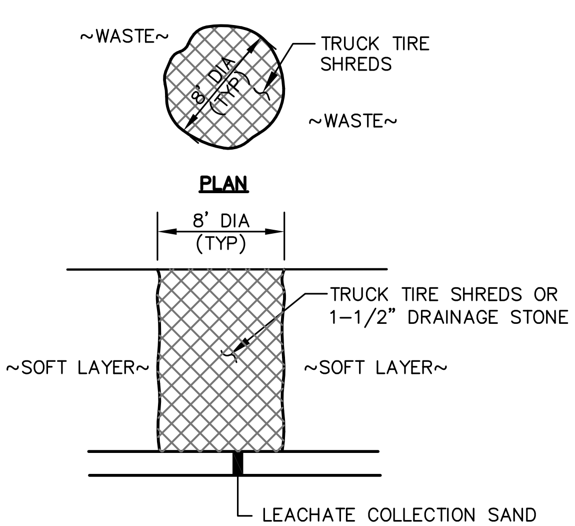
DOWNSPOUT SECTION
NTS



PERIMETER BERM DOWNSPOUT PROFILE
NTS

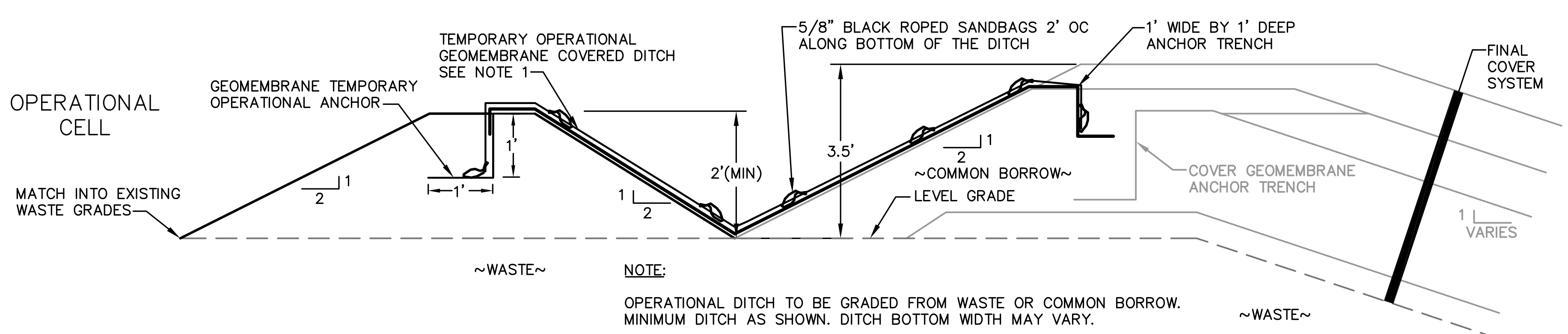


PLUNGE POOL
NTS



NOTE:
CHIMNEY DRAINS TO BE LOCATED AS SHOWN ON FIGURE 1

CHIMNEY DRAIN
NTS



NOTE:
OPERATIONAL DITCH TO BE GRADED FROM WASTE OR COMMON BORROW. MINIMUM DITCH AS SHOWN. DITCH BOTTOM WIDTH MAY VARY.

OPERATIONAL DITCH ALONG FINAL COVER

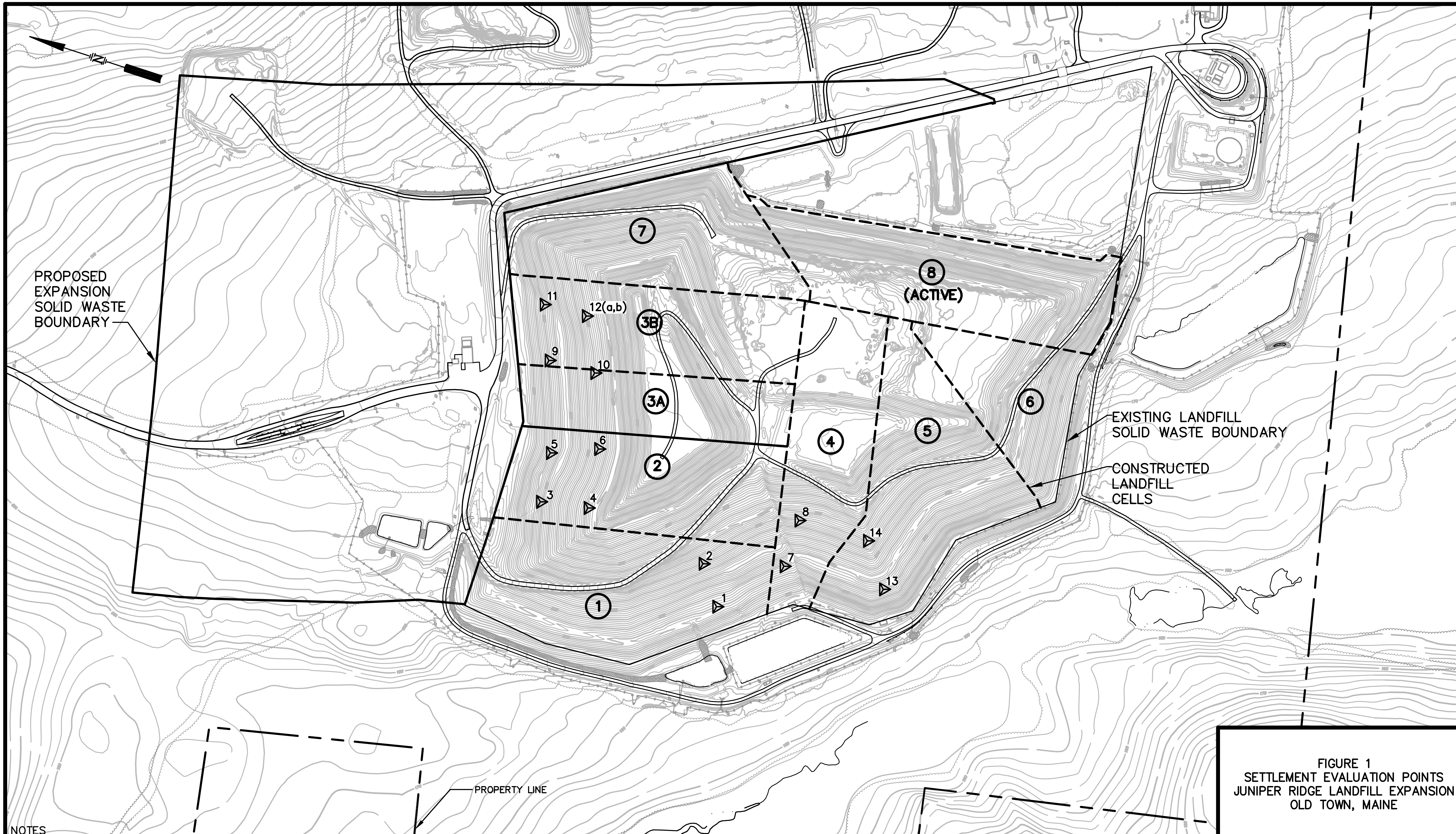
FIGURE E-3
TYPICAL DEVELOPMENT DETAILS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: C11-CELLDEV LMN: NONE CTB: SME-STD REV: 2/19/2016

VOLUME III APPENDIX F-8

- **FIGURE 1 SETTLEMENT EVALUATION POINTS**



NOTES

1. EXISTING GROUND CONTOURS FROM DECEMBER 31, 2014 AERIAL SURVEY PERFORMED BY AERIAL SURVEY AND PHOTO, INC. OF NORRIDGEWOCK, MAINE.
2. PROPERTY LINE LOCATIONS ARE A RESULT OF FIELD SURVEY PERFORMED BY HERRICK AND SALSBURY, INC. LAND SURVEYORS, ELLSWORTH, MAINE FOR TRYTON TREE FARM PROJECT, PATTEN CORPORATION–DOWNEAST, OLD TOWN, MAINE, FEBRUARY 23, 1988, REVISED APRIL 7, 1988.

LEGEND

1 SETTLEMENT EVALUATION POINT

150 0 300 FEET

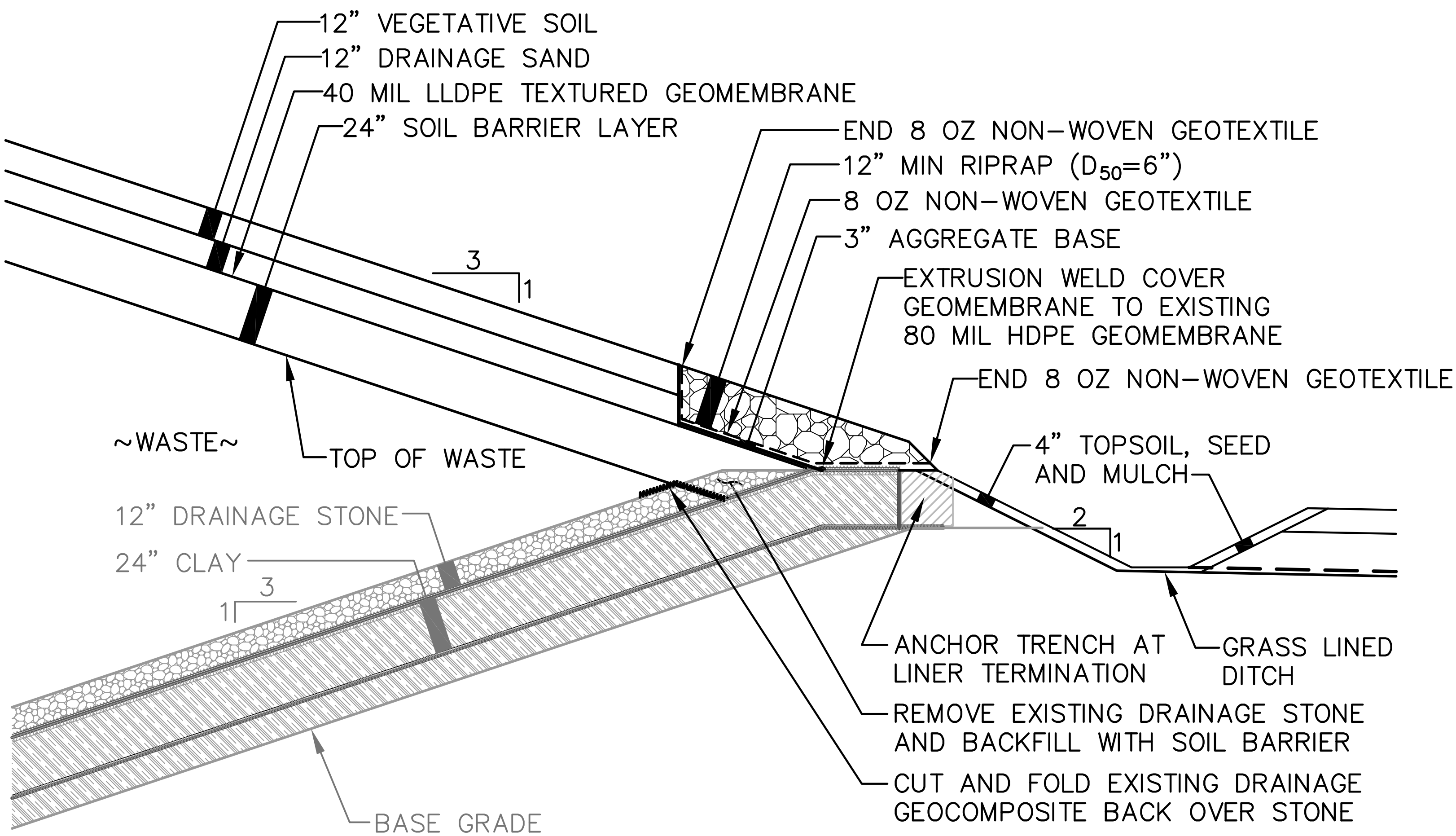
FIGURE 1
SETTLEMENT EVALUATION POINTS
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: FIGURES LMN: SETTLEMENT POINTS CTB: CONDUCTIVITY REV: 1/28/2016

VOLUME III APPENDIX E SUPPLEMENTAL FIGURES AND CALCULATION

- **FIGURE 1** **FINAL COVER TERMINATION**
- **FIGURE 2** **DOWNSPOUT TERMINATION**
- **FIGURE 3** **PIPE BOOT**
- **FIGURE 4** **FINAL COVER TO PUMP STATION CONNECTION**
- **FIGURE 5** **FUTURE CONNECTION TO TEMPORARY LEACHATE SUMP**
- **PLUNGE POOL RIPRAP CALCULATION**



FINAL COVER TERMINATION

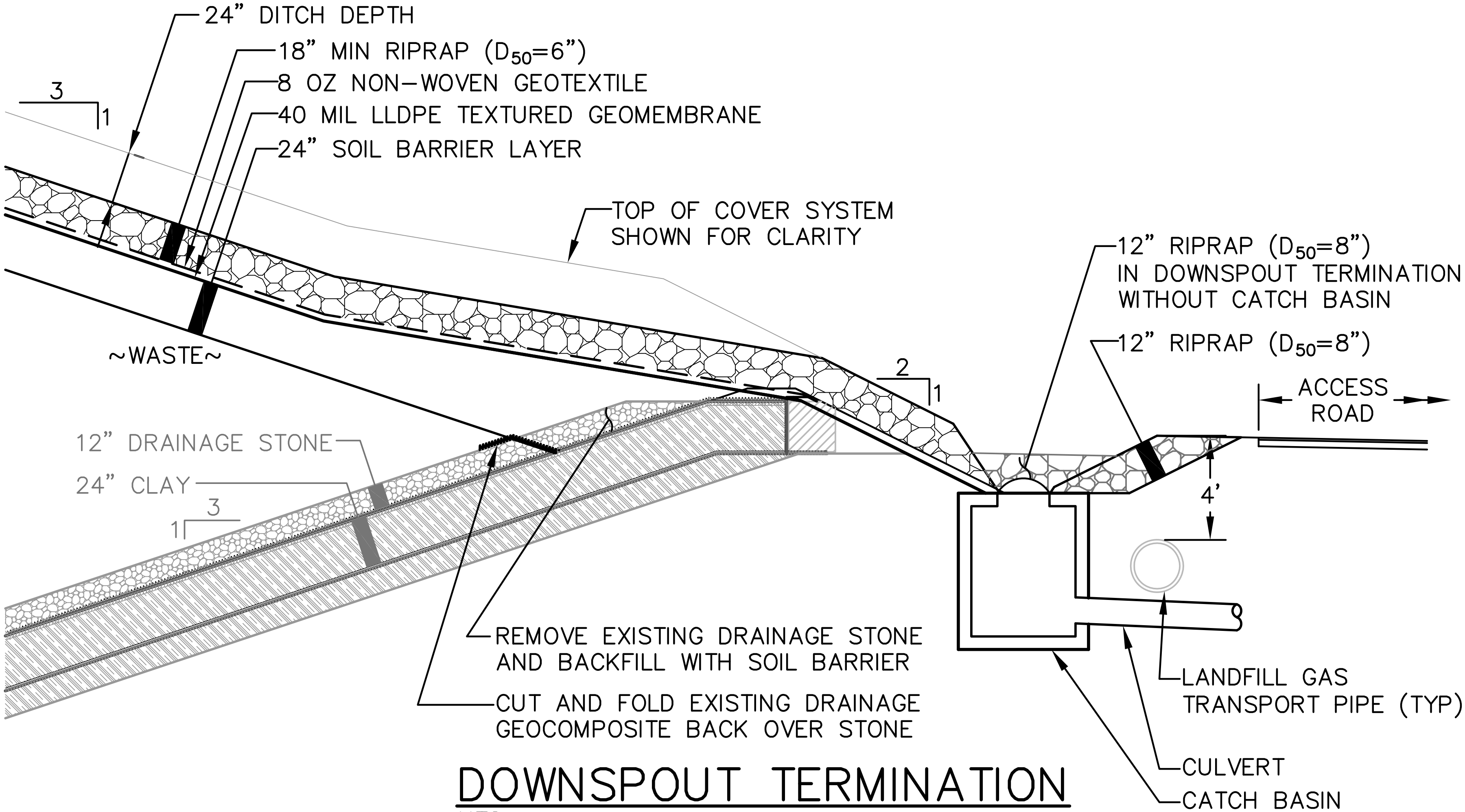
NTS

FIGURE 1
FINAL COVER TERMINATION
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL-FIGURES LMN: NONE CTB: SME-STD REV: 2/10/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\DETAIL-FIGURES.dwg, 2/10/2016 1:46:37 PM, sjm



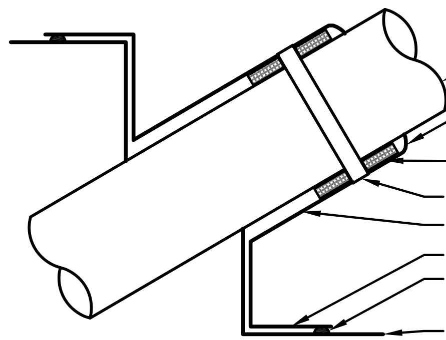
DOWNSPOUT TERMINATION

NTS

FIGURE 2
DOWNSPOUT TERMINATION
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL–FIGURES LMN: NONE CTB: SME–STD REV: 2/12/2016



CLEANOUT PIPE BOOT

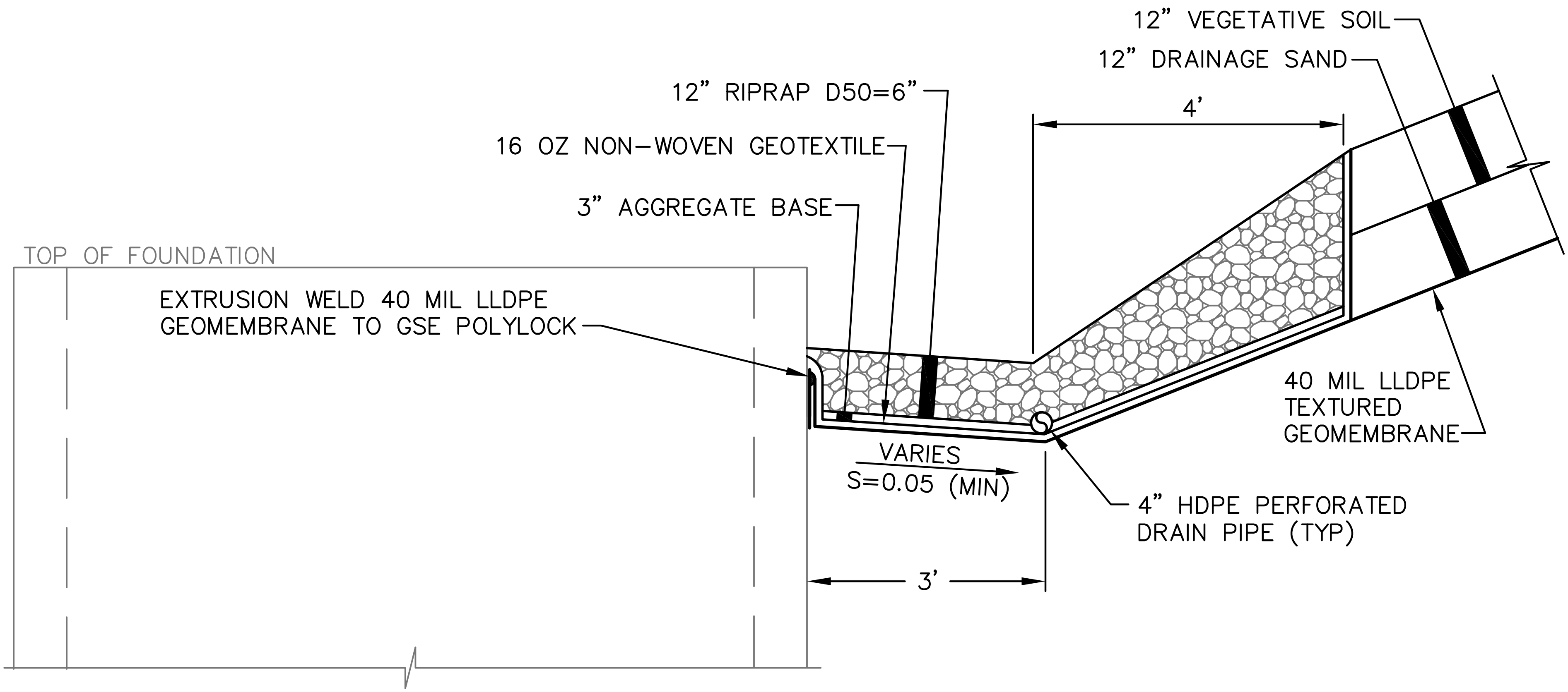
NTS

FIGURE 3
PIPE BOOT
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL-FIGURES LMN: NONE CTB: SME-STD REV: 2/17/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\DETAIL-FIGURES.dwg, 2/17/2016 7:32:40 AM, sjm



FINAL COVER TO PUMP STATION CONNECTION

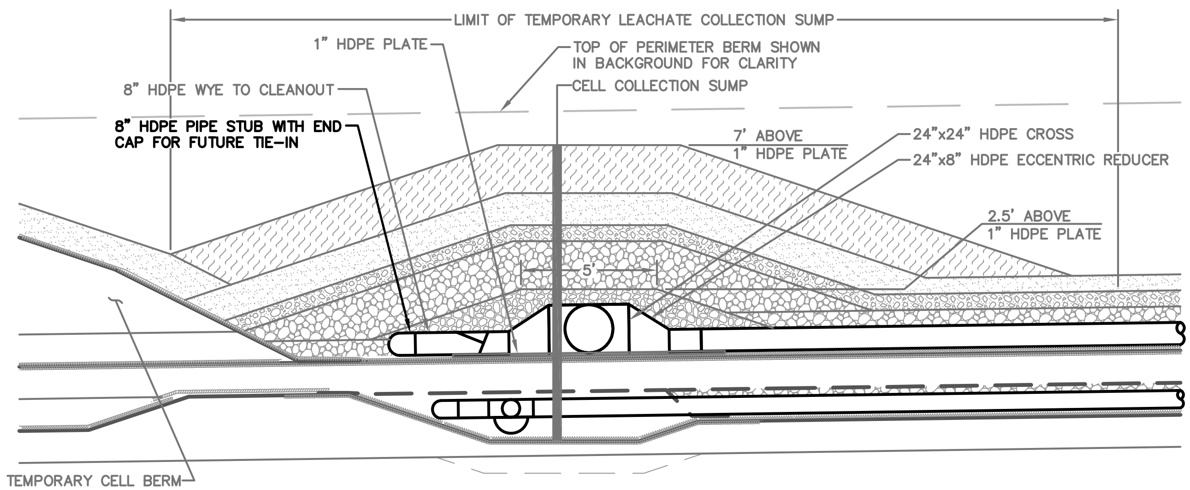
NTS

FIGURE 4
FINAL COVER TO
PUMP STATION CONNECTION
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

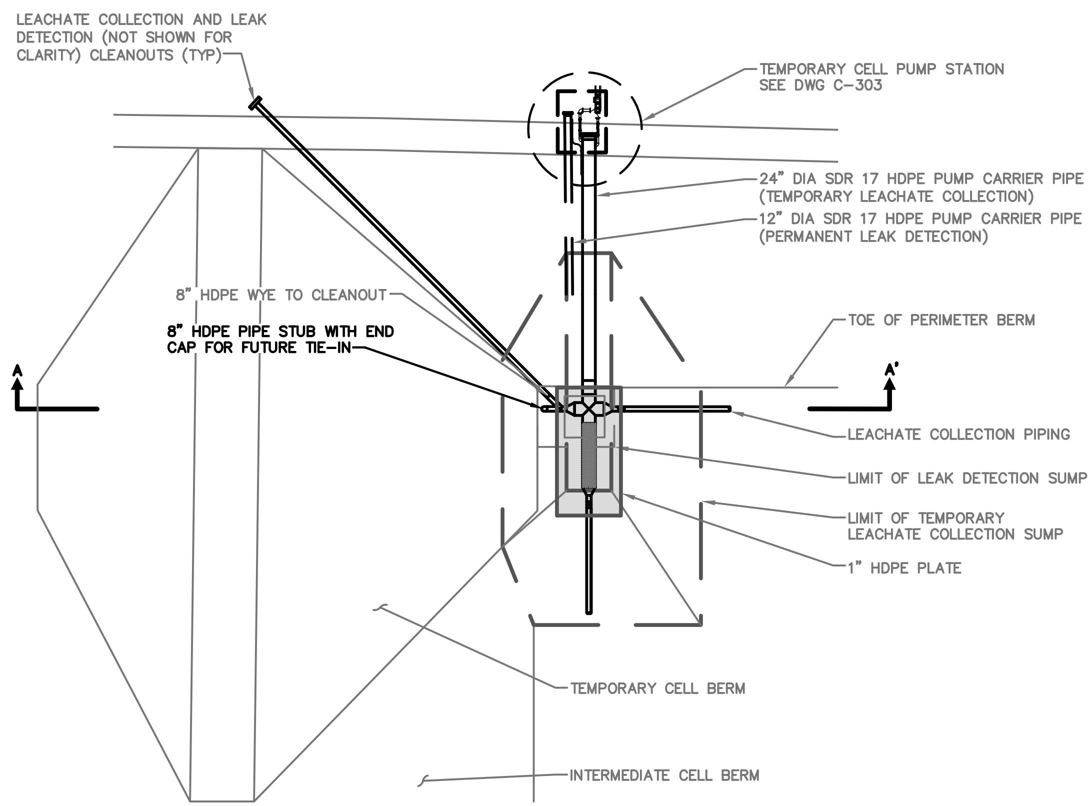
SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL-FIGURES LMN: NONE CTB: SME-STD REV: 2/10/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\DETAIL-FIGURES.dwg, 2/10/2016 1:48:13 PM, sjm



SECTION A–A'



TEMPORARY CELL SUMP AND PUMP STATION
NTS

FIGURE 5
FUTURE CONNECTION TO
TEMPORARY LEACHATE SUMP
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL-FIGURES LMN: NONE CTB: SME-STD REV: 2/5/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\DETAIL-FIGURES.dwg, 2/5/2016 8:57:23 AM, sjm

CELL DEVELOPMENT
OPERATIONAL PLUNGE POOL
RIPRAP SIZING

```
       ******  HYCHL  ****** (Version 6.1) ******          Date 02-26-2016
                                                            BY: TJP
 Commands Read From File: C:\DS2B.CHL                       CHK: PCM

       JOB DS-2B
       UNI 0
 ** UNITS PARAMETER = 0 (ENGLISH)
       CHL 0.333  22.34
 *** WARNING:  DATA IS OUT OF REASONABLE RANGE
       TRP 4      2      2
 ** LEFT SIDE SLOPE    2.0 AND RIGHT SIDE SLOPE    2.0
 ** THE BASE WIDTH OF THE TRAPEZOID (ft)    4.00
       LRR 0.833  2.42     2.65   0.15
 ** D50 (ft)     .83                           D50 = 10 INCH
 ** ANGLE OF REPOSE (DEGREES)    42.00
 ** SPECIFIC GRAVITY    2.65
 ** SHIELDS PARAMETER    .150
       END
***************END OF COMMAND FILE************

 DS-2B
 ------------
 INPUT REVIEW
 ------------
   DESIGN PARAMETERS:
       DESIGN DISCHARGE (ft^3/s):          22.34
       CHANNEL SHAPE:                 TRAPEZOIDAL
       CHANNEL SLOPE (ft/ft):              .333
 ------------------------------------
 HYDRAULIC CALCULATIONS USING BATHURST
 ------------------------------------

 FLOW (cfs)                  22.34
 MAX DEPTH (ft)                .43
 AREA (ft^2)                  2.12
 WETTED PERIMETER (ft)        5.94
 HYDRAULIC RADIUS (ft)         .36
 AVG VELOCITY (ft/s)         10.56
 MANNINGS EQUIVALENT          .042
 Davg / D50                    .44
 FROUDE NUMBER                2.83
 REYNOLDS NUMBER (10^5)       1.74
 ------------------
 STABILITY ANALYSIS
 ------------------
```

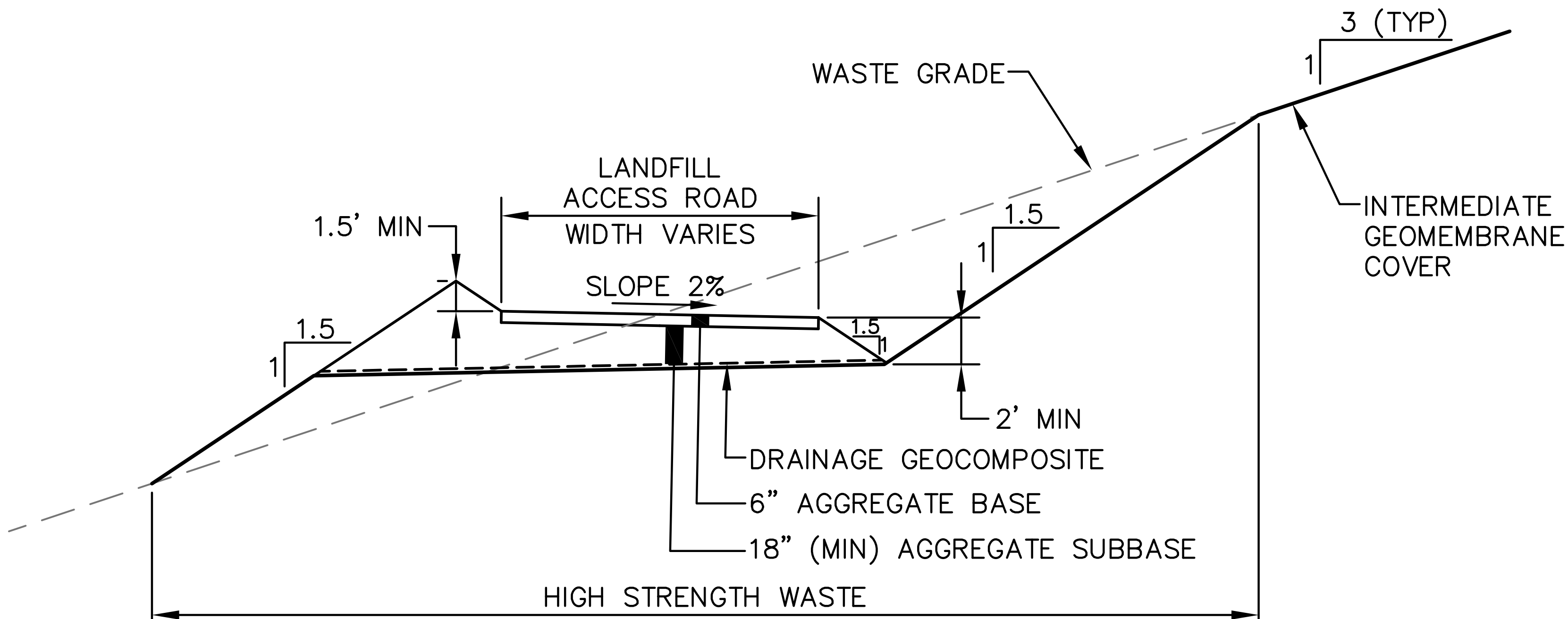
CONDITION	LINING TYPE	PERMIS SHR (LB/FT^2)	CALC. SHR (LB/FT^2)	STAB. FACTOR	REMARKS
BOTTOM; STRAIGHT	RIPRAP	12.86	8.99	1.43	STABLE
SIDE; STRAIGHT	RIPRAP	9.57	6.93	1.38	STABLE

*** NORMAL END OF HYCHL ***

D_{50} = 10 INCH OK

VOLUME IV APPENDIX E SUPPLEMENTAL FIGURES

- **FIGURE 6 LANDFILL ACCESS ROAD WITH INTERMEDIATE COVER**
- **FIGURE 7 SIDESLOPE INTERMEDIATE DOWNSPOUT SECTION**



LANDFILL ACCESS ROAD OVER INTERMEDIATE COVER

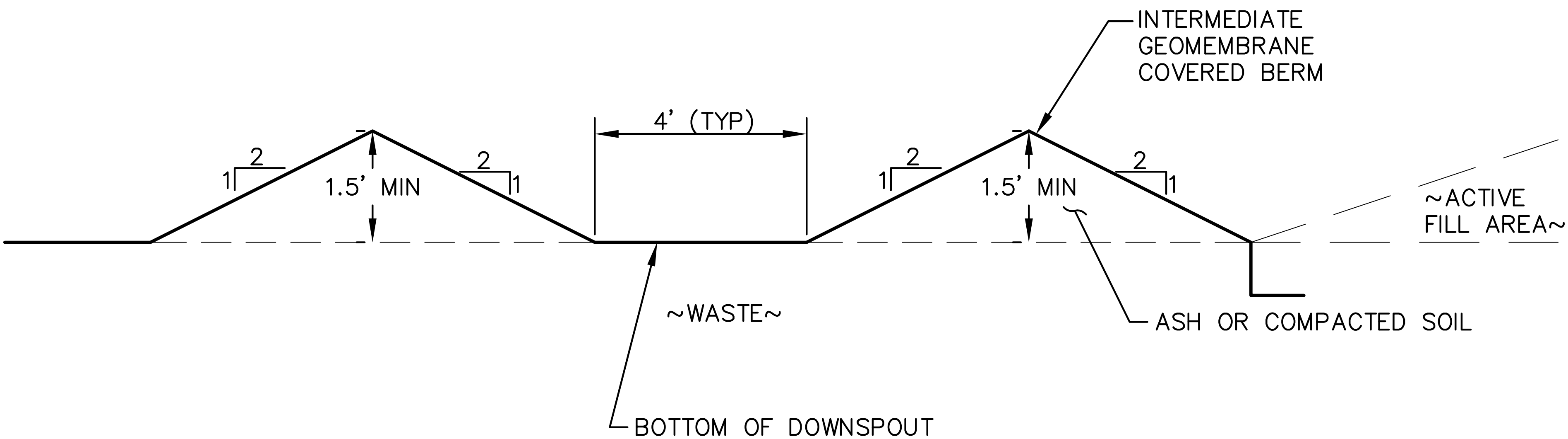
NTS

FIGURE 6
LANDFILL ACCESS ROAD
WITH INTERMEDIATE COVER
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL–FIGURES LMN: NONE CTB: SME–STD REV: 2/10/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\DETAIL-FIGURES.dwg, 2/10/2016 1:54:46 PM, sjm



SIDESLOPE INTERMEDIATE DOWNSPOUT SECTION

NTS

FIGURE 7
SIDESLOPE INTERMEDIATE
DOWNSPOUT SECTION
JUNIPER RIDGE LANDFILL EXPANSION
OLD TOWN, MAINE

SME
Sevee & Maher Engineers, Inc.
ENVIRONMENTAL • CIVIL • GEOTECHNICAL • WATER • COMPLIANCE

DWG: DETAIL-FIGURES LMN: NONE CTB: SME-STD REV: 2/23/2016

\\Nserver\cfs\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Project Regulatory Review\Acad\Figures\DETAIL-FIGURES.dwg, 2/23/2016 11:31:57 AM, sjm

SME 5

VOLUME III APPENDIX A UPDATED SPECIFICATIONS

• SECTION 02200	EARTHWORK
• SECTION 02272	GEOTEXTILES AND DRAINAGE GEOCOMPOSITE
• SECTION 02275	GEOSYNTHETIC CLAY LINER
• SECTION 02771	GEOMEMBRANE LINER, HIGH DENSITY POLYETHYLENE (HDPE)
• SECTION 02772	LEAK LOCATION SURVEY
• SECTION 02780	INTERFACIAL FRICTION ANGLE CONFORMANCE TESTING
• SECTION 15110	VALVES AND PIPE ACCESSORIES

VOLUME III APPENDIX B SECTION 5.0

- CONSTRUCTION QUALITY ASSURANCE MANUAL

SECTION 02200

EARTHWORK

PART 1 - GENERAL

1.01 RELATED DOCUMENTS:

A. The general provisions of the contract, including General and Supplementary Conditions and General Requirements (if any) apply to the work specified in this section.

B. The requirements set forth by the Quality Assurance/Quality Control Plan shall apply to work specified in this Section.

C. All work performed under this specification shall be performed in accordance with the Maine Department of Environmental Protection – Maine Erosion and Sedimentation Control Best Management Practices (October 2003).

1.02 DESCRIPTION: This work shall consist of survey layout, excavation, filling, and embankment construction, clay liner construction, underdrain and leachate collection sand placement, and grading including hauling, compaction, and disposal of all material encountered and necessary for construction of the project.

1.03 RELATED WORK SPECIFIED ELSEWHERE:

A. Subsurface Information: Section 02015

B. Site Preparation: Section 02100

C. Erosion Control: Section 02220

D. Geotextiles and Drainage Geocomposite: Section 02272

E. Pipe Installation: Section 02450

F. Corrugated Metal Pipe: Section 02451

G. Seeding and Mulching: Section 02800

H. Geosynthetic Clay Liner: Section 02275

I. Bituminous Pavement: Section 02510

J. Manholes, Catch Basins, and Drainage Structures: Section 02570

K. Cast-in-Place Concrete: Section 03300

1.04 REFERENCES

The publications listed below are part of this Technical Specification to the extent referenced. The publications are referred to in the text by the basic designation only.

A. American Association of State and Highway and Transportation Officials (AASHTO):

1. AASHTO T 104 Soundness of Aggregates by Use of Sodium Sulfate of Magnesium Sulfate
2. AASHTO T-289 Determining pH of Soil for Use in Corrosion Testing
3. AASHTO T-236 Direct Shear Test Of Soils Under Consolidated Drained Conditions

B. The American Society for Testing and Materials (ASTM):

1. ASTM D 422 Standard Test Method for Particle-Size Analysis of Soils;
2. ASTM D 698 Laboratory Compaction Characteristics of Soil Using Standard Effort
3. ASTM D 2216 Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
4. ASTM D 2434 Standard Test Method for Permeability of Granular Soils (Constant Head)
5. ASTM D 6938 Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)
7. ASTM D 4318 Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
8. ASTM D 3080 Standard Test Method for Direct Shear of Soils Under Consolidated Drained Conditions
9. ASTM D 5084 Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter

1.05 QUALITY ASSURANCE

A. All work pertaining to this Section shall be done according to the requirements outlined in the Quality Assurance/Quality Control Plan (Appendix B).

B. Codes and Standards: Perform excavation work in compliance with applicable requirements of governing authorities having jurisdiction.

C. Borrow Source Characterization Testing **(By Contractor)**:

1. The following borrow source characterization testing shall be performed on the materials used for the project's earthworks construction. The testing program will assure that borrow materials from on-site or off-site sources meet the requirements of this specification. Borrow source testing shall be performed prior to delivering off-site material to the site and prior to using on-site material for landfill construction. Changes in the borrow source and/or material properties shall be avoided. The Contractor shall employ a soils testing laboratory acceptable to the Engineer to perform soil testing of borrow source materials. Borrow source characterization will be performed in accordance with the Maine Department of Environmental Protection (MDEP) Solid Waste Management Regulations Chapter 401 Appendix A, or as directed by the Engineer.

Borrow source characterization testing on soil materials shall be done at the following frequencies:

TABLE 2-1

BORROW SOURCE CHARACTERIZATION TESTING

Borrow Material	Testing	Method	Frequency[1]
Underdrain and Drainage Sand[2]	Moisture Density[2]	ASTM D 698-91	1/2500 cy
	Remolded Hydraulic Conductivity	ASTM D2434-68	1/5000 cy
	Grain Size	ASTM D 422-63	1/2500 cy
	Moisture Content[3]	ASTM D 2216-92	1/2500 cy
	Calcium Carbonate[4] Content	ASTM D 4373 or approved equivalent	2 per Source
Drainage Stone	Grain Size	ASTM D 422-63	1/2500 cy
	Calcium Carbonate[4] Content	ASTM D 4373 or approved equivalent	2 per Source
Filter Stone	Grain Size	ASTM D 422-63	1/2500 cy
	Calcium Carbonate[4] Content	ASTM D 4373 or approved equivalent	2 per Source
Sump Stone	Grain Size	ASTM D 422-63	2 per Type
	Calcium Carbonate[3] Content	ASTM D 4373 or approved equivalent	2 per Source
Tire Chips (Type B)	Grain Size	ASTM C 136-05	2 per Source
Clay	Moisture Density	ASTM D 698-91	1/2500 cy
	Remolded Hydraulic Conductivity	ASTM D 5084-90	1/5000 cy
	Grain Size	ASTM D 422-63	1/2500 cy
	Moisture Content	ASTM D 2216-92	1/500 cy
	Atterberg Limits	ASTM D 4318-93	1/2500 cy
Common Borrow	Grain Size	ASTM D 422-63	1/2500 cy
	Moisture/Density	ASTM D 698-91	1/2500 cy
Structural Backfill	Grain Size	ASTM D 422-63	1/2500 cy
	Moisture/Density	ASTM D 698-91	1/2500 cy
Pipe Bedding	Grain Size	ASTM D 422-63	1/2500 cy
Aggregate Base and Aggregate Sub-Base	Grain Size	ASTM D 422-63	1/2500 cy
	Moisture/Density	ASTM D 698-91	1/2500 cy

Note:
1. Minimum of two tests per source.
2. Drainage Sand material includes both Leak Detection and Leachate Collection system sand.
3. Underdrain and Leak Detection Sand layers only.
4. Equivalent methods used to determine calcium carbonate content include Whole Rock Geochemistry ME XRF06 and ME ICP06.

1.06 SUBMITTALS:

A. The Contractor shall supply representative materials for testing as required by the Engineer. The Contractor shall schedule his operation and submissions so the Engineer has sufficient time to perform testing. Failing tests of materials quality, gradation, or field density will be charged to the Contractor and deducted from payments, in accordance with Part 1.05 of Section 01025.

B. Test Reports: Submit 2 copies of the borrow source test reports directly to the Engineer from the Contractor's testing subcontractor, with copy to the Contractor. If borrow source testing indicates a significant change in material index properties during the construction of the landfill liner, the in-place material testing specifications described herein should be modified.

1.07 JOB CONDITIONS:

A. Site Information: Data on indicated subsurface conditions are not intended as representations or warranties of accuracy or continuity between soil borings. It is expressly understood that Owner and/or Engineer will not be responsible for interpretations or conclusions drawn therefrom by Contractor. Data are made available for the convenience of Contractor.

Additional test borings and other exploratory operations may be made by Contractor at no cost to Owner.

B. Existing Utilities: The Contractor shall provide the services on On Target or Dig Safe to locate existing underground utilities in the areas of work, as necessary, or as required by law. If utilities are to remain in place, provide adequate means of protection during earthwork operations.

Should uncharted, or incorrectly charted, piping or other utilities be encountered during excavation, consult the Utility Owner immediately for directions. Cooperate with Owner and utilities companies in keeping respective services and facilities in operation. Repair damaged utilities to satisfaction of utility owner.

Do not interrupt existing utilities serving facilities occupied and used by Owner or others, except when permitted in writing by Engineer and then only after acceptable temporary utility services have been provided.

Demolish and completely remove from site existing underground utilities indicated to be removed. Coordinate with utility companies for shut-off of services if lines are active.

C. Use of Explosives: Blasting at the Juniper Ridge Landfill site is not permitted.

D. Protection of Persons and Property: Barricade open excavations occurring as part of this work and post with warning lights. Operate warning lights as recommended by authorities having jurisdiction. Protect structures, utilities, sidewalks, pavements, and other facilities from damage caused by settlement, lateral movement, undermining, washout, and other hazards created by earthwork operations. The Contractor shall comply with all applicable rules, procedures and regulations regarding safety as defined by all local, state, and federal agencies, including but not limited to, NEWSME Operations' safety procedures, State of Maine Labor Department Rules, and the Occupational Safety and Health Act (OSHA) Regulations.

E. Work related to geomembrane HDPE and geosynthetic clay liner installation:

The earthwork contractor shall provide the necessary labor, materials, equipment, and supervision to perform the following work in conjunction with the installation of the geomembrane HDPE liner and the geosynthetic clay liner.

- Excavation and backfill of all liner anchor trenches;
- Preparation of all liner subgrade materials including moisture control, picking stones as directed by the Engineer or CQA agent, providing a smooth rolled surface to the tolerances required by the Engineer or CQA agent;
- Coordination of work schedule to accommodate the liner installation including weekend and holiday work as necessary, and allowing sufficient turnaround time for results of destructive seam testing (typically 24 hours) prior to placing additional material over the HDPE seams;
- Placement of a minimum 3 feet of suitable material, approved by the Engineer or CQA agent, over the liner materials before traveling on the liner with equipment other than low-pressure equipment approved by the Engineer or CQA agent; and
- All other conditions noted in Section 00110 “Summary of Work”.

PART 2 - PRODUCTS

2.01 SOIL MATERIALS:

A. Roadways and Parking Lots:

1. Aggregate Subbase Material: Aggregate subbase shall be gravel consisting of hard, durable particles which are free from vegetation, lumps, or balls of clay, and other deleterious substances. Gravel subbase shall not contain particles of rock which will not pass the 6-in. square mesh sieve. The gradation of the portion which will pass a 3-inch sieve shall meet the grading requirements of the following table:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
1/4”	25-70
#40	0-30
#200	0- 7

(M.D.O.T. 703.06 Type D)

2. Aggregate Base Material: Shall be crushed gravel consisting of hard durable particles which are free from vegetable matter, lumps or balls of clay, and other deleterious substances. Gravel base shall not contain particles of rock which will not pass the 2-inch square mesh sieve. The gradation of the base materials shall meet the grading requirements of the following table:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
1/2”	45-70
1/4”	30-55
#40	0-20
#200	0- 5

(M.D.O.T. 703.06 Type A)

B. Structures:

1. Structural Backfill: Backfill placed within 5 ft of structures shall consist of a well graded sand free of vegetable matter, lumps, or balls of clay and other deleterious substances. The gradation shall meet the grading requirements of the following table:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
3”	100
#40	0- 70
#200	0- 10

Structural backfill shall not contain particles of rock which will not pass the 3-inch square mesh sieve.

C. Pipe Bedding Stone (Solid Wall Pipe):

Pipe Bedding Stone: Stone shall be obtained from rock of uniform quality and shall consist of clean, angular fragments of quarried rock free from soft disintegrated pieces or other questionable matter. The material shall meet the following gradation requirements:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
¾”	100
½”	85 -100
3/8”	0 - 15

D. Landfill:

1. Clay Borrow: Clay for landfill liner barrier soil, and imported soil layer shall be silty clay soil free of organic matter, debris, and frozen material. Cobbles and rock fragments larger than 1/2-inch in diameter shall not be permissible. Clay shall be capable of meeting the gradation and hydraulic conductivity requirements of this specification:

a) The specification for screened clay, 1/2" minus shall be as follows:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
½ "	100
#40	90-100
#200	75-100

b) Remolded Hydraulic conductivity (ASTM D 5084-90) maximum $\leq 1x10^{-7}$ cm/sec.

c) Liquid limit: $\geq$ 20
Plasticity index: 8 $\leq$ P.I. $\leq$30

2. Impervious Borrow: Clay for landfill anchor trench, and imported soil layer shall be silty clay soil free of organic matter, debris, and frozen material. Cobbles and rock fragments larger than 1/2-inch in diameter shall not be permissible. Clay shall be capable of meeting the gradation requirements of this specification:

a) The specification for screened clay, 1/2" minus shall be as follows:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
½ "	100
#40	90-100
#200	75-100

3. Drainage Stone: Shall be screened and washed stone free of organic matter, silt, or clay lumps, and deleterious material. The stone shall contain no more that 15% calcium carbonate as determined by ASTM D 4373 or equivalent method approved by Engineer (i.e., whole rock geochemistry methods). The material shall meet the following gradation requirements:

a) Pipe Drainage Stone: $D_{50} \geq 3/4$”

Sieve Designation	percent by Weight Passing Square Mesh Sieve
2”	100
1-1/2”	80-100
1”	50- 85
1/2”	0- 50
#4	0- 15
#10	0- 5

b) Leachate Sump Stone: will consist of rounded durable screened and washed stone or washed tailings meeting the calcium carbonate requirements above.

Sieve Designation	percent by Weight Passing Square Mesh Sieve
4”	100
2”	0

4. Filter Stone: Shall be screened and washed stone free of organic matter, silt, or clay lumps, and deleterious material. The stone shall contain no more that 15% calcium carbonate as determined by ASTM D 4373 or equivalent method approved by Engineer (i.e., whole rock geochemistry methods). The material shall meet the following gradation requirements:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
1”	100
No. 4	30-100
No. 20	0-20
No. 40	0-5

4. Underdrain Sand: Clean sand, free from organic matter, graded to meet the following criteria for the appropriate designation. Underdrain sand will have a hydraulic conductivity not less than $1x10^{-3}$ cm/sec and average of $1x10^{-2}$ cm/sec as determined by ASTM D 2434. The sand shall contain no more that 15% calcium carbonate as determined by ASTM D 4373 or equivalent method approved by Engineer (i.e., whole rock geochemistry methods).

   Gradation Requirements:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
1”	100
1/2”	90-100
#4	70 – 100
#10	50 - 85
#20	35 – 70
#60	0- 40
#200	0- 5[1]

1.Based on the portion passing the U.S. Standard Size No. 4.

5. Drainage Sand: Must meet the underdrain sand gradation stated above and have a remolded hydraulic conductivity average $1x10^{-2}$ cm/sec, minimum $5x10^{-3}$ cm/sec. as determined by ASTM D2434. The sand shall contain no more that 15% calcium carbonate as determined by ASTM D 4373 or equivalent method approved by Engineer (i.e., whole rock geochemistry methods).

6. Tire Chips: Type B shredded tire chips graded to meet the following criteria for the appropriate designation. Shall be free of contaminants such as oil, grease, gasoline, diesel fuel, etc. that could create a fire hazard. No remains of tires that have been subjected to a fire. Tire chips shall be from fragments of wood, wood chips and other fibrous organic matter. Tire chips shall have less than 1% (by weight) of metal fragments that are not at least partially encased in rubber. Metal fragments that are partially encased in rubber shall protrude no more than 1 inch from the cut edge of the tire shred on 75% of the pieces (by weight) and no more than 2 inches on 90% of the pieces (by weight)

Gradation Requirements:

Sieve Designation	percent by Weight Passing Square Mesh Sieve
18”	100
12”	90
8”	75
3”	50
1-1/2”	25
#4	1

F. Miscellaneous Materials:

1. Common Borrow: Shall be earth, suitable for embankment construction. It shall be free of frozen material, perishable rubbish, peat, organic matter, large rock fragments, or other unsuitable material. AASHTO M145 Classifications A-1 through A-5 may be used. Use of other materials as common borrow is at the discretion of the Engineer and only in approved areas.

2. Topsoil: Shall be in accordance with Section 02800 – Seeding and Mulching.

3. Riprap: Shall be in accordance with Section 02220 – Erosion Control.

4. Frost Protection Layer: Suitable frost protection material shall include bark, wood chips, sawdust, municipal solid waste, and/or other materials approved by the Owner and the MEDEP.

2.02 ON-SITE MATERIAL

A. Material on the site is the property of NEWSME Operations and may only be used with the approval of the Engineer. The Contractor will complete all borrow source testing requirements specified in Table 2-1, if they wish to use any on-site material.

B. Material not incorporated in the work because it is unsuitable will be hauled away and disposed of at the Contractor's expense.

1. Material designated to be saved by the Engineer will be stockpiled using MEDEP Erosion Control BMPs for construction, at a location shown on the drawings or designated by the Engineer.

2. Unsuitable material shall consist of grubbings or other materials which contain rock of size exceeding specifications, organic materials, frozen materials, or other materials of a deleterious nature as deemed by the Engineer.

PART 3 - EXECUTION

3.01 INSPECTION: Examine the areas and conditions under which excavating, filling, and grading are to be performed and notify the Engineer, in writing of conditions detrimental to the proper and timely completion of the work. Do not proceed with the work until unsatisfactory conditions have been corrected in an acceptable manner.

3.02 EXCAVATION: Excavation consists of removal and disposal of material encountered when establishing required grade elevations. All excavation shall be unclassified and shall include any and all material encountered. No extra compensation shall be allowed for excavation work covered by the bid proposal.

A. Excavation for Structures: Conform to elevations and dimensions shown within a vertical tolerance of one-half (1/2) in. and extending a sufficient horizontal distance from footings and foundations to permit placing and removal of concrete formwork, installation of services, other construction, and for inspection.

Excavation for footings and foundations shall extend to the depth necessary to remove all fill material above the native soils. When the footing and foundation grades extend into native soils, the native soil shall be excavated to the foundation grades specified on the plans.

In excavating for footings and foundations, take care not to disturb bottom of excavation. Excavate by hand to final grade just before concrete reinforcement is placed. Trim bottoms to required lines and grades to leave solid base to receive concrete.

Rock shattered due to drilling or ripping operations shall be removed. Excess rock excavation shall be filled with Class A or Class B concrete.

B. Excavation for Pavements: Conform to subgrade elevations and dimensions shown, within a vertical tolerance of one (1) in.

C. Excavation for Trenches: Conform to elevations and dimensions within a vertical tolerance of one (1) in. Excavate to the uniform width shown or required for the particular item to be installed. Provide adequate working space for compactive equipment.

Excavate trenches to the depth indicated or required. Carry the depth of trenches for piping to establish the indicated flow lines and invert elevations and provide suitable bedding. Pipe bedding as specified in paragraph 2.01C.

Where rock is encountered, carry the excavation six (6) in. below the required elevation and backfill with a 6" layer of crushed stone or gravel prior to installing pipe.

Grade bottoms of trenches as indicated, notching under pipe joints to provide solid bearing for the entire body of the pipe.

Do not backfill trenches until authorized by the Engineer. Use care in backfilling to avoid damage or displacement of pipe systems.

D. Site Excavation: Conform to elevations and dimensions shown within a vertical tolerance of one-tenth (0.1) of a foot. During the excavation to base grade, excavating equipment and trucks are to be kept off the subgrade to minimize disturbance of the subgrade. Excavate to a depth to provide for any subsequent loam, sod, or other specified surface material.

E. Excavation of Unsuitable Material: Shall consist of the excavation and removal of all fill materials including loose, uncompacted soils material, buried rubber tires and waste, buried vegetation and other organic or inorganic debris shown on the plans, encountered during the prosecution of the work, or as directed by the Engineer. The excavation shall extend to the limits and depth necessary to remove fill and unsuitable material as directed by the Resident Project Representative.

F. Muck Excavation: Muck excavation shall consist of the excavation and disposal of saturated and unsaturated mixtures soils and organic matter not suitable for foundation or embankment material, regardless of moisture content.

3.03 STABILITY OF EXCAVATIONS: Slope sides of excavations to comply with local codes and ordinances having jurisdiction. Shore and brace where sloping is not possible because of space restrictions or stability of material excavated.

Maintain sides and slopes of excavations in a safe condition until completion of backfilling.

3.04 SHORING AND BRACING: Provide materials for shoring and bracing, such as sheet piling, uprights, stringers, and cross-braces, in good serviceable condition.

Establish requirements for trench shoring and bracing to comply with local codes and authorities having jurisdiction.

Maintain shoring and bracing in excavations regardless of time period excavations will be open. Carry down shoring and bracing as excavation progresses.

Provide permanent steel sheet piling or pressure treated timber sheet piling wherever subsequent removal of sheet piling might permit lateral movement of soil under adjacent structures. Cut off tops as required and leave permanently in place.

3.05 MATERIAL STORAGE: Stockpile satisfactory excavated materials where directed, until required for backfill or fill. Place, grade, and shape stockpiles for proper drainage. Seed and mulch stockpiles in accordance with MDEP BMPs as necessary.

Locate and retain soil materials away from edge of excavations.

Dispose of excess soil material and waste materials as herein specified.

3.06 COLD WEATHER PROTECTION: Protect excavation bottoms against freezing when atmospheric temperature is less than 35^{o}F.

3.07 WINTER CONSTRUCTION OF EMBANKMENTS AND CLAY BARRIERS: Frozen material shall not be placed in the embankment or clay barriers. The construction of embankments may continue during cold weather only when all frozen material in the top of the embankment or the existing ground is moved to the waste storage area, or removed from the site, before placing additional material.

Compaction shall be in accordance with the specified method of embankment and clay barriers construction. When the prevailing temperatures are below 30 deg.F, all material used in embankment construction and clay barriers shall have a moisture content at the time of compaction equal to or less than the optimum moisture content.

The embankment shall not be constructed upon frozen material except that such construction of embankments outside the building area may be allowed providing the total depth of the added fill, including gravel bases, plus the depth of the frozen material beneath does not exceed 5 ft. Frozen material may be left in the embankment only if it has been compacted as specified prior to freezing. The Contractor shall not resume construction of any embankments built in this manner until all frozen material has thawed. If test holes are required to make this determination they shall be dug and backfilled with satisfactory compaction at the Contractor's expense. Before additional material is added, uncompacted material on the surface of such embankments shall be either recompacted in accordance with the specified method of embankment construction or removed. Clay layers shall not be constructed upon frozen material. The Contractor shall be responsible for protecting any clay barrier material placed from freezing. If previously placed clay material freezes, it shall be removed, thawed, and then replaced.

3.08 CLOSING ABANDONED UNDERGROUND UTILITIES: Close open ends of abandoned underground utilities, indicated to remain, permanently with closures sufficiently strong to withstand pressures which may result after closing.

Close open ends of metallic conduit and pipe with threaded galvanized metal caps or plastic plugs, or other suitable method for the type of material and size of pipe. Do not use wood plugs.

Close open ends of plastic PVC pipe abandoned in-place with a permanent plastic PVC plug. Pipe end is to be backfilled with bedding sand.

Close open ends of concrete and masonry utilities with not less than 8" thick brick masonry bulkheads, constructed to completely fill the opening.

Wet brick before laying. Lay brick in mortar so as to form a full bed with ends and side joints in one operation. Joints shall be more than three-eighths (3/8) in. wide. Protect fresh masonry from freezing or from rapid drying, as necessary, and maintain protection until mortar has set.

3.09 BACKFILL AND FILL:

A. General: Place acceptable soil material in layers to required subgrade elevations, for each area classification listed below.

1. In excavations, use satisfactory excavated or borrow material.

2. Under grassed areas, use satisfactory excavated or borrow material.

3. In pipe trenches, use material specified in typical trench section.

4. Landfill Base: Use satisfactory regraded fill material, till soils excavated from on-site, and/or clay borrow meeting the requirements of SWMR Ch. 401.2.D(3).

B. Backfill excavations as promptly as work permits, but not until completion of the following:

1. Acceptance by Engineer of construction below finish grade including, but not limited to, dampproofing, waterproofing, and perimeter insulation.

2. Inspection, testing, approval, and recording locations of underground utilities.

3. Removal of shoring and bracing, and backfilling of voids with satisfactory materials. Temporary sheet piling driven below bottom of structures shall be removed in a manner to prevent settlement of structures or utilities, or cut-off and left-in-place if required.

4. Removal of trash, debris, frozen and/or unsuitable materials.

C. Placement and Compaction: Place backfill and fill materials in layers not more than 12" compacted depth for material compacted by heavy compaction equipment, and not more than 6" in loose depth for material compacted by hand-operated tampers. Except for clay as noted below.

Before compaction, moisten or aerate each layer as necessary to provide the optimum moisture content. Compact each layer to required percentage of maximum dry density for each area classification. Do not place backfill or fill material on surfaces that are muddy, frozen, or contain frost or ice.

Prior to the placement of a layer of clay material or base till to reach subgrades in the landfill, at test pad program shall be completed to demonstrate that he compaction techniques used can achieved the required performance standards. The test pad shall include: an area of approximately 50,000 square feet within the cell or cover area. The test pad shall encompass the transition from base liner to perimeter berm. The test pads for the secondary liner system shall encompass the transition from standard liner to augmented liner where applicable. Test pads for cover system shall encompass the transition from topslope to sideslope. The test pad program will consist of placing and compacting the clay in one, or two lifts, each with a compacted thickness of 12 inches. During placement and compaction in-place moisture/density tests will be obtained at a frequency of 15 per acre per lift. Provided the field test demonstrates an adequate compaction effort has been obtained through the entire lift thickness, the type of compaction, (e.g. compaction equipment and number of passes) will be noted in the field logs. In-place hydraulic conductivity tests will be randomly collected from the test pad area with a total of 7 hydraulic conductivity tests collected for each one-foot lift. The hydraulic conductivity tests will be performed on clay samples representative of the top, middle, and bottom of each lift. If the clay is place with two 12 inch lifts, the top lift samples will test the interface between the two lifts. These hydraulic conductivity tests will be sent to the laboratory and the hydraulic conductivity, density, and moisture content of the in-place clay measured. If the field and laboratory tests confirm that this construction technique can achieve the project performance criteria up to four shallow test pits will be excavated in the test pad area to evaluate the remolding, and bonding of the clay through visual observations. Pictures will be used to visually document the condition of the compacted clay. The data collected and testing results will be compiled and discussed the construction stakeholders and upon concurrence that the construction techniques are appropriate to achieve the specified in-place properties of the clay the technique will be employed for the remainder of the project

Mating of new clay lifts with previously placed clay lifts shall be done by excavating the mating edge of the existing clay lifts in a stepped manner. Each step shall have a vertical height no greater than 12 inches and a horizontal width of 4 to 6 feet. The surface of each of the steps in the old clay layer shall be scarified to maximize bonding between the new and old sections.

Place backfill and fill materials evenly adjacent to structures, to required elevations. Take care to prevent wedging action of backfill against structures by carrying the material uniformly around structure to approximately same elevation in each lift.

Equipment such as bulldozers and sheepsfoot drum rollers will be used during the placement and compaction of the clay liner. Grading of the clay material will utilize laser survey technology to eliminate the need for grade stakes. The type of equipment, operating speed, and number of passes to adequately compact the clay soils shall be determined by ability of equipment to achieve performance requirements and is to be approved by the Engineer. In addition, extra precaution will be taken with the equipment used for placement and compaction to avoid sudden turns, stops, or starts that could disturb the clay liner. Similar equipment and precautions will be utilized during the placement of sand above the HDPE geomembrane. The equipment used in placement of materials on top of the geomembrane shall be approved by the Engineer.

A smooth drum roller is to be used to seal the lifts. Sealing the lifts will encourage runoff from storms, thus limiting development of excessively moist or wet lenses of soil within the barrier layer. The lift surface shall be scarified or otherwise roughened by tracking with a bulldozer prior to placing the next lift of material to promote good bonding between lifts.

At the completion of the clay and prior to the installation of HDPE geomembrane, the clay surface shall be proofrolled with 3 passes of a smooth drummed 10-ton vibratory roller.

Equipment used to construct the landfill shall be approved by the Engineer prior to or during the pre-construction meeting.

3.10 GRADING:

A. General: Uniformly grade areas within limits of grading under this section, including adjacent transition areas. Smooth finished surface within specified tolerances, compact with uniform levels or slopes between points where elevations are shown, or between such points and existing grades.

B. Compaction: After grading, compact subgrade surfaces to the depth and percentage of maximum density for each area classification.

3.11 COMPACTION:

A. General: Control soil compaction during construction providing minimum percentage of density specified for each area classification.

B. Percentage of Maximum Density Requirements: Compact soil within 4 percent above the optimum moisture content, to not less than the following percentages of maximum dry density (determined in accordance with ASTM D 698).

1. Landfill:

a) Subgrades: Compact subgrades below clay layer to at least 90 percent of maximum dry density.

Fill soil approved to be left in place shall be proofrolled prior to clay placement. Proofrolling shall include a minimum of three passes of a heavy vibratory compactor. The type and weight of the compactor shall be approved by a qualified geotechnical engineer. The engineer shall recommend removal of any unsatisfactory fill material, approve compaction equipment, and observe the compaction effort.

b) Clay Layers: Compact clay layer material to achieve the following properties:

In-Place Hydraulic Conductivity	$\leq 1 \times 10^{-7}$ cm/sec at 90 percent standard proctor density
Moisture Content	0-4 percent above optimum

Compaction test frequency of clay material will be at 9/acre/lift.

c) Embankments and Anchor Trench: Compact soil at least 90 percent of maximum dry density. Embankments compaction test frequency: 1 test/100 lf/lift. Anchor trench compaction test frequency at Engineer's discretion.

d) Underdrain Sand: Compact sand to at least 90 percent of maximum dry density. Compaction test frequency at Engineer's discretion.

2. Footings and Foundations:

a. Footings founded on native sands: Compact subgrade with at least six complete passes of an approved vibratory plate.

b. Compact each layer of base material to at least 95 percent of maximum dry density.

3. Structural Slabs: Compact each layer of backfill material to at least 95 percent of maximum dry density.

4. Adjacent to Structures: Compact each layer backfill or fill material to at least 92 percent of maximum dry density.

5. Lawn or Unpaved Areas: Compact each layer backfill or fill material to at least 85 percent of maximum dry density.

6. Pavements: Compact subgrade and each layer of gravel borrow, subbase material, and base material to at least 95 percent of maximum dry density.

7. Pipe Trenches: Compact bedding material and each layer of backfill to six (6) inches over pipe to at least 90 percent maximum dry density.

C. Moisture Control: Where subgrade or a layer of soil material must be moisture conditioned before compaction, uniformly apply water to surface of subgrade, or layer of soil material, in proper quantities to prevent free water appearing on surface during or subsequent to compaction operations.

Remove and replace, or scarify and air dry, soil material that is too wet to permit compaction to specified density.

Soil material that has been removed because it is too wet to permit compaction may be stockpiled or spread and allowed to dry. Assist drying by discing, harrowing, or pulverizing until moisture content is reduced to a satisfactory level.
To control clay layer desiccation during dry conditions, the liner will be moisture conditioned as described above and a 4-mil plastic cover will be temporarily placed over the clay. Alternate plans to control clay layer desiccation shall be approved by the Project Manager.

3.12 BASE AND SUBBASE COURSES:

A. General: This work consists of placing aggregate base and subbase material, in layers of specified thickness, over subgrade surface and geotextile fabric.

See Section 02510 for paving specifications.

B. Grade Control: During construction, maintain lines and grades including crown and cross-slope of subbase course.

C. Placing: Place subbase and base course material on prepared surfaces in layers of uniform thickness, conforming to indicated cross-section and thickness. Maintain optimum moisture content for compacting materrial during placement operations.

When a compacted subbase course is shown to be 6" thick or less, place material in a single layer. When shown to be more than 6" thick, place material in equal layers, except no single layer more than 12" or less than 3" in thickness when compacted.

3.13 VEGETATIVE COVER:

A. General: This work consists of placing vegetative cover soil of the specified thickness on prepared subgrade in all areas disturbed by construction and not otherwise surfaced or covered by structures and shall be in accordance with Section 02800 – Seeding and Mulching.

B. Material: Use suitable loam stripped from site where possible.

C. Placing and Grading: Place loam at the locations at specified thickness. Grade and rake loam to remove all foreign material and rocks over two (2) inches. Leave the surface uncompacted to receive the seeding operations.

3.14 FIELD QUALITY CONTROL **(Testing Services by Owner)**:

Quality Control Testing During Construction: The Owner will perform quality control testing of materials used in the work (i.e., borrow source construction testing and in-place construction testing, in accordance with the Maine Department of Environmental Protection (MDEP) Solid Waste Management Regulations Chapter 401 Appendix A). The Contractor shall supply representative materials for testing as required by the Owner's Representative. The Contractor shall schedule his operation and submissions so the Owner's Representative has sufficient time to perform material quality control testing. Failing quality control tests of materials will be charged to the Contractor and deducted from payments. Quality control procedures outlined in the site's Quality Assurance/Quality Control Plan (Revision November 2006), shall be implemented by the Contractor during construction at the landfill. The Contractor will allow the Owner's testing service to examine and test subgrades and fill layers before further construction work is performed. Test results meeting the requirements of 3.09 herein, shall be obtained prior to placing additional materials.

A. Borrow Source Construction Testing: The following borrow source quality control testing of the screened till, clay, granular drainage material, and underdrain material as a minimum shall be required. Samples to be tested will be collected by the Owner's Representative. Samples will be taken from material that has arrived on-site in accordance with ASTM D420 and C702.

TABLE 3-1

BORROW SOURCE CONSTRUCTION TESTING

Material	Tests	Frequency
Clay - Soil Material	Moisture Density ASTM 698	1/2,500 cy
	Grain Size ASTM 422	1/2,500 cy
	Moisture Content ASTM D 2216	1/500 cy
Underdrain and Drainage Sand [1]	Moisture Density ASTM 698[2]	1/2,500 cy
	Grain Size ASTM D 422	1/1,000 cy
	Moisture Content ASTM D 2216[2]	1/500 cy
Drainage Stone	Grain Size ASTM D 422	1/500 cy
Filter Stone	Grain Size ASTM D 422	1/500 cy
Sump Stone	Grain Size ASTM D-422-3	1/500 cy

1. Drainage Sand material includes both Leak Detection and Leachate Collection system sand.
2. Underdrain and Leak Detection sand layers only.

B. In-Place Construction Testing: The following in-place construction quality control testing shall be performed.

TABLE 3-2

IN-PLACE CONSTRUCTION TESTING

Material	Tests	Frequency
Clay Soil Layer	Density ASTM D 6938	9/acre/lift
	Moisture Content ASTM D 6938	9/acre/lift
	Moisture Content ASTM D 2216	1/9 nuclear method test
	Undisturbed Hydraulic Conductivity ASTM 5084	5/acre/lift
	Thickness	5/acre/lift
	Lift Interface Bond and Remolding	5/acre/lift
Underdrain and Drainage Sand[1]	Density ASTM D 6938[2]	5/acre/lift
	Moisture Content ASTM D 6938[2]	5/acre/lift
	Thickness	5/acre/lift
	Remolded Hydraulic Conductivity ASTM D 2434	5/acre/lift

Material	Tests	Frequency
Embankments	Density ASTM D 6938	1/100/LF/LIFT
Retained Fill (Common Borrow)	Density ASTM D 6938	1/lift/100 LF
	Moisture Content ASTM D 6938	1/lift/100 LF

1. Drainage Sand material includes both Leak Detection and Leachate Collection system sand.
2. Underdrain and Leak Detection Sand layers only.

Perform field density tests in accordance with ASTM D1556 (sand cone method), ASTM D2167 (rubber balloon method), or ASTM D6938 (Nuclear Device), as applicable.

1. Measurement of Soil Thickness and Soil Characteristics: The Contractor shall maintain daily logs of measurements of soil lifts, soil characteristics, and other observations. The thickness of clay liner and the underdrain sand layers shall be recorded at the required frequency. The thickness of the coarse aggregate shall be recorded once per 100 lineal feet of material placed.

2. The Owner's Representative shall identify test locations. The Contractor shall assist the Owner's Representative in obtaining the required samples. The location of in-place clay and drainage sand layer tests shall be determined by gridding the placement area plan into 75-foot squares. A total of 8 squares will constitute an acre. Each grid intersection will be assigned a number and a random number generator will be used to select the locations to be tested for each lift. Test sample locations for in-place hydraulic conductivity, moisture density, and thickness will be selected in this manner. The Owner's Representative shall maintain daily logs of measurements of soil lifts soil characteristics and other observations. The thickness of the drainage stone shall be recorded at 100-foot intervals.

3. Sampling clay for hydraulic conductivity will entail collecting the clay sample with a thin-walled tube. The tube will be jacked slowly into the 9-inch clay lift to a depth $\geq$9 inches using a hydraulic jack. The tube will be removed by slowly twisting the tube 360° while slowly pulling the tube from the clay. Once successfully sampled, the tube ends are capped and taped. If the samples are not to be tested immediately, the ends will be sealed by waxing both ends of the clay sample. The tube is then labeled with the site name, sample location, date, and time. Samples shall be transported vertically to the Owner's contracted testing laboratory. Samples will be tested for hydraulic conductivity according to ASTM 5084-90.

4. Deficient Areas of Work: Contractor placed subgrades and fills that are found deficient by inspection and/or failing quality control testing will be reworked. Subgrades and clay borrow that are below specified density or thickness, or outside the specified hydraulic conductivity range will be corrected at the Contractor's expense and the Contractor will reimburse the Owner for additional testing expenses. The Owner has the option of retesting the area in question prior to the Contractor taking corrective action. The deficient area is defined as the area encompassing half the distance surrounding sampling points which satisfactorily meet the testing requirements. The Contractor can elect to rework a rejected area; if it fails again the Contractor must remove the failing material and start anew with new material. The reworked area will be inspected and tested until the area is in conformance. An area which has been rejected for

nonconformance due to deficient specified thickness or grading requirements may be reworked by the Contractor until it meets the specification, provided this work does not cause the rejected block to deviate from the other requirements of conformance.

5. Clay Sample Holes:

Holes formed in the clay layer by the removal of thin-walled tube samples and moisture density testing, or for other testing or sampling, shall be backfilled by the Contractor to previous grade. The backfill will consist of either bentonite chips or compacted clay such that the hydraulic conductivity requirements for the clay layer are satisfied.

3.15 SLAB AND FOUNDATION WALL BACKFILL

Take at least 2 field density tests, at location and elevation as directed by Engineer.

3.17 MAINTENANCE

A. Protection of Graded Areas: Protect newly graded areas from traffic and erosion. Keep free of trash and debris.

Repair and re-establish grades in settled, eroded, and rutted areas to specified tolerances.

Control of clay liner desiccation shall be in accordance with Section 3.09(C).

B. Reconditioning Compacted Areas: Where completed compacted areas are disturbed by subsequent construction operations or adverse weather, scarify surface, re-shape, and compact to required density prior to further construction. This work shall be performed at no additional cost to the Owner.

3.18 DISPOSAL OF EXCESS AND WASTE MATERIALS:

Remove waste materials, including unacceptable excavated material, trash and debris, and dispose of it in areas designated by Owner/Engineer.

END OF SECTION

SECTION 02272

GEOTEXTILES AND DRAINAGE GEOCOMPOSITE

PART 1 - GENERAL

1.01 RELATED DOCUMENTS

A. Drawings and General Terms and Conditions as outlined in Section 1 of the Construction Agreement and Division-1 Specification sections, apply to work of this section.

B. Requirements set forth by the project's Quality Assurance/Quality Control Plan shall apply to the work specified in this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE:

A. Earthwork: Section 02200

B. Erosion Control: Section 02220

C. Geomembrane Liner: Section 02771

D. Interfacial Friction Angle Conformance Testing: Section 02780

1.03 DESCRIPTION:

A. Furnish and install woven, non-woven geotextiles, or geonet, whichever is called for at the locations and in the manner shown on the drawings or as directed by the Engineer.

1.04 SUBMITTALS:

A. If brand name materials other than those suggested in this Section are proposed for use, furnish certified copy of laboratory test results and material sample as evidence that the material is similar and equal in strength, durability, and permeability.

B. Submit, 2 copies plus number of copies required by the contractor, of quality control documentation described in 1.05 A, B and C herein, and 6-1 and 7-1 of Appendix B - Quality Assurance Quality Control Plan. .

1.05 QUALITY CONTROL

A. Manufacturer's Experience: The manufacturer supplying the geosynthetic materials shall satisfactorily demonstrate previous experience by letter of certification. Certification shall indicate that the manufacturer has produced, and has in service in similar applications for a period of not less than one (1) year, at least fifteen (15) million sq ft of geotextiles and drainage net meeting these Specifications.

B. Installer's Experience: The Installer proposing to install the lining shall satisfactorily demonstrate previous experience by letter of certification. Certification shall indicate the Installer's successful past installation of at least 5,000,000 sq ft of geotextiles and drainage net.

C. Prior to the installation of any geotextile and drainage geocomposite, the Manufacturer or Installer shall provide the Project Manager with the following information:

1. The origin (resin supplier's name and resin production plant) and identification (brand name and number) of the resin used to manufacture the geotextile.

2. Copies of the quality control certificates issued by the resin supplier.

3. Reports on tests conducted by the Manufacturer to verify that the quality of the resin used to manufacture the geotextile meets the Manufacturer's resin specifications.

4. Reports on quality control tests conducted by the Manufacturer to verify that the geotextile manufactured for the project meets the project specifications.

5. A statement indicating that the reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness.

6. A list of the materials which comprise the geotextile, expressed in the following categories as percent by weight: base polymer, carbon black, other additives.

7. A specification for the geotextile which includes all properties contained in the project specifications measured using the appropriate test methods.

8. Written certification that minimum average roll values given in the specification are guaranteed by the Manufacturer.

9. For non-woven geotextiles, written certification that the Manufacturer has continuously inspected the geotextile for the presence of needles and found the geotextile to be needle free.

10. Quality control certificates, signed by a responsible party employed by the Manufacturer. The quality control certificates shall include roll identification numbers, sampling procedures and results of quality control tests. At a minimum, results shall be given for:

<u>Geotextile</u>

a. Mass per unit area
b. Grab tensile/elongation
c. Trapezoidal tear strength
d. Puncture
e. Apparent opening size
f. Water permeability

<u>Geonet</u>

a. Density
b. Carbon Black
c. Thickness

Drainage Geocomposite

a. Transmissivity
b. Ply adhesion
c. Tensile strength

Quality control tests shall be performed in accordance with the test methods specified in Section 2.01B and 2.01C for at least every 100,000 ft^2 (10,000 m^2) of geonet and geotextile produced.

The Manufacturer shall identify all rolls of geotextiles and drainage geocomposite with the following:

1. Manufacturer's name
2. Product identification
3. Roll number
4. Roll dimensions

1.06 CONFORMANCE TESTING:

The geocomposite will be sampled by the Geosynthetic CQA or his agent(s), at the manufacturer or upon delivery to the site. Conformance samples shall be collected in accordance with the project's Quality Assurance Quality Control Plan. The Geosynthetic CQA shall assure that conformance test samples are obtained for the geocomposite at a rate of 1 test per 100,000 square feet of each geotextile and geocomposite with a minimum of once per lot, for testing, to assure conformance to the specification. A lot consists of a group of material which is manufactured from a specific batch of raw materials (e.g., HDPE resin, or bentonite clay). The manufacturer shall identify the consecutively numbered rolls of material that are inclusive within a lot. It is not required that all rolls included in a lot be supplied to the project, as long as the specified certification test results are supplied by the manufacture to bracket the rolls delivered to the project. Conformance tests shall be performed in accordance with the test methods specified in Section 2.01B and 2.01C. Interfacial friction angle testing shall be done according to Section 02780 and at a frequency as indicated in Section 02780.2.01(D).

The contractor or sub-contractor shall coordinate information on manufacture and delivery of the geocomposite with the Geosynthetic CQA to assure that sampling and testing occur in a timely manner as to avoid construction delays.

Any further testing required to assure conformance shall be the responsibility of the contractor in accordance with 1.05 (C) of Section 01025. The testing laboratory and the results shall be acceptable to the Engineer.

Geocomposite conformance testing shall include the following:

1. Geotextile

Mass per unit area
Grab tensile/elongation
Trapezoidal tear strength
Puncture

2. Drainage Geocomposite

Transmissivity
Ply adhesion
Tensile strength
Interfacial friction angle (ref. Specification Section 02780)

(a) Between drainage geocomposite and drainage sand

(b) Between drainage geocomposite and textured HDPE geomembrane

PART 2 - PRODUCTS

2.01 MATERIALS:

A. Woven Geotextile:

1. Synthetic fibers woven to provide a strong, water permeable material.

2. Fiber shall be made from polypropylene.

3. Edges: Selvaged to prevent raveling.

4. Geotextile shall be resistant to rot, mildew, insects, rodents, salt water and other biological and chemical substances commonly encountered in the ground.

5. Woven Geotextiles:

a) Roadway Stabilization Geotextile: The geotextile used in construction of the landfill access roads shall be Mirafi HP-270.

b) Riprap/Plunge Pool Geotextile Filter: The geotextile used in construction of riprap ditches and or riprap spillways shall be Mirafi® Filterweave FW700 as produced by Ten Cate Nicolon or approved equal.

c) Underdrain Pipe Geotextile: The geotextile used to wrap the underdrain pipe and stone shall be Mirafi® Filterweave FW300 as produced by Ten Cate Nicolon or approved equal.

B. Non-Woven Geotextiles:

1. Produced by heat bonding, needle punching or by the use of external adhesives.

2. The network of fibers shall be bonded so the fibers will retain their relative position with respect to each other.

3. Fibers may be made from polypropylene.

4. Geotextile shall be resistant to rot, mildew, insects, salt water, rodents and any other biological and chemical substances commonly encountered in the ground.

5. Non-Woven Geotextiles:

a) 6 oz/sy: The geotextile used as a filter fabric and heat bonded to both sides of the HDPE drainage net shall meet the Minimum Average Roll Values (MARV) listed below.

Property	Unit	Test Method	MARV
Weight	oz/yd^2	ASTM D 5261-92	6
Grab Strength	lbs	ASTM D 4632-91	150
Trapezoid Tear Strength	lbs	ASTM D 4533-91	60
Puncture Strength	lbs	ASTM D 6241	90
Apparent Opening Size	Std Sieve	ASTM D 4751-95	70
Permittivity	Sec^{-1}	ASTM D 4491-99a	1.6
UV Resistance	% min. 500 hr	ASTM D 4355	70

b) 8 oz/sy: The geotextile used for leak detection piping and riprap downspouts shall meet the Minimum Average Roll Values listed below:

Property	Unit	Test Method	MARV
Weight	oz/yd^2	ASTM D 5261-92	8
Grab Strength	lbs	ASTM D 4632-91	225
Trapezoid Tear Strength	lbs	ASTM D 4533-91	90
Puncture Resistance	lbs	ASTM D 6241	600
Apparent Opening Size	Std Sieve	ASTM D 4751-95	80
Permittivity	Sec^{-1}	ASTM D 4491-99a	1.26
UV Resistance	% min. 500 hr	ASTM D 4355	70

c) 10 oz/sy: The geotextile used for landfill gas header pipes shall meet the Minimum Average Roll Values listed below:

Property	Unit	Test Method	MARV
Weight	oz/yd^2	ASTM D 5261-92	10
Grab Strength	lbs	ASTM D 4632-91	250
Trapezoid Tear Strength	lbs	ASTM D 4533-91	100
Puncture Resistance	lbs	ASTM D 6241	700

Apparent Opening Size	Std Sieve	ASTM D 4751-95	100
Permittivity	Sec^{-1}	ASTM D 4491-99a	1.20
UV Resistance	% min. 500 hr	ASTM D 4355	70

C. Drainage Geocomposite:

1. Fabric wrapped high density polyethylene (HDPE) geonet drainage media.
2. Geonet: Two overlapping strands of HDPE.
3. Geotextile fabric heat-bonded to both sides of geonet and conform to the properties listed in Subsection 2.B.5.a. Fabric shall be bonded to the geonet so that it lies flat on the geonet, free of wrinkles and folds.
4. The geotextile shall be bonded to the geonet over 100 percent of the surface. Any material not meeting this requirement will be rejected for use on this project.
5. Geotextile extends a minimum of 2 inches beyond the edge of the geonet. The fabric edge shall be true and even to the edge of the geonet.
6. Drainage geocomposite conforming to the following properties:

Note A: Drainage geocomposite shall have a minimum calculated required transmissivity of 3.2 $x10^{-4}$ m^2/sec at a normal pressure of 10,000 psf and a gradient of 0.02.

The minimum calculated required transmissivity $_{(qreqd)}$ has been determined as follows:

$$q_{reqd} = q_{design}\ [FS_d x RF_{IN} x RF_{CR} x RF_{CC} x RF_{BC}]$$

Property	Unit	Test Method	Value
Specific Gravity (geonet)	gm/cm^3	ASTM D 1505-96	0.93 5
Melt Flow Index (geonet)	gm/10 min.	ASTM D 1238-95	0.30
Thickness (geonet)	mil	ASTM D 5199	200-330
Percent Carbon Black (geonet)	%	ASTM D 1603-94	2-3
Transmissivity $_{(qreqd)}$	m^2/sec	ASTM D 4716-87	See Note A.
Ply Adhesion	lbs/in	ASTM D 7005-03	1.0 min
Tensile Strength at Yield MD	lbs/in	ASTM D 7179	45

Where; q_{design} Minimum calculated design transmissivity for geocomposite shall be as given below:
q_{design} = 4.9 $x10^{-5}$$m^2$/sec @ 10,000 psf and 0.02gradient

FS_d Factor of Safety for drainage = 3
RFIN Intrusion Reduction Factor = 1.0[1]
RFCR Creep Reduction Factor = 1.1[1]
RFCC Chemical Clogging Reduction Factor = 1.4[1]
RFBC Biological Clogging Reduction Factor = 1.4[1]

Notes

1. These Reduction Factor (RF) values were assumed in this analysis, based on published data for a wide range of GCD materials. Project specific RF values can be applied during the material approval process

if supporting documentation is provided by the manufacturer and approved by the project engineer.

Each type of drainage geocomposite shall meet the following requirement:

$$q_{100} \geq q_{reqd}$$

Where;

q_{100} Flow rate of the geocomposite tested in accordance with ASTM D4716 for all manufacturers quality control and quality assurance conformance testing, with the following conditions:

- performed with boundary condition of Ottowa Sand and liner;
- test duration = 100 hours;
- performed at a normal pressures of 10,000 psf;
- tested at hydraulic gradient of 0.02; and
- oriented in the machine direction.

D. Interfacial Friction Angle:

1. Between drainage geocomposite and leachate collection sand.

2. Between drainage geocomposite and textured HDPE geomembrane.

3. Interfacial friction angle properties shall be in accordance with the test methods specified in Section 02780.

PART 3 - EXECUTION

3.01 PREPARATION OF BASE FOR GEOTEXTILE: Subgrade surfaces shall be properly graded and compacted as called for. All sharp or protruding objects shall be removed from the subgrade surface prior to fabric placement. Geotextile fabric shall not be placed until the foundation preparation is completed and the subgrade surfaces have been inspected and approved by the Engineer.

3.02 INSTALLATION:

A. Geotextile installation shall be in accordance with manufacturer's recommendations and shall include the following considerations:

1. Place in the manner and at the locations shown on the drawings.

2. At the time of installation, geotextile shall be rejected if it has defects, rips, holes, flaws, deterioration or damage incurred during manufacture, transportation or storage.

3. Geotextile used for the underdrain of the liner system shall be sewn together continuously to provide proper separation of soil materials.

B. Drainage Geocomposite Placement:

1. Place in the manner and at the locations shown on Drawings.

2. At the time of installation, drainage net shall be rejected if it has defects, rips, flaws, deterioration, or damage incurred during manufacture, transportation, or storage.

3. Join adjacent sheets of geonet with plastic ties spaced at minimum of 5 feet on-center. Adjoining sheet ends shall be heat bonded.

4. Place long dimensions perpendicular to the contours of the sideslopes (i.e. up and down). Lay smooth and free of tension, stress, folds, wrinkles, or creases.

5. Provide a minimum width of 4 in. of overlap for each joint of the geotextile.

6. Geotextile used in the Drainage Geocomposite shall be heat bonded at all overlays.

7. Place so that the upstream strip of geotextile will overlap the downstream strip.

C. Protection of Geotextile and Drainage Net:

1. Upon delivery to the site, geotextiles shall be set up off the ground and be wrapped with a plastic tarp or stored inside a trailer or building to prevent dust clogging.

2. Exercise necessary care while transporting, storing and installing the geotextile to prevent damaging it.

3. Protect from prolonged direct exposure to sunlight.

4. Repair all damaged areas of the geotextile by placing another piece of geotextile of sufficient size to extend a minimum of 1.0 foot beyond the limits of the damage in all directions over the damaged area.

5. Do not leave exposed more than seven (7) days without being covered by backfill.

6. Overlap successive pieces of geotextile a minimum of 1.0 foot.

7. When required, sew overlaps and repairs to damaged geotextile using a portable machine to provide seam strengths of at least 90 percent of the filter fabric strength.

END OF SECTION

SECTION 02275

GEOSYNTHETIC CLAY LINER

PART 1 - GENERAL

1.01 RELATED DOCUMENTS

A. Drawings and General Terms and Conditions as outlined in Section 1 of the Construction Agreement and Division-1 Specification sections, apply to work of this section.

B. The requirements set forth by the Quality Assurance/Quality Control Plan shall apply to the work specified in this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE:

A. Earthwork: Section 02200

B. Geotextiles and Drainage Geocomposite: Section 02272

C. Geomembrane Liner: Section 02771

D. Interfacial Friction Angle Conformance Testing: Section 02780

1.03 DESCRIPTION:

A. Furnish and install a geosynthetic clay liner (GCL) as part of the composite liner in the landfill. Sufficient liner material shall be furnished to cover all areas shown on the drawings including overlaps at field seams and anchor trenches. The GCL manufacturer shall provide the service of a technical representative as described in Section 3.02.A for a minimum of 1 working day.

B. All material shall be furnished from no more then 3 lots. A lot is defined as a group of consecutively numbered rolls of material from the same manufacturing line or batch.

1.04 SUBMITTALS:

A. A plan showing the proposed liner layout.

B. All required manufacturer's quality control certifications as described in GR1-GCL3 “standard specification for Test Methods, Required Properties, and Test Frequencies of Geosynthetic Clay Liners” and in para. 8.1 of Appendix B – Quality Assurance Quality Control Plan, including, but not limited to, the following:

- verification that clay component of the finished product is 70 to 90 percent sodium montmorillonite clay of bentonite deposits;
- verification that the proper mass per unit area of bentonite clay has been added to the finished product;

- verification that the actual geotextiles used in the finished product meet the manufacturer's specification based on the minimum average roll value (MARV) concept;

- verification that needle-punched non-woven geotextiles have been inspected continuously for the presence of broken needles using an in-line metal detector.

- manufacturer supplying the GCL shall demonstrate previous experience by letter of certification. Certification shall indicate that the manufacture has produced, and has in service in similar applications for a period of not less than one (1) year, at least five (5) million sq.ft. of GCL material, meeting these Specifications.

- manufacturer's Quality Control Certification for the tests described in Table 2.01A.

1.05 CONFORMANCE TESTING:

The GCL will be sampled by the Geosynthetic CQA or his agent(s), at the manufacturer or upon delivery to the site. The Geosynthetic CQA shall assure that conformance test samples are obtained for the GCL at a rate of 1 test per 50,000 square feet of GCL with a minimum of once per lot, for testing, to assure conformance to the specification. On site samples should be taken from selected rolls by removing the protective wrapping and cutting a full width 3-foot long sample from the outer wrap of the selected rolls. The rolls should be immediately rewrapped and replaced in the shipping trailers or in the temporary field storage area. Test samples should be identified by type, style, lot, and roll numbers. The machine direction should also be noted on the samples with a waterproof marker. A lot consists of a group of material which is manufactured from a specific batch of raw materials (e.g., HDPE resin, or bentonite clay). The manufacturer shall identify the consecutively numbered rolls of material that are inclusive within a lot. It is not required that all rolls included in a lot be supplied to the project, as long as the specified certification test results are supplied by the manufacture to bracket the rolls delivered to the project. The interfacial and internal friction angle tests shall be performed at the frequency defined in Section 02780.

The contractor or sub-contractor shall coordinate information on manufacture and delivery of the GCL with the Geosynthetic CQA to assure that sampling and testing occur in a timely manner as to avoid construction delays.

Any further testing required to assure conformance shall be the responsibility of the contractor in accordance with 1.05 (C) of Section 01025. The testing laboratory and the results shall be acceptable to the Engineer.

GCL Conformance testing shall include the following:

A. Bentonite mass per unit area per ASTM D 5993;

B. Free swell of clay component per ASTM D 5890

C. Hydraulic conductivity per (ASTM D 5887-99).

D. Grab Strength and Peel Strength (ASTM D 6768-02 and 6496-99, respectively)

E. Interfacial Friction Angle (ASTM D 6243): Testing for interfacial friction angle properties shall be done on the following interfaces:

1. Between HDPE textured geomembrane and non-woven side of GCL.

2. GCL internal.

3. Between woven side of GCL and compacted clay.

The interfacial friction angle tests shall be performed according to Section 02780 and at a frequency as indicated in Section 02780.2.01(D). Test results must meet the minimum requirements stated in Section 02780.3.01.

PART 2 - PRODUCTS

2.01 MATERIALS:

A. The GCL liner shall be Bentomat ST as produced by CETCO and meet the following properties.

Material Property	Test Method	Required Values	Manufacturer's Test Frequency
Bentonite Swell Index	ASTM D 5890	24 mL/2g min.	1 per 50 tons
Bentonite Fluid Loss	ASTM D 5891	18 mL max.	1 per 50 tons
Bentonite Mass/Area^2	ASTM D 5993	0.75 lb/ft^2 (3.6 kg/m^2)	1 per 5,000 yd^2
GCL Grab Strength	ASTM D 6768-02	22.5 lbs/in.	1 per 30,000 yd^2
GCL Peel Strength	ASTM D 6496-99	2.5 lbs/in.	1 per 5,000 yd^2
GCL Index Flux	ASTM 5887-99	1x10^{-8} m^3/m^2/sec.	1 per 30,000 yd^2
GCL Hydraulic Conductivity	ASTM 5887-99	5x10^{-9} cm/sec.	1 per 30,000 yd^2
GCL Hydrated Internal Shear Strength	ASTM D 6243	See Section 02780 (3.01)	
GCL Non-Woven Side vs HDPE Textured Geomembrane Shear Strength	ASTM D 6243	See Section 02780 (3.01)	
GCL Woven Side vs Compacted Clay Shear Strength	ASTM D 6243	See Section 02780 (3.01)	

B. The geotextiles used in the manufacture of the GCL shall be made up from polypropylene fibers. Any and all substitutions shall be approved by the Project Manager prior to their use.

C. The liner shall be manufactured by the mechanical bonding of the needlepunch process to enhance the friction characteristics of the liner and to maintain the integrity of the liner under hydration. No glues or adhesives shall be used in lieu of the needlepunch process so as to retain these characteristics.

Needled GCLs are those which, by the process of a needling board (similar to that used in the manufacture of standard nonwoven geotextiles) have fibers of a nonwoven geotextile pushed through the bentonite clay core and integrated into the woven geotextile.

D. The GCL shall have a hydraulic conductivity equal to or less than 5×10^{-9} cm/sec at 5 psi confining stress.

PART 3 - EXECUTION

3.01 SHIPPING AND HANDLING:

A. Covering of the Rolls:

1. Manufacturers should clearly stipulate the type of protective covering and the manner of cover placement. The covering should be verified as to its capability for safe storage and proper transportation of the product.

2. The covering should be placed around the GCL in a workmanlike manner so as to effectively protect the product on all of its exposed surfaces and edges.

3. The central core should be accessible for handling by forklift vehicles. Handling of the GCL during shipment (i.e. loading, transport, and unloading) shall be performed using techniques approved by the manufacturer.

4. Clearly visible labels should identify the name and address of the manufacturer, trademark, date of manufacture, location of manufacture, style, roll number, lot number, serial number, dimensions, and weight in accordance with ASTM D 4873.

B. Storage at the Manufacturing Facility:

1. GCLs should always be stored indoors until they are ready to be transported to the field site.

2. Handling of the GCL should be such that the protective wrapping is not damaged. If it is, it must be immediately rewrapped by machine or hand; in the case of minor tears it may be taped.

3. Placement and stacking of rolls should be done in a manner so as to prevent thinning of the product at points of contact with the storage frame or with one another.

C. Shipment:

1. The GCLs should be shipped by themselves with no other cargo which could damage them in transit during stops or while off-loading other materials.

2. Method of loading GCLs rolls, transporting them, and off-loading them at the job site should not cause any damage to the GCL, its core or its protective wrapping.

3. Any protective wrapping that is damaged or stripped off the rolls should be repaired immediately or the roll should be moved to an enclosed facility until its repair can be made to the approval of the quality assurance personnel.

4. If any clay has been lost during transportation or from damage of any type, the outer layers of the GCL should be discarded until undamaged product is evidenced. The remaining roll must be wrapped in accordance with the manufacturer's original method to prevent hydration or further damage to the remaining roll.

D. Storage at the Site:

1. Handling of the GCLs should be done in a competent manner such that damage does not occur to the product nor to its protective wrapping.

2. The location of temporary field storage should not be in areas where water can accumulate. The rolls should be stored on high, flat ground or elevated off the ground so as not to form a dam creating the ponding of water. It is recommended to construct a platform so that GCL rolls are continuously supported along their length. GCL stored outdoors should be covered completely by tarps or other protective materials.

3. The rolls should not be stacked so high as to cause thinning of the product at points of contact (i.e. no higher than 3 rolls high). Furthermore, they should be stacked in such a way that access for conformance testing is possible.

4. If outdoor storage of rolls is to be longer than a few weeks, particular care (e.g. using tarpins) should be taken to minimize moisture pick-up or accidental damage. For storage periods longer than one season, a temporary enclosure should be placed over the rolls or they should be moved within an enclosed facility.

3.02 INSTALLATION:

A. The manufacturer shall provide the service of a technical representative during start-up. Start-up service provided by the manufacturer shall include transportation, lodging, expenses, materials, and equipment. Start-up service shall be provided by the manufacturer for a minimum of 1 working day.

B. General: The Contractor shall schedule a pre-installation conference as specified in Section 01041, Part 1.02. Prior to ordering GCL material, the Contractor shall submit, for the Project Manager's approval, a description or drawing of the method of sheet layout, detailing the orientation of sheets and the direction of the overlap between sheets. During installation of the GCL, the Installer shall label each sheet immediately after deployment, with the roll number, panel number, and date it was installed.

Installation of the GCL shall include the following considerations:

1. The GCL shall be installed with the woven-coated side contacting the compacted clay and the non-woven side contacting the HDPE textured geomembrane. Place in the manner and at the locations shown on the drawings.

2. Rolls shall be handled utilizing a 3 inch schedule 80 steel pipe through the core and slings or straps attached to the ends of the pipe (core pipe). The core pipe shall be suspended from a spreader bar so that the edges of the liner are not damaged by the suspending straps or chains.

3. Work on the slopes shall be undertaken before the bottom to permit drainage in the event of rainfall. Panels may be pulled up from the bottom of the slope to the anchor trench or anchored first and the roll slowly lowered down the slope. Seams shall be perpendicular to the toe of the slope at all times. Seams at the base of the slope shall be a minimum of 5 feet away from the toe of the slope. Roll end seams or joints will not be allowed on the sideslopes.

4. Seam areas or runs shall be flat and clear of any large rocks, debris or ruts. Contacting surfaces shall be clean and clear of dirt or native soil with all edges pulled tight to maximize contact and to smooth out any wrinkles or creases. Overlaps shall be a minimum of 6 inches and verified by QA/QC personnel. A proper seam shall cover the lap line and leave the match line exposed.

5. Seams shall be augmented with granular bentonite to insure seam integrity. Granular bentonite shall be spread evenly from the panel edge to the lap line at a minimum rate of 1/4 pound per lineal foot. This rate of application will be assured by using one 50-pound bag of granular bentonite (evenly spaced along the seam) per roll of GCL. Accessory bentonite shall be of the same type as the material within the composite liner itself. Fasteners, anchor pins or adhesives may be used on seams to keep panels in place during backfill operations if necessary. All butt seams shall have a minimum of 2 foot overlap.

6. The contractor shall only work on an area that can be completed in one working day. Completion shall be defined as the full installation of the liner and placement of the textured HDPE geomembrane to cover the GCL. Prior to deployment of the GCL, the subgrade (i.e. clay) will be smooth rolled to provide a smooth surface free of debris, roots and angular rocks. The GCL subgrade will be inspected and certified by the CQA personnel prior to placement of the GCL. Deployment of the GCL will be visually inspected to assure that no potentially harmful objects are present (e.g. stones, cutting blades, small tools, sandbags, etc.).

7. For any penetrations or structures the liner will contact, a small notch shall be cut along the edge of the area. The liner shall be brought up to the appurtenance and trimmed to fit snugly. The contractor shall hand apply and compact a pure bead or dry mixture of 1 part bentonite to 4 parts soil (by volume), blended dry, into half of the notch. The liner shall then be inserted into the notch, with the remaining area in the notch refilled with the 1 to 4 mixture and compacted.

8. For protection and proper performance, the primary geomembrane liner (80-mil HDPE textured liner) shall be applied immediately over the liner. During the liner application, a care shall be taken to avoid sharp turns and any quick stops or starts so as to avoid pinching or moving the liner.

9. Large rips, or tears shall be repaired by placing a patch over the defect, with a minimum overlap of 12 inches on all edges. Accessory bentonite shall be placed between the patch and the repaired material at a rate of 1/4 pound per lineal foot of edge. Prematurely hydrated GCL shall be removed and replaced with new GCL in accordance with the repair procedures described above.

10. Overlap joints and seams shall be measured as a single layer of geotextile.

11. Securely anchor GCL as shown on the drawings.

12. Soil cover shall be placed over the GCL using construction equipment that minimizes stresses on the GCL. A minimum thickness of 1 foot of cover should be maintained between the equipment tires/tracks and the GCL at all times during the covering process. This thickness recommendation does not apply to frequent traffic areas or roadways for which a minimum thickness of 2 feet is required.

13. During deployment of the GCL the material shall not be dragged over the subbase

C. Protection of Bentonite Geocomposites:

1. Exercise necessary care while transporting and installing the geocomposite to prevent damaging it.

2. Stored rolls shall be on a flat dry surface and to avoid any unnecessary stress on the packaging.

END OF SECTION

SECTION 02771

GEOMEMBRANE LINER HIGH DENSITY POLYETHYLENE (HDPE)

PART 1 - GENERAL

1.01 RELATED DOCUMENTS:

A. Drawings and General Terms and Conditions as outlined in Section 1 of the Construction Agreement and Division-1 Specification sections, apply to work of this section.

B. The requirements set forth by the Quality Assurance/Quality Control Plan shall apply to the work specified in this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE:

A. Project Coordination: Section 01041

B. Earthwork: Section 02200

C. Geotextiles and Drainage Geocomposite: Section 02272

D. Geosynthetic Clay Liner: Section 02275

E. Interfacial Friction Angle Conformance Testing: Section 02780

1.03 DESCRIPTION OF WORK: Extent of flexible membrane lining work is shown on the contract drawings. 80 mil textured HDPE geomembrane shall be installed as part of the primary liner system and 60 mil textured HDPE geomembrane shall be installed in the secondary liner system.

1.04 SUBMITTALS:

A. Submit 2 copies plus the number of copies required by the contractor of quality control documentation described in 1.05 herein and para. 5.1 of Appendix B – Quality Assurance Quality Control Manual.

B. Prior to ordering HDPE material, the Contractor shall submit, for the CQA Project Manager's approval, a description or drawing of the method of sheet layout, detailing the orientation of sheets and the direction of the overlap between sheets.

1.05 QUALITY CONTROL DOCUMENTATION: (Furnished by Liner Manufacturer and Liner Installer)

A. Manufacturer's Experience: The manufacturer supplying the membrane shall be GRI certified and shall satisfactorily demonstrate previous experience by letter of certification. Certification shall indicate that the manufacturer has produced, and has in service in similar applications for a period of not less than one (1) year, at least ten (10) million sq ft of HDPE material meeting these Specifications.

B. Installer's Experience: The Installer proposing to install the lining shall satisfactorily demonstrate previous experience by letter of certification. Certification shall indicate the Installer's successful past installation of at least 10,000,000 sq ft of HDPE membrane lining.

Installation shall be performed under the direction of a single installation supervisor who shall remain on site and be in responsible charge throughout the liner installation, including subgrade acceptance, liner layout, seaming, testing and repairs, and all other activities contracted for with the Installer. The installation supervisor shall have supervised the installation of at least 10,000,000 sf of polyethylene geomembrane. Actual seaming shall be performed under the direction of a master seamer who may be the same person as the installation supervisor, and who has a minimum of 1,000,000 sf polyethylene geomembrane seaming experience using the same type of seaming apparatus as that specified in this project. **No seaming may be done by any individual with less than 500,000 sf of polyethylene geomembrane seaming experience.** The installation supervisor or master seamer must be on site whenever seaming is being performed.

C. QUALITY CONTROL DURING MANUFACTURE: Random sampling shall be performed by the Manufacturer throughout the production run at the frequencies indicated below for each of the listed properties for the geomembrane to be delivered to the site.

	Test	Frequency
Thickness	ASTM D 5994	1 per roll
Asperity	ASTM D 7466	Per GM 13
Oxidative induction time	ASTM D 3895 ASTM D 5885	1 per 200,000 lbs
UV resistance	GM11 ASTM D 5885	1 per formulation
Oven Aging at 85°C	ASTM D 5721 ASTM D 3895 ASTM D 5885	1 per formulation
Carbon black content	ASTM D 1603-95	1 per 20,000 lbs
Carbon black dispersion	ASTM D 5596	1 per 45,000 lbs
Force per unit width at yield	ASTM D 6693-01	1 per 20,000 lbs
Force per unit width at break	ASTM D 6693-01	1 per 20,000 lbs
Elongation at yield	ASTM D 6693-01	1 per 20,000 lbs
Elongation at break	ASTM D 6693-01	1 per 20,000 lbs
Tear resistance	ASTM D 1004-94a	1 per 45,000 lbs
Puncture resistance	ASTM D 4833	1 per 45,000 lbs
Density	ASTM D 792-91	1 per 200,000 lbs
stress crack resistance	ASTM D 5397	per GRI GM 10

Value must meet the requirements specified in Table 1 of Part 2.

The Resin Manufacturer shall provide certification to the Geomembrane Sheet Manufacturer that the resin meets or exceeds the specifications for Density, Environmental Stress Crack and Melt Index.

Sheet thickness shall be monitored continuously during manufacture and shall be nominal thickness ±10% across the sheet.

Rolls not satisfying the specifications shall be rejected. The Manufacturer shall provide certification of testing as described in Part 2.

1.06 CONFORMANCE TESTING:

The geomembrane will be sampled by the Geosynthetic CQA or his agent(s), at the manufacturer or upon delivery to the site. The Geosynthetic CQA shall assure that conformance test samples are obtained for the geomembrane at a rate of 1 test per 50,000 square feet of geomembrane with a minimum of once per lot, for testing, to assure conformance to the specification. A lot consists of a group of material which is manufactured from a specific batch of raw materials (e.g., HDPE resin, or bentonite clay). The manufacturer shall identify the consecutively numbered rolls of material, that are inclusive within a lot. It is not required that all rolls included in a lot be supplied to the project, as long as the specified certification test results are supplied by the manufacture to bracket the rolls delivered to the project. The interfacial and internal friction angle tests shall be performed at the frequency defined in Section 02780.

The contractor or sub-contractor shall coordinate information on manufacture and delivery of the geomembrane with the Geosynthetic CQA to assure that sampling and testing occur in a timely manner as to avoid construction delays.

Any further testing required to assure conformance shall be the responsibility of the contractor in accordance with 1.05 (C) of Section 01025. The testing laboratory and the results shall be acceptable to the Engineer.

Geomembrane Conformance testing shall include the following:

Thickness	ASTM D 5994
Asperity	ASTM D 7466
Carbon black content	ASTM D 1603-94
Carbon black dispersion	ASTM D 5596
Force per unit width at yield	ASTM D 6693-01
Force per unit width at break	ASTM D 6693-01
Elongation at yield	ASTM D 6693-01
Elongation at break	ASTM D 6693-01
Tear resistance	ASTM D 1004 - 94a
Puncture resistance	ASTM D 4833
Interface friction angle between drainage geocomposite and geomembrane	ASTM D 5321-92
Interface friction angle between Geosynthetic clay liner and the Geomembrane	ASTM D 6243-98

These conformance tests shall be performed in accordance with the test method specified. The Contractor shall supply the materials required to perform the conformance testing.

1.07 SPECIAL PRODUCT WARRANTY: (Furnished by Liner Installer)

A. Manufacturer's Guarantee: The manufacturer of the membrane liner shall enter into agreement with the Owner guaranteeing the membrane as follows:

The manufacturer warrants the HDPE liner which is manufactured, sold as first quality, and installed with technical assistance and/or by an approved installation contractor to be (1) furnished free of manufacturing defects in workmanship or material for a period of one year from the time of delivery with the basis for judgment of defects being the applicable product specifications in effect at the time the order was placed unless modified by mutual written agreement; (2) shall withstand normal weathering due to the effects of normal service for a period of forty (40) years from the date of delivery. "Normal service" does not include physical damage caused by acts of God, casualty, or catastrophe such as (but not limited to) earthquakes, fire, explosion, floods, lightning, piercing hail, tornadoes, corrosive air pollution, mechanical abuse by machinery, equipment, people or animals, or excessive flexures, pressures or stress from any source other than faulty installation, and (3) immune to chemical attack and degradation by chemicals, specified in the manufacturer's chemical resistance literature, as compatible with, and as not having an adverse effect on the membrane; and (4) resistance to specific chemicals when tested for them by the manufacturer at the request of the Owner.

Should defects or weathering degradation within the scope of the above warranty occur, the manufacturer shall refund to the purchaser-user the pro-rata part for the unexpired term of the warranty of the purchaser-user's original cost of such product, or will supply repair or replacement materials at the then-current price. In the event the manufacturer supplies repair or replacement materials, against the then- current price, the manufacturer will credit the lesser of (1) the pro-rata part of the original sales price of the material so repaired or replaced for the unelapsed period of the warranty, or (2) the pro-rata part of the then-current price of the material so repaired or replaced to the unelapsed period of the warranty. The warranty shall continue in effect on the repaired or replaced material for the unelapsed term of the original warranty. To enable the manufacturer's technical staff to properly determine the cause of any alleged defect and to take appropriate steps to effect timely corrective measures if such defect is within the warranty, any claim for alleged breach of warranty will be made and presented in writing to manufacturer and the installing Contractor within thirty (30) days after the alleged defect was first noticed.

PART 2 - PRODUCTS (Furnished by Installer)

2.01 TEXTURED HIGH DENSITY POLYETHYLENE (HDPE) MEMBRANE:

A. General: The materials supplied under these Specifications shall be first quality products designed and manufactured specifically for the purposes of this work, and which have been satisfactorily demonstrated by prior use to be suitable and durable for such purposes. The materials supplied shall not be manufactured from more than three different lots (resin batches), unless the manufacturer agrees to perform conformance testing on the additional lots. Conformance testing shall be performed in accordance to Part 4.

B. Description of 80-mil HDPE Material: The membrane shall be a high density polyethylene (HDPE) of 80-mils thickness containing no additives, fillers or extenders. The lining material shall be manufactured a minimum of 15 feet seamless widths, and have the following physical characteristics:

TABLE 1

Test	Test Designation	Requirement
Sheet thickness, textured	ASTM D 5994	±10% for individual for 8 out of 10 values ±15% for individual for any of the 10 values
Asperity height	ASTM D 7466	16 mil (min. average)
Tensile strength yield	ASTM D-6693-01	min. 168 lb per in. width
Tensile strength at break	ASTM D-6693-01	min. 120 lb per in. width
Elongation at yield	ASTM D-6693-01	min. 12%
Elongation at break	ASTM D-6693-01	min. 100 %
Tear resistance	ASTM D-1004-94A	min. 56 lb
Puncture resistance	ASTM D 4833	min. 120 lb
Oxidative induction time		
(standard)	ASTM D 3895	100 minutes
(high pressure)	ASTM D 5885	400 minutes
Oven aging at 85°C		
(standard OIT)	ASTM D 3895	55% retained at 90 days
(high pressure OIT)	ASTM D 5885	80% retained at 90 days
Ultra Violet Resistance	GM 11, ASTM D-5885	min. 50% retained @ 1600 hrs.
stress crack resistance	ASTM D-5397	500 hrs
Carbon black content	ASTM D-1603-95	2 to 3%
Carbon black dispersion	ASTM D-5596 and GRI GM 13	9 in category 1 or 2 1 in category 3

Test	Test Designation	Requirement
Interfacial Friction Angle		
Between drainage geocomposite and geomembrane	ASTM D 5321-92 (see Section 02780)	See Section 02780 3.01
Between geosynthetic clay liner (non-woven side) and geomembrane	ASTM D 6243-98 (See Section 02780)	See Section 02780 3.01

In addition, the 80-mil geomembrane shall be produced as to be free of holes, blisters, undispersed raw materials, or any sign of contamination by foreign matter, and shall not have striations, roughness, pinholes or bubbles on the surface.

C. Description of 60-mil HDPE Material: The membrane shall be a high density polyethylene (HDPE) of 60-mils thickness containing no additives, fillers or extenders. The lining material shall be manufactured a minimum of 15 feet seamless widths, and have the following physical characteristics:

TABLE 2

Test	Test Designation	Requirement
Sheet thickness, textured	ASTM D 5994	±10% for individual for 8 out of 10 values ±15% for individual for any of the 10 values
Asperity height	ASTM D 7466	16 mil (min. average)
Tensile strength yield	ASTM D-6693-01	min. 126 lb per in. width
Tensile strength at break	ASTM D-6693-01	min. 90 lb per in. width
Elongation at yield	ASTM D-6693-01	min. 12%
Elongation at break	ASTM D-6693-01	min. 100 %
Tear resistance	ASTM D-1004-94A	min. 42 lb
Puncture resistance	ASTM D 4833	min. 90 lb
Oxidative induction time		
(standard)	ASTM D 3895	100 minutes
(high pressure)	ASTM D 5885	400 minutes
Oven aging at 85°C		
(standard OIT)	ASTM D 3895	55% retained at 90 days
(high pressure OIT)	ASTM D 5885	80% retained at 90 days
Ultra Violet Resistance	GM 11, ASTM D-5885	min. 50% retained @ 1600 hrs.

Test	Test Designation	Requirement
stress crack resistance	ASTM D-5397	500 hrs
Carbon black content	ASTM D-1603-95	2 to 3%
Carbon black dispersion	ASTM D-5596 and GRI GM 13	9 in category 1 or 2 1 in category 3

Test	Test Designation	Requirement
Interfacial Friction Angle		
Between drainage geocomposite and geomembrane	ASTM D 5321-92 (see Section 02780)	See Section 02780 3.01
Between clay borrow and geomembrane		(See Section 02780)

In addition, the 60-mil geomembrane shall be produced as to be free of holes, blisters, undispersed raw materials, or any sign of contamination by foreign matter, and shall not have striations, roughness, pinholes or bubbles on the surface.

D. Factory Bonded Seam: Calendered HDPE sheeting may not be fabricated into large sections at the factory.

E. Extrusion Joining Resin: Resin for extrusion joining sheets shall be HDPE produced from the same material as the sheet resin. Physical properties shall be the same as those of the resin used in the manufacture of the HDPE liner. The resin shall be supplied in black and/or natural color. Natural resin shall be colored black through addition of 2.0 to 3.0 percent master batch colorant before use.

F. Documentation: Prior to delivery of the geomembrane to the job site, the Installer shall be required to provide the Owner with a written certification that the product to be delivered was extruded from the specified resin. The manufacturer shall provide clear and concise quality control certificates for each batch of resin and each shift's production of geomembrane, and shall follow the quality control testing program as described in Part 4. These quality control certificates shall be signed by responsible parties employed by the Manufacturer, and shall be supplied to the Owner. No geomembrane will be permitted to be delivered until the Owner has in his possession such certification. The manufacturer shall permit conformance test sampling by an independent party to be performed at the manufacturing plant prior to shipment if requested.

G. Roll Identification: Each roll shall have permanently affixed both inside and outside the roll the following information: name of manufacturer; date of manufacture; resin batch code; thickness of the material; roll number; roll length; and roll width. Unlabeled rolls will not be used and shall be returned to the manufacturer at the Contractor’s expense.

2.02 MISCELLANEOUS MATERIALS: (Furnished by Installer)

A. Pipe Boots, Vents, and Patches: All such devices shall be of the same material as the lining or a compatible approved equal.

B. Mechanical Fastenings: Mechanical fastenings shall be of the material, size, and type as detailed on the plans or approved shop drawings.

PART 3 - EXECUTION

3.01 GENERAL: The Contractor shall schedule a pre-installation conference as specified in Section 01041, Part 1.02. During installation of the geomembrane, the Installer shall label each sheet immediately after deployment, with the roll number, panel number, and date it was installed.

3.02 SHIPPING AND HANDLING: Each roll shall be individually packaged in heavy cardboard or wooden crate fully enclosed and protected to prevent damage to it during shipment, prominently identified in the same fashion as the sheet within and showing the date of shipment. Until installed, the rolls shall be stored indoors in their original unopened crates; if outdoors, they shall be stored on pallet and shall be protected from the direct rays of the sun under a light-colored heat-reflective opaque cover in a manner that provides a free-flowing air space between the crate and cover.

3.03 SURFACE PREPARATION:

A. Conditions: Surfaces to be lined shall be smooth and free of all angular rocks, stones greater than ½-inch, sticks, roots, sharp objects or debris of any kind. The surface shall provide a firm, unyielding foundation. No standing water or excessive moisture shall be allowed.

B. Acceptance: The Geosynthetic CQA and the Installer shall certify in writing to the Resident Project Representative that the surface to be lined is acceptable. Submittal of written acceptance may proceed incrementally according to installation schedule. No geomembrane shall be placed on subgrade deemed unsuitable by the Installer/or Engineer or the Geosynthetic CQA.

3.04 ANCHOR TRENCH: Excavation, backfill and compaction of the anchor trench will be the responsibility of the General Contractor. The anchor trench shall be excavated along the lines shown on the design drawings. The length of open trench should not exceed the amount of liner to be placed in a two (2) day period unless approval has been provided by Owner or Owner's testing agency.

The anchor trench may be partially backfilled during geomembrane panel placement, however the anchor trench shall <u>not</u> be compacted until the geomembrane has experienced sufficient expansion/contraction cycles. Compaction of the anchor trench backfill shall be performed using manually operated compaction equipment. Backfill shall be placed in lifts not greater than 12 inches in loose thickness and shall be compacted to at least 90% of the maximum dry density according to ASTM D 698. This backfill shall be compacted 1” minus impervious borrow. Borrow shall be placed at, or 4 percent above optimum. Owner's testing agency shall approve backfill material prior to anchor trench placement, and must be notified prior to compaction.

3.05 FIELD SEAMS: The Geosynthetic QARE can, at his sole discretion, not allow any individual seamer or seaming equipment to be used for the project, based on observations made in the field. The Geosynthetic QARE will notify the Contractor of the individual or seaming equipment which may not be used on the project and the reason or steps necessary to demonstrate the person or equipment acceptance on the job. The Contractor shall have no recourse for this decision against the Owner, Engineer, or other parties.

A. No horizontal seams shall be allowed on the sideslopes of the cell.

B. Layout: Overlap panels in shingle style from high to low elevation. Minimum panel length shall be 70 feet.

C. Preparation: All areas which are to become seam interfaces shall be cleaned of dust and dirt. When extrusion joining is required, the slick surfaces of the HDPE sheet which are to become seam interfaces shall be prepared by sanding or grinding (perpendicular to the seam) to a depth of less than .005 in. before joining the sheets. Field joints shall not take place unless the sheet is dry.

D. Seaming Methods: Installer shall submit to the Owner prior to construction a list of the seaming equipment and testing equipment, including manufacturer and model number to be used on-site. Field seams shall be made by overlapping adjacent sheets the appropriate amount and using one of the following seaming techniques:

<u>Hot Air/Hot Wedge</u>: Hot air/hot wedge technique shall be made by either a nozzle which directs hot air between the sheets or a hot metal surface in contact between the sheets. Each seaming unit must include a thermometer giving the temperature of the machine at the nozzle or metal surface. The seaming unit shall maintain a recordable temperature determined by on-site conditions and shall not vary more than, 50 deg.F above, or below the recommended seaming temperature. The adjacent geomembrane sheets shall be overlapped 6 inches. The overlapped sheets are then pressed together by mechanical means. Seaming equipment that makes a split hot wedge seam will be the preferred method of seaming; single hot wedge seaming will be allowed only with the approval of the Owner.

<u>Extrusion Bonding</u>: Extrusion and fusion bonding will be limited to areas where hot wedge cannot be used, such as pipe boots, and to any necessary repairs. The use of extrusion and fusion bonding as the primary seaming method will be allowed only with the approval of the Owner. The adjacent sheets to be seamed shall be overlapped a minimum of 3 inches. The joining procedure shall consist of softening the liner material by heated air. The temperature of the air impinging on the sheet for this purpose, shall range from, 420 deg.F to 680 deg.F. The exact temperature used shall be determined by the installation supervisor. Directly following the application of heat, a one and one-half inch minimum width strip of the same high density polyethylene resin from which the sheet is made shall be extruded between the overlapped sheets. The temperature of the resin as it emerges from the extrusion die shall range from, 428 deg.F to 536 deg.F. The overlapped sheets shall be firmly pressed together by mechanical means to form the extrusion joint.

<u>Fusion Bonding</u>: Extrusion and fusion bonding will be limited to areas where hot wedge cannot be used, such as pipe boots, and to any necessary repairs. The major seaming of the liner will be done with hot wedge. Fusion bonding shall be by means of a homogeneous overlap extrusion fusion process which provides continuous dynamic

integration of the extrudate bead with the lining material. The composition of the extrudate shall be identical to the lining material. The seaming unit shall be capable of continuously monitoring and controlling the temperature of the extrudate and the zone of contact where the machine is actually fusing the lining material. Temperature of the extrudate shall range from, 428 deg.F to 536 deg.F.

E. Tie-in Seams and Cross-Seams (Butt Seam): Seaming of geomembrane at new/new geomembrane that has become soiled and new/old geomembrane connections shall be properly prepared prior to seaming. All tie-in seams shall be pressure washed steam-cleaned and/or scrubbed to remove dirt or other deposits on the geomembrane. No seaming will take place on dirty or soiled geomembrane. The CQA shall inspect the tie-in preparation prior to seaming. The installer shall trial weld representative samples taken from the seaming area. Passing trial welds shall be attained in accordance with Section 4.03(B).

F. Seaming Wrinkles: Fishmouths or wrinkles at the seam overlaps shall be cut along the ridge of the wrinkle, back into the panel so as to affect a flat overlap. The cut fishmouths or wrinkles shall be seamed as well as possible, and shall then be patched with an oval or round patch extending a minimum of 6 inches beyond the cut in all directions.

G. Repairs: Any required repair of scratches >5% of the sheet thickness and small holes in the liner surface shall be made with the extrusion hand welder. Liner material shall be cleaned of all dirt, dust and other foreign material, all smooth HDPE surfaces roughened, air heated to the prescribed temperature, and a strip of HDPE resin extruded over the hole to produce an extruded welded repair.

H. Quality of Workmanship: All joints, on completion of the work, shall be tightly bonded. Any lining surface showing injury due to crimping, scuffing, penetration by foreign objects, scratching by welding equipment, or distress from rough subgrade shall, as directed by the Engineer, be replaced or covered and sealed with an additional layer of HDPE of the proper size. The Installer shall inspect the final installation and any defects shall be <u>repaired and tested</u> until satisfactory.

I. No seaming shall be allowed if the Geosynthetic CQA is not on-site.

3.06 PIPE BOOTS, VENTS, MECHANICAL FASTENINGS, and PATCHES: The geomembrane shall be installed around any pipes, concrete structures or other penetrations through the geomembrane in accordance with the detailed Specifications shown on the Drawings. Prior to the start of construction, the Installer may provide, for the approval of the Owner, alternate installation methods or details to successfully perform geomembrane termination.

All clamps, bolts, nuts, gaskets or other materials used to secure the geomembrane shall be compatible with and have a lifespan at least equal to that of the geomembrane.

Care shall be taken to protect the underside of the geomembrane from damage due to settling at any underbedding to concrete transition.

Extreme care shall be taken while welding around any penetration or similar structure since destructive testing is not likely to be possible in such areas. All seaming in these areas shall be performed by the Installer's Master Seamer and the operations shall be observed on a full time basis by the Geosynthetic CQA. Non destructive electric spark test of both the skirt and sleeve of the pipe boot shall be performed as described in Section 4.03(B) of this specification.

3.07 SEAMING WEATHER CONDITIONS:

A. Normal Weather Conditions: The normal required weather conditions for seaming are as follows:

1. Ambient temperature between 32°F (0°C) and 104°F (40°C).

2. Dry conditions, i.e. no precipitation or other excessive moisture, such as fog or dew.

3. No excessive winds.

The Geosynthetic CQA shall verify that these weather conditions are fulfilled and notify the Project Manager in writing if they are not. Ambient temperature shall be measured by the Geosynthetic CQA in the area in which the panels are to be placed. The Project Manager will then decide if the installation is to be stopped or special procedures used.

B. Cold Weather Conditions: To assure a quality installation, if seaming is conducted when the ambient temperature is below 32°F (0°C), the following conditions must be met:

1. Geomembrane surface temperatures shall be determined by the Geosynthetic CQA at intervals of at least once per 100 foot of seam length to determine if preheating is required. For extrusion welding, preheating is required if the surface temperature of the geomembrane is below 32°F (0°C).

2. Preheating may be waived by the Project Manager based on a recommendation from the Geosynthetic CQA, if the Installer demonstrates to the Geosynthetic CQA's satisfaction that welds of equivalent quality may be obtained without preheating at the expected temperature of installation.

3. If preheating is required, the Geosynthetic CQA shall inspect all areas of geomembrane that have been preheated by a hot air device prior to seaming, to assure that they have not been overheated.

4. Care shall be taken to confirm that the surface temperatures are not lowered below the minimum surface temperatures specified for welding due to winds or other adverse conditions. It may be necessary to provide wind protection for the seam area.

5. All preheating devices shall be approved prior to use by the Project Manager.

6. Additional destructive tests (as described in Section 5.5.6.2 of the Quality Assurance Plan (QAP)) shall be taken at an interval between 500 feet and 250 feet of seam length, at the direction of the Geosynthetic CQA.

7. Sheet grinding may be performed before preheating, if applicable.

8. Trial seaming, as described in Section 5.5.4 of the QAP, shall be conducted under the same ambient temperature and preheating conditions as the actual seams. Under cold weather conditions, new trial seams shall be conducted if the

ambient temperature drops by more than 5°F from the initial trial seam test conditions.

9. All snow and ice shall be removed from the liner using plastic shovels. The CQA will also have authority to suspend installation activities during severe weather conditions.

C. Warm Weather Conditions: At ambient temperatures above 104°F, no seaming of the geomembrane shall be permitted unless the Installer can demonstrate to the satisfaction of the Project Manager that geomembrane seam quality is not compromised.

Trial seaming, as described in Section 5.5.4 of the QAP, shall be conducted under the same ambient temperature conditions as the actual seams.

At the option of the Geosynthetic CQA, additional destructive tests (as described in Section 5.7 of the QAP) may be required for any suspect areas.

3.08 QUALITY CONTROL DURING INSTALLATION:

A. Site Test Equipment: The Installer shall maintain on site, in good working order, the following items:

Field Tensiometer: The tensiometer shall be motor driven and have jaws capable of traveling at a measured rate of 2 in./min. The tensiometer shall be equipped with a gauge which measures the force in unit pounds exerted between the jaws.

Vacuum Box: The vacuum box shall consist of a rigid housing with a transparent viewing window on top and a soft, closed- cell neoprene gasket attached to the bottom of the housing. The housing shall be equipped with a bleed valve. A separate vacuum source shall be connected to the vacuum box such that a negative pressure can be created and maintained between 4 and 8 psi. The vacuum box shall be equipped with a vacuum gauge capable of registering a minimum of 10 psi in increments of ¾ psi. A sudsy solution consisting of soap and distilled water shall be dispensed on the seam immediately ahead of the vacuum box.

Air Pressure Test Equipment: This method shall apply only when the split hot wedge seaming method is used. Equipment shall consist of an air pump capable of generating and maintaining a positive pressure of between 30 and 40 psi. A manometer capable of reading up to 40 psi attached to a needle or nipple shall be used to pressurize the air channel in the seam.

Alternative testing methods shall be submitted to the Owner or his Authorized Representative for approval prior to commencement of testing.

B. Non-Destructive Testing:

Thickness: Prior to deployment geomembrane rolls or upon deployment of individual panels, the Geosynthetic CQA shall randomly check the thickness at a minimum of twice (once per side) per 100 linear feet, in conformance with the specification in Section 2.01.

Test Seams: Test trial seams shall be made at a minimum every 4 hours to verify that adequate conditions exist for field seaming to proceed. Each seamer shall produce a test seam at the beginning of each shift. In addition, if a seaming operation has been suspended for more than 1 hour or if a breakdown of the seaming equipment occurs, a test seam shall be produced prior to resumption of seaming operations.

Test seams shall be made in the field on pieces of the approved geomembrane. Each test seam shall be at least 4 ft long by 1 ft wide and with sufficient overlap for peel testing in the field tensiometer.

Five samples, 1 in. wide shall be taken across the seam using an approved template. The samples shall be tested in the field tensiometer, three in peel and two in shear and shall meet the requirements of Section 3.08(C).

If the seam fails to pass, the seaming apparatus shall not be used for field seaming until any deficiencies have been corrected. This shall be verified by the production and successful testing of two consecutive test seams.

<u>Vacuum Testing</u>: All extrusion welded and solid fusion welded seams shall be evaluated using vacuum box testing in accordance with test method ASTM D 5641. Any seam overlap will be trimmed prior to testing.

A sudsy solution shall be applied to the test section and the vacuum box placed over the section. The bleed valve is then closed and the vacuum valve opened. Once a tight seal has been established (3 to 8 psi), the test section shall be visually examined for a period of not less than 15 seconds to determine whether bubbling of the soapy solution is occurring. The vacuum box is then moved and the process is repeated on the next adjacent section. A minimum 3 in. overlap shall be provided between all test sections.

All locations where bubbling of the sudsy solution was observed shall be clearly marked for repairs with a high visibility marker and recorded by number on field test reports. Any failed portion of seam shall be repaired by cap strip in accordance with Section 4.03(E).

<u>Air Pressure Testing</u>: All hot air/hot wedge seams shall be evaluated using air pressure testing. The seam shall be sealed off at one end and air passed through insuring an open passage. Once the seam is proven continuous the second end of the seam shall be sealed. If a seam end will be an integral part of the geomembrane the sealing shall be done in such a way that it does not harm the geomembrane. The seam should be pressurized to 30-35 psi. The feed valve shall be closed and the pressure sustained for a period of not less than 5 minutes. If a pressure loss of greater than 2 psi is observed or if the required pressure cannot be reached then the seam shall be rejected.

All faulty areas along the seam shall be identified and repaired by cap strip in accordance with Section 3.08(D). Vacuum testing shall be allowed on split wedge welds only when the faulty area cannot be isolated using air pressure testing. Any overlapping material must be removed prior to vacuum testing. All holes created during air pressure testing shall be sealed on completion of the test and vacuum tested.

All seams shall be non-destructively tested by the Installer over their full length to verify the integrity of the seam. Non-destructive testing shall be performed concurrently with field seaming. Prefabricated field seams which will be inaccessible after installation, such as those under structures or fastened to penetrations, shall be tested prior to final installation. All non-destructive testing shall be observed and documented by the Geosynthetic CQA.

Approved non-destructive testing procedure is as above. Alternate procedures shall be submitted for approval to the Geosynthetic CQA prior to the commencement of non-destructive testing.

Membrane Penetrations: Accessible pipe boot seems for all membrane penetrations shall be vacuum tested as described above. Areas not accessible to vacuum testing shall be non-destructively tested using an electric spark test. The electric spark test shall employ a continuous length of 24 gauge copper wire, placed under the geomembrane seam within 1/4-inch of the edge. A portable pulse-type detector, equipped with a brush-type electrode, charged with a low amperage current of 20,000 to 30,000 volts, will be advanced along the seam at approximately 20 to 30 feet per minute. Seam defects are detected when a spark arks from the sire to the electrode, closing the circuit and sounding an audible alarm. All seam defects shall be repaired and retested as described herein. Alternative testing methods shall be submitted to the Geosynthetic CQA and the MEDEP for approval prior to commencement of testing.

C. Destructive Testing:

Destructive testing of field seams shall be performed at selected locations in order to verify the criteria given in section "Test Seams". All sampling and testing shall be done concurrently with field seaming so that verification of field seam properties is made as the work progresses and corrective action implemented.

Test samples shall be taken at an average frequency of one test location per 1,000 ft of seam for dual-wedge seams and one test per 500 ft of seam for extrusion seams. A minimum of one test sample shall be taken from each seam that connects to existing geomembrane liner. More frequent sampling shall be performed at the discretion of the Geosynthetic CQA based on field trial welds, destructive seam test results, weather conditions, inspection of seams, and overall seaming performance. Sample locations shall be determined by the Geosynthetics CQA. The Installer shall not be informed in advance of the locations where the seam samples will be taken.

The Geosynthetic CQA may increase the amount of destructive testing based on the results of previous testing. Additional samples may also be required when the Geosynthetic CQA has reason to suspect the presence of excess crystallinity, contamination, faulty seaming equipment or any other reason affecting seam quality.

The test sample shall measure approximately 12 in. wide by 48 in. long with seam centered lengthwise along the sample. Five one-in. wide coupon strips shall be cut using an approved template from the ends of the sample. These coupons shall be tested by the Installer in the field tensiometer in both peel and shear in accordance with section "Test Seams". The remainder of the sample shall be cut into two 12-in. lengths, and one 18-in. length. The 18-inch sample shall be taken by the Geosynthetic CQA for independent laboratory testing; of the remaining two, one shall be given to the Installer for his own records or testing, and one shall be kept by the Owner for permanent record.

Samples shall be cut by the Installer under the direction of the Geosynthetic CQA. Each sample shall be indelibly numbered and identified. The sample number and location shall be recorded by the Installer on the panel layout drawing and on the sheet where the sample was taken.

The results of laboratory testing shall be made available to the Installer by the Geosynthetic CQA not more than 48 hours after the samples have been received by the testing facility. The results of laboratory testing shall determine the acceptability of a seam. Laboratory testing shall be performed in accordance with the methods given in ASTM D 6693 and ASTM D 6392 and meet the following criteria:

Test	Test Designation	Requirement
Bonded Seam Strength (Shear)	ASTM D-6392-99 and ASTM D-6693	4 of 5 coupons must be greater than 90% of parent material strength with none less than 70% of parent material strength* and have an acceptable mode of rupture
Peel Adhesion (both tracks)	ASTM D-6392-99, and ASTM D-6693	4 of 5 coupons on each side of the weld must be greater than 60% of parent material strength* or 70% of the specified value, whichever is greater, with none less than 50% of parent material strength* and have an acceptable mode of rupture

* Parent material strength shall be defined as the maximum yield tensile strength value in the cross machine direction of the manufacturers roll certification testing for all the geomembrane rolls delivered the site and the conformance test results. The Geosynthetic CQA is responsible for reviewing both the conformance and roll certification test results to determine the parent material strength value. The Geosynthetic CQA Project Manager and the MEDEP shall approve the parent material strength value.

Tests for peel adhesion and Shear shall be in a free condition (not 90^{o} or 180^{o}). Acceptable Locus of break codes for the specimen rupture mode shall be the following:

For Dual Wedge Seams: BRK, SE1, SIP and AD-BRK ≤ 25% adhesion failure;

For Extrusion Welds: SE1, SE2, SE3, BRK1, BRK2, HT, SIP, AD-WLD if strength is achieved, AD-BRK ≤ 25% adhesion failure.

Any seam that fails laboratory testing shall be repaired in accordance with section 3.08(D). The costs of repairing and retesting areas which failed destructive tests shall be the responsibility of the Installer.

The area from which the destructive test sample was taken shall be repaired without delay in accordance with the procedures given in section 3.08(D).

D. Inspection and Acceptance: As the work progresses, the Geosynthetic CQA shall document all locations requiring repair work and shall verify and document that all repairs have been successfully made by the Installer.

A field seam shall only be considered acceptable when bounded by two destructive test locations which have passed laboratory testing and applicable non-destructive testing. The following procedures shall apply in the event that a seam fails laboratory testing:

The Installer may reconstruct the seam with cap strips between the previous passing test location and the next passing test location (up to a maximum 75 feet) and retest, or;

The Installer may elect to trace the extent of an unacceptable seam to some intermediate location. This shall involve taking 1-in. template-cut cross-sections from the seam at a minimum distance of 10 ft in both directions from the failed test location. These samples shall be tested in the field tensiometer in both shear and peel in accordance with section "Test Seams". If one or both of these samples fail the field test, tracing along the seam shall continue at minimum 10-ft increments until a passing result is recorded in both directions from the failed test location. At these locations large samples shall be cut for laboratory testing as in section "Destructive Testing". If laboratory testing verifies the acceptability of the seam at these locations, the Installer shall reconstruct the seam (with cap strips) between the two passing test locations. If laboratory testing shows the seam to be unacceptable, the Installer shall further trace the unacceptable seam until acceptable test results are recorded in both directions.

Reconstructed seams shall be capped by cutting out the unacceptable seam, at least 6 inches each side of the seam and a minimum 6" beyond the defect, and patching with an acceptable material.

Reconstructed seams less than 150 ft in length shall be non-destructively tested in accordance with section "Non-Destructive Testing (3.08(B)". Reconstructed seams greater than 150 ft in length shall be destructively tested in accordance with section "Destructive Testing (3.08(C)."

The entire geomembrane surface shall be examined by the Geosynthetic CQA to confirm that it is free of any defects, holes, blisters, undispersed raw materials, or contamination by foreign matter. The geomembrane surface shall be cleaned by the Installer, if required so that it is free of dust, mud, debris or any other material which may inhibit a thorough examination of the surface. Any suspect areas shall be clearly marked by the Geosynthetic CQA and non-destructively tested in accordance with section "Test Seams."

E. Overburden: The Geosynthetic CQA shall monitor all overburden soils over the geomembrane liner and geocomposite. The Geosynthetic CQA shall identify any large wrinkles which may have been built into the geomembrane. Any such wrinkle not built in to accommodate thermal contraction of the geomembrane prior to placement of the overburden shall be cut, repaired and tested by the Installer.

The Geosynthetic CQA shall identify any slope toe, declivity, or other surface transitions which might result in bridging of the geomembrane during placement of the overburden. Any such area shall be cut, repaired and tested by the Installer.

Equipment used for placing and compacting the overburden shall not be driven directly on the geomembrane. Such equipment shall be closely monitored during placement to ensure that no damage occurs.

A minimum thickness of 1 ft of cover shall be maintained between the geomembrane and light earth moving equipment. Such equipment shall have a maximum ground pressure of 5 psi. Equipment shall have no cleats and no turning of any equipment shall be allowed on the initial 1 ft of cover. A minimum thickness of 3 feet of cover shall be maintained between the geomembrane and all rubber-tired earthmoving equipment.

In all cases, the placement of overburden shall be done with caution and in a manner which is least likely to cause wrinkles in, or damage to, the geomembrane.

For grades greater than 2 percent, all soil shall be placed in an upslope direction.

3.09 COMPLETION OF WORK:

A. The installation of the geomembrane shall be considered totally complete when:

1. The installation of the lining system, or section thereof, is finished.

2. Verification of the adequacy of all seams and repairs, including associated testing, is completed.

3. All documentation of installation is completed.

4. The leak location survey has been performed on the primary geomembrane liner in accordance with ASTM D7007 (Standard Practices for Electrical Methods for Locating Leaks in Geomembranes Covered with Water or Earth Materials), any leaks found have been repaired appropriately by the liner installer and a report documenting the results of the survey has been reviewed and approved by the Geosynthetic CQA Project Manager.

5. The Geosynthetic QARE is able to recommend acceptance.

6. The Owner and/or his Authorized Representative is satisfied that the geomembrane has been installed in accordance with the above Specifications.

END OF SECTION

SECTION 02772

LEAK LOCATION SURVEY

PART 1 - GENERAL

1.01 RELATED DOCUMENTS

A. Drawings and General Terms and Conditions as outlined in Section 1 of the Construction Agreement and Division-1 Specification sections, apply to work of this section.

B. Requirements set forth by the Quality Assurance/Quality Control Plan shall apply to the work specified in this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE:

A. Geomembrane Liner: Section 02771

B. Drainage Geocomposite: Section 02272

C. Earthwork: Section 02200

1.03 DESCRIPTION:

A. Leak Detection Survey to be performed by Leak Location Services, Inc. of San Antonio Texas

B. Leak Detection Survey shall be performed on the geomembrane liner in accordance with ASTM D 7007 (Standard Practices for Electrical Methods for Locating Leaks in Geomembrane Covered with Water or Earth Materials).

C. The General Contractor and liner installation subcontractor shall furnish all labor, material, and equipment to support the Leak Survey crew, including line surveys, water, water truck with a driver, hose, electrical (110VAC. 5A) source/supply, a supervised crew that will prepare the soils for the leak survey, and any repairs of the geomembrane.

D. The leak location contractor shall furnish all labor, materials and equipment to perform the leak survey and any necessary reporting.

1.04 SUBMITTALS:

Leak location contractor shall submit: a location plan and or profile of proposed electrode placement; a drawing showing any necessary points to be surveyed prior to arriving on site; any necessary additional requirements; and a proposed schedule beginning at arrival on site.

1.05 QUALITY CONTROL PERFORMANCE TEST:

After excavating the leachate collection sand and removing a 2-foot diameter circle of drainage geocomposite, a test hole with a diameter of 0.25 inches shall be made in the geomembrane. A drill or other suitable instrument shall be used to remove rather than displace the material in the hole. The drill bit must be reciprocated within the hole to remove burrs from the edges or bottom side of the test hole. The test hole must be made in an area where the geomembrane has intimate contact with the supporting sub-grade or GCL.

If electrodes are installed under the geomembrane (not necessary in single liner systems), the test hole shall be placed at the farthest position away from any electrode, but at least 50 feet from the edge of the geomembrane

The Contractor will survey and document the location of the test hole relative to site benchmarks. Leak location measurements shall be made and recorded along closely spaced parallel lines in the vicinity of the test hole. This data shall be analyzed to determine the leak detection distance for detecting the 0.25-inch test hole.

PART 3 - EXECUTION

A. Preparation: Any electrodes to be installed under the geomembrane shall be installed prior to the geomembrane. Leaks must be filled with moisture or earth material to be detected. To accomplish this, the following must occur at least 24 hours prior to the start of the survey:

   Uniformly wet the earth materials covering the geomembrane with water to field capacity. All leachate collection sand or stone shall be kept a minimum of 5 feet from the anchor trench for the duration of the Leak Test and there shall be a dry area located between the sand/stone and the edge of the geomembrane. The drainage inlet structures shall be constructed after the completion of the leak location survey to avoid masking of a potential leak signal. Any metal materials that will potentially interfere (mask a potential leak) with the leak survey including liner battens, couplings shall be insulated prior to the leak survey.

B. Leak Survey: The leak location data shall be taken on survey lines spaced no farther apart than twice the leak detection distance determined for a 0.25 inch diameter leak in the performance test. The measurement electrode spacing shall be no less than that used for the performance test. The spacing between measurements shall be no more than that used for the performance test.

   Data shall be recorded, plotted and analyzed for leak signals. The positions for these leak signals shall be located and the leaks excavated and repaired by the liner installer at the contractor's expense. Additional leak location survey data shall be collected near the located leak after the leak is repaired and electrically isolated to ensure no additional leaks are present. The survey data shall be repeated on the two closest survey lines for a distance extending at least 20 feet before and beyond the leak signal. If another leak signal is detected, this process shall be repeated until no additional leaks are detected.

C. Reporting: The daily results of the work shall be communicated to the owner's representative. A list of locations of the leaks found will be submitted to the Owners Representative after completion of the field work and before the survey personnel leave the site. A report documenting the electrical leak location surveys shall be submitted within 14 days of the completion of each leak survey. The reports shall document the methodology used to locate and repair the leaks, a description of the size and nature of the liner defect, and a diagram of the cell showing the approximate leak locations.

END OF SECTION

SECTION 02780

INTERFACIAL FRICTION ANGLE CONFORMANCE TESTING

PART 1 - GENERAL

1.01 RELATED DOCUMENTS:

A. Drawings and General Terms and Conditions as outlined in Section 1 of the Construction Agreement and Division-1 Specification sections, apply to work of this section.

B. The requirements set forth by the Quality Assurance/Quality Control Plan shall apply to the work specified in this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE:

A. Project Coordination: Section 01041

B. Geotextiles and Drainage Geocomposite: Section 02272

C. Geosynthetic Clay Liner: Section 02275

D. Geomembrane Liner (High-Density Polyethylene (HDPE)): Section 02771

E. Earthwork: Section 02200

1.03 DESCRIPTION OF WORK: The work in this Section includes all labor, materials, tools, and equipment necessary to perform conformance interfacial friction angle testing for the following interfaces for both the primary and secondary liner:

1. Drainage sand and drainage geocomposite.
2. HDPE textured geomembrane and drainage geocomposite.
3. HDPE textured geomembrane and GCL (non-woven side)
4. GCL internal
5. GCL (woven side) and compacted clay
6. HDPE Texture geomembrane and compacted clay

1.04 QUALITY CONTROL: (furnished by Geosynthetic Laboratory)

A. Geosynthetics Quality Conformance Laboratory (QCL) Experience. The testing laboratory performing the interfacial friction angle conformance testing shall be accredited by the Geosynthetics Accreditation Institute for interfacial friction angle testing and shall have satisfactorily demonstrated previous experience by letter of certification. Certification shall indicate the testing laboratory's experience with the materials to be tested and any limitations the materials may evoke on the testing program.

B. Test Method. The Geosynthetics QCL shall perform the required interfacial friction angle testing in accordance with ASTM 5321-02 and ASTM D 6243-98 for all GCL interfacial friction angle testing.

C. Test Reports. The Geosynthetics QCL shall provide test results to the project manager within 5 days of receipt of test samples. Test results shall be in the form of figures that present shear force versus displacement and shear stress versus normal stress. Both the peak strength and the large displacement strength shall be plotted. The laboratory shall report any influences or conditions that may have affected the test results. The laboratory shall indicate the correlation coefficient of the best fit lines drawn through the strength data, and the resulting peak strength and large displacement strength values for adhesion and friction angle.

PART 2 – MATERIALS AND TESTING CONDITIONS

2.01 MATERIAL SAMPLING:

A. Materials to be tested shall be obtained from materials that will be placed into service at the Juniper Ridge Landfill site. Including both primary and secondary liner.

B. Sample size shall be determined by the Geosynthetic QCL requirements.

C. Soil components used in the laboratory testing shall be obtained from the borrow source or from soil stockpiles to be utilized in the construction of the soil components of the landfill.

D. Sample and testing frequency for geosynthetics components shall be as indicated below. These tests and sampling frequency apply to both the primary and secondary liners

Interface	Testing Frequency
Drainage sand/drainage geocomposite	2 tests
HDPE textured geomembrane/drainage geocomposite	2 tests
HDPE textured geomembrane/GCL	2 tests
GCL internal	1 tests
GCL/compacted clay	2 tests
HDPE textured geomembrane /compacted clay (secondary liner)	2 tests

Test frequency represents minimum number of tests. Additional tests may be required at the discretion of the CQA project manager.

2.02 TESTING CONDITIONS:

The following testing conditions shall be utilized for interfacial friction angle testing:

A. Use 12-inch by 12-inch-square direct shear apparatus as defined by Test Method ASTM 5321-92 and ASTM D 6243-98 for all GCL interfacial friction angle testing.

B. Test specimens shall be fully secured to the direct shear apparatus to prevent premature slippage.

C. Use site-specific soils and materials.

D. Test all geosynthetics in the direction parallel to the length of the roll (machine direction).

E. Orient surface texturing of HDPE textured geomembrane so that machine direction is oriented parallel to the direction of movement of the testing apparatus.

F. Soil components shall be remolded into the testing apparatus according to the project earthwork specification Section 02200.

G. Tests shall be run wet.

H. The seating pressure, seating time, normal pressure(s), consolidation time and strain rate for each interface to be tested shall be as indicated below:

Table 3.1

Interface	**Seating Stress** (psi)	**Soak Time** (hrs) (prior to application of normal stress)	**Normal Stress(es) (psf)**	**Consolidation Time After Application of Normal Pressure (hrs)**	**Shear Force Displacement Rate (in/min.)**
Drainage sand and drainage geocomposite	NA	1	1,440, 4,320, 7,200, 16,000, 20,000	4	0.2
HDPE textured geomembrane and drainage geocomposite	NA	NA	1,440, 4,320, 7,200, 16,000, 20,000	4	0.2
HDPE textured geomembrane and GCL (nonwoven geotextile side)	1	48	1,440, 4,320, 7,200, 16,000, 20,000	24	0.004
GCL internal	1	48	1,444, 4,320, 7,200, 16,000	24	0.004
GCL (woven geotextile side) and compacted clay	1	48	1,440, 4,320, 7,200, 16,000, 20,000	24	0.004

All tests shall be run out to 20 percent strain in the shear displacement direction. The large-displacement strengths shall be defined as the strength occurring at maximum horizontal test strain.

PART 3 –REQUIREMENTS

3.01 The geosynthetic materials tested shall demonstrate their adequacy for use in the construction of the landfill by meeting or exceeding the following requirements:

A. The interfaces listed below shall have test results for peak strength and large displacement strength that plot above the “average minimum strength envelope” as defined by Table 3.2.

Between drainage sand and drainage geocomposite.
Between HDPE textured geomembrane and drainage geocomposite.
Between HDPE textured geomembrane and GCL (non-woven)
Between GCL (woven) and compacted clay
Between HDPE textured geomembrane and compacted clay

Table 3.2

Normal Stress (psf)	Peak Shear Stress (psf)	Large Displacement Shear Stress (psf)
1,440	830	463
4,320	1,811	924
7,200	2,315	1,480
16,000	3,855	3,179
20,000	4,555	3,951

B. The internal shear strengths listed below shall have test results for peak strength and large displacement strength that plot above the “average minimum strength envelope” as defined by Table 3.3.

GCL Internal

Table 3.3

Normal Stress (psf)	Peak Shear Stress (psf)	Large Displacement Shear Stress (psf)
1,440	1,500	430
4,320	3,000	870
7,200	4,000	1,060
16,000	6,000	1,793

3.02 REVIEW OF TEST RESULTS:

The Geosynthetics CQA project manager shall review all test reports to determine if the test results meet the minimum requirements stated above.

3.03 RETESTING:

The owner, contractor, installer, and the manufacturer may elect to retest failed tests. Testing may be done at the same laboratory or an independent laboratory. The testing laboratory shall be approved by CQA project manager and the testing conditions shall be in accordance with ASTM 5321-92, ASTM D 6243-98 for all GCL interfacial friction angle testing, and this section. Testing shall be done at contractor’s installer’s, or manufacturer’s expense. Test results shall be reviewed by the CQA project manager.

END OF SECTION

SECTION 15110

VALVES AND PIPE ACCESSORIES

PART 1 - GENERAL

1.01 RELATED DOCUMENTS: Drawings and General Terms and Conditions as outlined in Section 1 of the Construction Agreement and Division-1 Specification sections, apply to work of this section.

The requirements set forth by the Quality Assurance/Quality Control Plan shall apply to the work specified in this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE:

A. Pipe Installation: Section 02450

B. Manholes, Catch Basins, and Drainage Structures: Section 02570

C. HDPE Pipe and Fittings: Section 15100

1.03 REFERENCE:

A. ASTM A536-84(1999)e1, "Standard Specification for Ductile Iron Castings"

B. ASTM A126-95(2001), "Standard Specification for Gray Iron Castings for Valves, Flanges, and Pipe Fittings"

C. ASTM A743/A743M-03, "Standard Specification for Castings, Iron-Chromium, Iron-Chromium-Nickel, Corrosion Resistant, for General Application"

1.03 DESCRIPTION OF WORK:

A. Work of this Section shall consist of furnishing all labor, materials, and equipment to install valves, fittings and other appurtenances. Only the appropriate portions of this section pertaining to the specific contract work identified in Section 01010 "Summary of Work" or as directed by the Engineer, will apply.

B. Accept valves on site in shipping containers with labeling in place. Inspect for damage.

1.04 SUBMITTALS:

A. Product Data: Submit Manufacturers shop drawings, technical product data, installation instructions and catalog information for each valve fitting or accessory.

B. The Contractor shall submit the Quality Control Documentation as specified in the QA/QC Plan contained in Appendix B of this document. .

C. Manufacturer's Installation Instructions: Submit hanging or support methods and joining procedures.

PART 2 – PRODUCTS

2.01 BALL VALVES:

A. General: Provide ball valves for the pipe size as indicated on Contract Drawings.

B. Description:
   1. Valves for sizes 4" diameter to 8" diameter shall be KTM Model EB100, full bore, metaltite seats as manufactured by Tyco International. Valves for size 12" diameter shall be KTM Model EO1100 as manufactured by Tyco International.

   2. The ball shall be 316, stainless steel.

   3. Seals and O-rings shall be constructed of Viton® where possible and applicable.

   4. Flanges shall be drilled according to ANSI B16.5, Class 150.

2.02 PLUG VALVES:

A. General: Provide plug valves for the pipe size as indicated on the Contract Drawings.

B. Description:
   1. Plug valves shall be PEC Eccentric Plug Valves as manufactured by DeZURIK. The valve shall have a cast iron body with a stainless steel plug.

   2. The plug shall be 316, stainless steel. The body can be either cast iron or stainless steel provided it meets the pressure requirement.

   3. Seals and O-rings shall be constructed of Viton® where possible and applicable.

   4. Flanges shall be drilled according to ANSI B16.5, Class 150.

2.03 CHECK VALVES:

A. General: Provide check valves for the pipe size as indicated on the Contract Drawings.

B. Description:

   1. Check valve shall be a rubber flapper swing check valve as manufactured by APCO and be capable of 100 psi operating pressure.

   2. The flapper material shall be Viton A®.

   3. The body shall be stainless steel and the inside shall be unlined.

   4. Check valves shall be equipped with a backflow device.

   5. The flanges shall be drilled according to ANSI B16.5, Class 150.

2.04 RESILIENT SEATED GATE VALVES:

A. Resilient seated Gate Valves shall be ductile iron body, bronze mounted, with 18-8, 304 stainless steel bolts, resilient wedge gate with two inch operating nut and a PIV single flange (lug) connection. Valves shall conform in every respect to AWWA C509 Valve shall be MJ-PIV, F-Series Resilient Wedge Valve as Manufactured by Clow or approved equal. Valves shall open left.

B. Valves shall be provided with "O" rings. The design of the valve shall be such that the seal plate can be fitted with new "O" rings while the valve is under pressure in a fully open position.

C. Valves shall have a 100 percent solids thermoset or fusion bonded epoxy protective coating, holiday-free in the waterway, which shall meet all requirements of AWWA C550. The epoxy coating shall not impart taste or odor to the water. The coating shall be a product acceptable to the National Sanitation Foundation (NSF) for use in potable water and shall be so listed in the most current NSF summary of approved products under ANSI/NSF Standard 61. The coating shall be applied and cured in strict conformance with the coating manufacturer's cautions and instructions. The coatings shall be applied by the valve manufacturer under controlled factory conditions and field application is strictly prohibited.

2.05 PIV INDICATOR POST:

A. Indicator post shall be used to actuate and indicate the closed or open status of remotely installed PIV valves.

B. Indicator post and any necessary extensions shall be supplied and installed as shown on the contract drawings. Indicator post supplied shall open in the same direction as the valve and indicate as such. All bolts shall be 316, stainless steel.

2.06 FITTINGS:

A. Fittings shall be flanged ductile iron. All fittings shall be cement lined, and coated as specified hereinbefore for ductile iron pipe. Fittings greater than 12 inches shall conform to ANSIA21.10 (AWWA C110) and fittings 12 inches or less shall conform to ANSI A21.11 (AWWA C111). Compact fittings shall be Class 150 conforming to ANSI A21.53 (AWWA C153) and shall be cement lined in compliance with AWWA C104 for fittings 12 inches or less. Compact fittings greater than 12 inches are not acceptable. Fittings shall come complete with gaskets, and 316 stainless steel bolts.

B. Plugs, caps and blind flanges shall be stainless steel and shall conform to the weights and dimensions shown and be provided complete with all necessary gaskets and 316 stainless steel bolts.

C. All fasteners (nuts and bolts) shall be 316, stainless steel and shall be the correct size and dimensions for the size flanges and size of pipe.

2.07 CONCRETE:

A. Concrete used for any purpose such as, but not limited to thrust restraints, encasements, and chimneys shall have a minimum compressive strength of 3000 psi and conform to the specifications contained in Section 03300, Cast-in-Place Concrete.

2.08 STAINLESS STEEL REPAIR CLAMPS:

A. Repair clamps shall be as manufactured by Romac Industries.

2.09 PRESSURE TRANSMITTERS:

A. Pressure transmitters shall be installed to monitor the interstitial space of the dual containment system. They shall have an operating pressure range from 0 to 150 psi. The pressure transmitters shall be MBS 33, Cable type, 4 – 20 mA output, as manufactured by Danfoss.

B. The interstitial space shall be accessed by installing a threaded insert through the end termination or side wall of the containment pipe inside the manhole. The pressure transmitter shall be installed into this threaded insert or a coupling. This shall be done on the lowest end elevation of the pipe run.

C. Threaded connections shall be installed using Teflon tape, or approved sealant.

2.10 PRESSURE GAUGE:

A. Pressure gauges shall be furnished and installed as shown on the contract drawings

B. Pressure shall be transmitted to the gauge by a (diaphragm seal) completely sealed fluid so as no leachate is in contact with the gauge. Fluid shall be a 50 percent mixture of ethylene glycol and water. Pressure gauges shall be protected from excessive line pressures by a stainless steel ball valve installed between the pressure sensor and the pressure gauge.

C. Pressure gauges shall be pressure type with an appropriate operating range and a 4-1/2 inch dial. Gauges shall be ½ inch bourdon tube type and calibrated in psi and the dial shall so indicate. Gauges accuracy of plus of minus ½ percent. Pressure ranges shall be 0 PSIG to 150 PSIG unless otherwise indicated.

2.11 PRESSURE TRANSDUCER:

A. Pressure transducers shall be installed in the leachate collection sand layer of the Cells 11-16 liner systems to measure the pressure head acting on the Geomembrane in the location shown on the contract drawings.

B. The pressure transducers shall have an operating pressure range from 0-5 psig and shall be Esterline's Series 700 4-20mA Open Face Model (Part Number 700-140-0005).

C. The pressure transducers shall be accessorized with the Esterline Option-009 Surge Protection Kit and the Series 810 Vent Filter (Part Number 810).

PART 3 - EXECUTION

3.01 GENERAL:

A. All materials shall be stored and handled in accordance with the manufacturer's recommendations.

B. Verify piping system is ready for installation.

C. Provide non-conducting dielectric connections wherever jointing dissimilar metals.
D. Install valves with stems upright or horizontal, not inverted.

E. Install valves as shown on contract drawings for shut-off and to isolate equipment, part of systems, or vertical risers.

F. Valves and Fittings shall be braced against movement by installation of yoke and stanchions as applicable.

G. Inspection: Pipe installation shall be subject to inspection by the Engineer for quality, adherence to line and grade, jointing, and proper backfill. Any joint not satisfactory to the Engineer shall be removed and remade to his satisfaction at the Contractor's expense. No pipe shall be backfilled until it has been approved by the Engineer.

END OF SECTION

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

TABLE OF CONTENTS
(continued)

TABLE OF CONTENTS
(continued)

Section No. Title Page No.

QUALITY ASSURANCE/QUALITY CONTROL PLAN

1.0 GENERAL

1.1 Scope

This Quality Assurance/Quality Control Plan (QAP) addresses the quality assurance and quality control of the installation of all facility components used by NEWSME Landfill Operations, LLC (NEWSME Operations) at the Juniper Ridge Landfill (JRL) in Old Town, Maine. Facility components included in this QAP include the following engineered systems: underdrain system, composite liner system, leachate collection system, leachate transport system, leachate storage systems, gas management systems, and stormwater management systems.

In the context of this plan, quality assurance refers to means and actions employed to assure conformity of the facility component production and installation with the project-specific QAP, drawings, specifications, and contractual and regulatory requirements. Quality assurance is provided by a party independent from production and installation. Quality control refers only to those actions taken to ensure that materials and workmanship meet the requirements of the plans and specifications. Quality control is provided by the manufacturers and installers of the various components of the facility, and by compliance with applicable sections of the construction specifications contained in the contract documents.

The scope of this QAP applies to characterization of manufacturing, shipment, handling, and installation of facility components. This QAP does not address design guidelines, installation specifications, or selection of facility components. The technical specifications define the quality of materials and workmanship used on the construction of the facility

The QAP is the means to assure the level of material and workmanship used in the construction of the facility components meets or exceeds the requirements of the design specifications and drawings. This QAP was developed based on U.S.EPA guidance included in "Construction Quality Assurance for Hazardous Waste Landfill Disposal Facilities", U.S.EPA/530-SW-86-031, October 1986, and "Quality Assurance and Quality Control for Waste Containment Facilities", U.S.EPA/600/R-93/182 September 1993.

1.2 Parties

The parties discussed in this section are associated with the ownership, design, manufacture, transportation, installation, and quality assurance of the facility components. The definitions, qualifications, and responsibilities of these parties are outlined in the following subsections.

1.2.1 Site Owner/Landfill Operator.

1.2.1.1 Definitions

The Owner of the Juniper Ridge Landfill site is the State of Maine acting through the State Planning Office. The operator of the JRL is NEWSME Landfill Operations, LLC (NEWSME Operations). Funding for the Expansion Project is by NEWSME Operations. The NEWSME Operations contact person for this project is Wayne Boyd.

1.2.2 Project Manager/Resident Project Representative.

1.2.2.1 Definitions

The Project Manager/Resident Project Representative (RPR) is the official representative of NEWSME Operations; in this plan, the term Project Manager/RPR shall apply equally to "Construction Coordinator", i.e., the individual responsible for coordinating construction and quality assurance activities for the project. Project Manager/RPR may also be referred to as the “Owner’s Representative” in other parts of these Bid Documents and Specifications.

1.2.2.2 Responsibilities

The Project manager is responsible for construction quality assurance activities. The Project Manager/RPR is responsible for the organization and implementation of the QAP for the project as outlined in Section 1.1 of this plan.

The Project Manager/RPR shall serve as communications coordinator for the project, initiating the resolution, pre-construction, and construction meetings outlined in Section 1.3. As communications coordinator, the Project Manager/RPR shall serve as a liaison between all parties involved in the project to ensure that communications are maintained.

The Project Manager/RPR shall also be responsible for proper resolution of all quality assurance issues that arise during construction.

1.2.2.3 Qualifications

The selection of the Project Manager/RPR is the direct responsibility of NEWSME Operations. Qualifications for this position include familiarity with the following:

1. Applicable QAPs.
2. General knowledge of the construction materials and techniques necessary to construct the facility.
3. Applicable regulatory requirements.
4. Company policies and procedures for project management.

1.2.3 Designer.

1.2.3.1 Definitions

The Designer is the individual and/or firm responsible for the preparation of the design, including plans and project-specific technical specifications for the facility components and systems. The Designer may also be referred to as the “Engineer” in other parts of these Bid Documents and Specifications.

1.2.3.2 Responsibilities

The Designer is responsible for performing the engineering design and preparing the associated drawings and specifications for all the components of the landfill facility. The Designer is responsible for approving all design and specification changes and making design clarifications necessitated during construction of the landfill and associated facilities. The Designer may attend the resolution and pre-construction meetings outlined in Section 1.3 of this plan upon the request of the Project Manager/RPR.

1.2.3.3 Qualifications

The Designer shall be a qualified engineer, certified or licensed as required by regulation. The Designer shall be familiar with all the landfill components and applicable regulatory requirements.

1.2.3.4 Submittals

The Designer shall submit the project design drawings and specifications to NEWSME Operations. The Designer shall submit completed design clarification forms to NEWSME Operations in a timely manner upon request.

1.2.4 Construction Quality Assurance Agent and Quality Assurance Resident Engineer

1.2.4.1 Definitions

The Construction Quality Assurance Agent (CQA) is a firm independent from NEWSME Operations that shall be responsible for observing and documenting activities related to the quality assurance of all phases of the landfill and associated construction activities, on behalf of NEWSME Operations. The required staffing level will be a function of the Installer's schedule.

In this QAP the term Quality Assurance Resident Engineer (QARE) shall be used to designate the engineer in charge of the project-specific quality assurance work. In this QAP the terms "construction Quality Assurance Agent (CQA)" and "Quality Assurance Resident Engineer" are interchangeable. In some cases the duties of the QARE described below may be shared by two individuals: A Quality Assurance Managing Engineer located at the headquarters of the CQA, and a Quality Assurance Resident Engineer located at the site. The personnel of the CQA also include Quality Assurance Monitors who are located at the site for construction observation and documentation. Construction Quality Assurance Agent and Quality Assurance Resident Engineers will also be described in this QAP by the specific tasks they oversee; specifically, Soils (i.e., Construction) QARE/CQA or Geosynthetic QARE/CQA.

1.2.4.2 Responsibilities

The QARE is responsible for observing and documenting activities related to the quality assurance of the production and installation of all landfill components. The QARE is responsible for implementation of the project QAP prepared by the Project Manager/RPR and coordination of the on-site and off-site materials testing program. The QARE is also responsible for issuing a final certification report, sealed by a registered professional engineer, as outlined in Section 2.0 of this QAP.

The specific duties of the QARE personnel are as follows:

1. The QARE:

    a. Reviews all design drawings and specifications.

    b. Reviews other site-specific documentation, including proposed layouts, and manufacturer's and installer's literature.

    c. Develops a site-specific addendum for quality assurance of materials and construction techniques (if necessary) with the assistance of the Project Manager/RPR.

    d. Administers the QAP, e.g., assigns and manages quality assurance personnel, reviews all field reports, and provides engineering review of all quality assurance related issues.

    e. Reviews changes to design drawings and specifications as issued by the Designer.

    f. Acts as the on-site (resident) representative of the Project Manager/RPR.

    g. Familiarizes Geosynthetic Quality Assurance Monitors with the site and the project QAP.

    h. Attends quality assurance related meetings, e.g., resolution, pre-construction, daily, weekly.

    i. Reviews Manufacturer and Installer certifications and documentation and makes appropriate recommendations.

    j. Reviews the Installer's personnel qualifications for conformance with those qualifications pre-approved for work on site.

    k. Manages the preparation of the as-built drawing(s).

    l. Provides on-site testing of soils for compaction, using Nuclear Methods.

    m. Sampling of on-site soils and other materials and coordinates testing.

    n. Reviews Geosynthetic Quality Assurance Monitor's daily reports, logs and photographs.

    o. Notes any on-site activities that could result in damage and/or delays.

    p. Reports to the Project Manager/RPR, and logs in the daily report, any relevant observations reported by the Geosynthetic Quality Assurance Monitors.

q. Prepares his own daily report.

r. Prepares the weekly report of construction activities.

t. Oversees the marking, packaging, and shipping of laboratory test samples.

u. Reviews the results of laboratory testing and makes appropriate recommendations.

v. Designates a qualified Quality Assurance Monitor to represent the QARE whenever he is absent from the site while operations are ongoing.

w. Reports any unapproved deviations from the QAP to the Project Manager/RPR.

x. Prepares the final certification report.

y. Verifies that the supporting soil has met requirements set in Section 4.3 and overburden soil placement requirements set in Section 4.4.

z. Complete geotechnical/monitoring program form, as presented in the site's Geotechnical Monitoring Plan.

2. The Quality Assurance Monitor:

a. Monitors, logs, photographs and/or documents all construction operations. Photographs shall be taken routinely and in critical areas of the installation sequence. These duties shall be assigned by the QARE.

b. Monitors the following operations for all construction activities.

(1) Material delivery.
(2) Unloading and on-site transport and storage.
(3) Sampling for conformance testing.
(4) Material placement.
(5) In-place conformance testing.
(6) Visual inspection by walkover.
(7) Construction stability.

c. Monitors and documents construction operations, including:

(1) Subgrade preparation, testing, and approval.
(2) Placement, testing, sampling, and approval of soil materials used to construct the landfill.
(3) Placement of geosynthetic materials that are not related to the landfill liner system.
(4) Installation and testing of piping systems and associated appurtenances.

(5) Installation and testing of pump stations and associated equipment.
(6) Installation and testing of the leachate storage and associated plumbing and equipment.
(7) Installation of the loading rack and loading arm.

1.2.4.3 Qualifications

The CQA shall be pre-qualified by NEWSME Operations. The CQA shall be experienced in quality assurance of landfill facilities. The CQA shall be experienced in the preparation of quality assurance documentation including: quality assurance forms, reports, certifications, and manuals.

The Quality Assurance Managing Engineer shall be a degreed engineer and be registered as a professional Engineer in the State of Maine. The Quality Assurance Resident Engineer shall be specifically experienced in the landfill construction techniques and shall be trained by the CQA in the duties of a QARE.

Quality Assurance Monitors shall be quality assurance personnel who have been specifically trained in the quality assurance of landfills.

The Geosynthetic Quality Assurance Monitor shall be certified by the National Institute for Certification in Engineering Technology (NICET), the Geosynthetic Institute's Construction Quality Assurance – Inspectors Certification Program (CQA-ICP) or equivalent or work under the direct supervision of a NICET or CQA-ICP certified professional.

1.2.4.4 Submittals

Pre-qualification: To be considered for pre-qualification, the CQA must provide the following information:

1. Corporate background and information.

2. Quality assurance capabilities:

   a. A summary of the firm's experience with landfill construction.

   b. A summary of the firm's experience in quality assurance, including installation quality assurance of similar landfill construction projects.

   c. A summary of quality assurance documentation and methods used by the firm, including sample quality assurance forms, reports, certifications, and manuals prepared by the firm.

   d. Resumes of key personnel.

Pre-installation: Prior to beginning work on a project, the CQA must provide the Project Manager/RPR with the following information:

1. Resumes of personnel to be involved in the project including QARE, and Quality Assurance Monitors.

2. Proof of professional engineering registration for the engineer to be designated as the Quality Assurance Managing Engineer (QAME).

3. Proof of the required quality assurance experience of the quality assurance personnel.

1.2.5 Geosynthetic Manufacturer

1.2.5.1 Definitions

The Manufacturer is the firm responsible for production of any of the various geosynthetic liner system components outlined in this QAP.

1.2.5.2 Responsibilities

Each Manufacturer is responsible for the production of its geosynthetic product. In addition, each Manufacturer is responsible for the condition of the geosynthetic until the material is accepted by the Project Manager/RPR upon delivery. Each Manufacturer shall produce a consistent product meeting the project specifications. Each Manufacturer shall provide quality control documentation for its product as specified in this QAP.

1.2.5.3 Qualifications

Each Manufacturer shall be pre-qualified by NEWSME Operations. Each Manufacturer shall provide sufficient production capacity and qualified personnel to meet the demands of the project. Each Manufacturer shall have an internal quality control program for its product that meets the requirements presented in this QAP.

1.2.5.4 Submittals

Pre-qualification: A Manufacturer shall meet the following requirements and submit the following information to be considered for pre-qualification:

1. Corporate background and information.

2. Manufacturing capabilities:

    a. Information on plant size, equipment, personnel, number of shifts per day, and capacity per shift.

    b. Daily production quantity available for NEWSME Operations' facilities.

    c. A list of material properties including certified test results, to which are attached geosynthetic samples.

    d. A list of at least 15 completed landfill or surface impoundment facilities totaling the minimum area (see project specification) identified in the project specifications, for which the Manufacturer has manufactured a geosynthetic. For each facility, the following information shall be provided:

        (1) Name and purpose of facility, its location and date of installation.

(2) Name of owner, project manager, designer, fabricator (if any) and installer.
(3) Type of geosynthetic, surface area of geosynthetic manufactured.
(4) Available information on the performance of the lining system and the facility.

3. The Manufacturer's quality control manual, including a description of the quality control laboratory facilities.

4. The origin (supplier's name and production plant) and identification (brand name and number) of resin used to manufacture the product.

Pre-installation: Prior to the installation of any geosynthetic material, a Manufacturer must submit to the Project Manager/RPR all quality control documentation required by the appropriate section of this QAP. This documentation shall be reviewed by the Geosynthetic Construction Quality Assurance Agent as outlined in Section 1.2.4 of this QAP before installation can begin.

1.2.6 Geosynthetic Installer.

1.2.6.1 Definitions

The Geosynthetic Installer is the firm responsible for installation of the geosynthetics. The Installer may be affiliated with the Manufacturer.

The Superintendent is responsible for the Installer's field crew. The Superintendent shall represent the Installer at all site meetings and shall be responsible for acting as the Installer's spokesman on the project.

The Master Seamer shall be the most experienced seamer of the Installer's field crew. The Master Seamer shall provide direct supervision over less experienced seamers.

1.2.6.2 Responsibilities

The Geosynthetic Installer shall be responsible for field handling, storing, deploying, seaming, temporary restraining and all other aspects of the geosynthetics installation. The Installer may also be responsible for transportation of these materials to the site and for anchor systems, if required by the project specifications. The Installer shall be responsible for submittal of the documentation listed in Section 1.2.5.4.

1.2.6.3 Qualifications

The Geosynthetic Installer shall be pre-qualified and approved by NEWSME Operations. The Installer shall be able to provide qualified personnel to meet the demands of the project. At a minimum, the Installer shall provide a Superintendent, Seamers and a Master Seamer as described below.

The Superintendent must be qualified based on previously demonstrated experience, management ability, and authority. The Superintendent, unless otherwise approved by the Project Manager/RPR, shall have previously managed, at a minimum, two installation projects which entailed the installation of at least a combined total of 10,000,000 sq. ft of polyethylene geomembrane.

For geomembrane installation all personnel performing seaming operations shall be qualified with not less than 500,000 sq. ft of polyethylene geomembrane seaming experience. The Master Seamer shall have experience seaming a minimum of 1,000,000 sq. ft of polyethylene geomembrane using the same type of seaming apparatus to be used at the site.

1.2.6.4 Submittals

Pre-qualification: To be considered for pre-qualification, the Geosynthetic Installer shall submit the following information:

1. Corporate background and information.

2. Description of installation capabilities:

    a. Information on equipment (numbers and types), and personnel (number of superintendents, number of crews).

    b. Average daily production anticipated.

    c. Samples of field geomembrane seams and a list of minimum values for geomembrane seam properties.

3. A list of at least ten completed facilities, totaling a minimum of 10,000,000 sq. ft for which the Installer has installed geosynthetics. For each installation, the following information shall be provided:

    a. Name and purpose of facility, its location, and date of installation.

    b. Name of owner, project manager, designer, manufacturer, fabricator (if any), and name of contact at the facility who can discuss the project.

    c. Name and qualifications of the Superintendent(s) of the Installer's crew(s).

    d. Type of geosynthetic, and surface area installed.

    e. Type of seaming and type of seaming apparatus used.

    f. Duration of installation.

    g. Available information on the performance of the lining system and the facility.

4. The Geosynthetic Installer's quality control manual.

5. A copy of a letter of recommendation supplied by the geomembrane manufacturer.

Pre-installation: Prior to commencement of the installation, the Installer must submit to the Project Manager/RPR:

1. Resume of the Superintendent to be assigned to this project, including dates and duration of employment.

2. Resume of the Master Seamer to be assigned to this project, including dates and duration of employment.

3. A panel layout drawing showing the installation layout identifying field seams as well as any variance or additional details which deviate from the engineering drawings. The layout shall be adequate for use as a construction plan and shall include dimensions, details, etc.

4. Installation schedule.

5. A list of personnel performing field seaming operations along with pertinent experience information.

6. All geosynthetic quality control certificates as required by this QAP (unless submitted directly to the Project Manager/RPR by the Geosynthetic Manufacturer).

7. Certification that extrudate to be used is comprised of the same resin as the geomembrane to be used.

This documentation shall be reviewed by the Geosynthetic Construction Quality Assurance Agent, as outlined in Section 1.3.3 of this QAP, before installation of the geosynthetic can begin.

Installation: During the installation, the Installer shall be responsible for the submission of:

1. Quality control documentation recorded during installation.

2. Subgrade surface acceptance certificates for each area to be covered by the lining system, signed by the Geosynthetic Installer.

Completion: Upon completion of the installation, the Installer shall submit:

1. The warranty obtained from the Manufacturer.

2. The installation warranty.

1.2.7 Geosynthetic Quality Assurance Laboratory

1.2.7.1 Definitions

The Geosynthetic Quality Assurance Laboratory (QAL) is a firm, independent from the Geosynthetic Manufacturer(s) and Geosynthetic Installer responsible for conducting tests on samples of geosynthetics taken from the site.

1.2.7.2 Responsibilities

The Geosynthetic QAL shall be responsible for conducting the appropriate laboratory tests as directed by the Geosynthetic QAE. The test procedures shall be done in accordance with the test methods outlined in this QAP and/or the project specifications. The Geosynthetic QAL shall be responsible for providing test results as outlined in Section 1.2.7.4.

1.2.7.3 Qualifications

The Geosynthetic QAL shall have experience in testing geosynthetics and be familiar with American Society for Testing and Materials (ASTM), Federal Test Method Standard (FTMS), National Sanitation Foundation (NSF), and other applicable test standards. The laboratory shall be GAI-2AP certified. The Geosynthetic QAL shall be capable of providing verbal results of destructive seam tests within 24 hours of receipt of test samples and shall maintain that standard throughout the installation. The Geosynthetic QAL shall be approved by the Project Manager/RPR or NEWSME Operations.

On-site laboratory facilities may be used by the Geosynthetic QAL, provided they are appropriately equipped and approved by the Geosynthetic QARE and the Project Manager/RPR.

1.2.7.4 Submittals

The Geosynthetic QAL shall submit destructive seam test results to the Geosynthetic QARE in written form within 48 hours of receipt of test samples unless otherwise specified by the Project Manager/RPR. Geomembrane destructive test results shall typically be provided verbally to the Geosynthetic QARE within 24 hours of receipt of test samples.

Written test results shall be in an easily readable format and include references to the standard test methods used.

1.2.8 Soils Quality Assurance Laboratory

1.2.8.1 Definitions

The Soils Quality Assurance Laboratory (SQAL) is a firm, independent from the contractor responsible for conducting tests on construction soils materials taken from the site.

1.2.8.2 Responsibilities

The Soils QAL shall be responsible for conducting the appropriate laboratory tests as directed by the Soils QARE or CQA. The test procedures shall be done in accordance with the test methods outlined in this QAP and/or the project specifications. The Soils QAL shall be responsible for providing test results as outlined in Section 1.2.8.4.

1.2.8.3 Qualifications

The Soils QAL shall have experience in testing geosynthetics and be familiar with American Society for Testing and Materials (ASTM), Federal Test Method Standard (FTMS), and other applicable test standards. The Soils QAL shall be capable of providing verbal results of destructive seam tests within 24 hours of receipt of test samples and shall maintain that standard throughout the installation. The Soils QAL shall be approved by the Project Manager/RPR and/or NEWSME Operations.

On-site laboratory facilities may be used by the Soils QAL, provided they are appropriately equipped and approved by the Soils QARE/CQA and the Project Manager/RPR.

1.2.8.4 Submittals

The Soils QAL shall submit soil test results to the Soils QARE in written form within 5 days of receipt of test samples unless otherwise specified by the Project Manager/RPR. Soil test results shall typically be provided verbally to the Soils QARE/CQA within 24 hours of receipt of test samples, or a time period as dictated by the testing requirements. Written test results shall be in an easily readable format and include references to the standard test methods used.

Written test results shall be in an easily readable format and include references to the standard test methods used.

1.3 Communication

To guarantee a high degree of quality during the construction process and assure a final product that meets all project specifications, clear, open channels of communication are essential. This section discusses appropriate lines of communication and describes all necessary meetings.

1.3.1 Lines of Communication. The Soils QARE/CQA and the Geosynthetic QARE shall be capable of direct communication with the Project Manager/RPR at all times. The project manager shall be capable of direct communications with NEWSME Operations and the MEDEP at all times.

1.3.2 Resolution Meeting. Following permit approval and the completion of the construction drawings and specifications for the project, a resolution meeting may be held. If a resolution meeting is required, it is recommended that the meeting be held prior to bidding the construction work and include the parties when involved, typically including the Project Manager/RPR, Designer, Soils QARE/CQA, Geosynthetic QARE, and a NEWSME Operations representative. If necessary, this meeting can be held in conjunction with the pre-construction meeting.

The purpose of this meeting is to establish lines of communication, review construction drawings and specifications for completeness and clarity, begin planning for coordination of tasks, anticipate any problems which might cause difficulties and delays in construction, and review the QAP. Aspects of the design shall be reviewed during this meeting so that clarification and/or design changes may be made before the construction work is bid. In addition, the guidelines regarding quality assurance testing and problem resolution must be known and accepted by all.

The meeting shall be documented by a person designated at the beginning of the meeting, and minutes shall be transmitted to all parties.

1.3.3 Pre-Construction Meeting. A pre-construction meeting shall be held at the site prior to the start of earthwork and geosynthetic deployment. Typically, the meeting shall be attended by the Project Manager/RPR, Designer, Geosynthetic Installer, Soils QARE/CQA, Geosynthetic QARE, and a NEWSME Operations representative. NEWSME Operations or their agent shall notify the MEDEP of the pre-construction meeting 7 days prior to the meeting date so that an MEDEP representative may attend if desired.

Specific topics considered for this meeting include review of the project QAP for any problems or additions. In addition, the responsibilities of each party should be reviewed and understood clearly. The meeting shall be documented by a person designated at the beginning of the meeting, and minutes shall be transmitted to all parties.

1.3.4 Progress Meetings. A weekly progress meeting shall be held between the Contractor, Soils QARE/CQA, Geosynthetic QARE, Geosynthetic Installer's Superintendent, Project Manager/RPR, and any other concerned parties. This meeting shall discuss current progress, planned activities for the next week, issues requiring resolution, and any new business or revisions to the work. The CQAs shall log any problems, decisions, or questions arising at this meeting in his weekly report. If any matter remains unresolved at the end of this meeting, the Project Manager/RPR shall be responsible for the resolution of the matter and the communication of the decision to the appropriate parties.

2.0 DOCUMENTATION

An effective QAP depends largely on identification of the construction activities that shall be monitored, and on assigning responsibilities for the monitoring of each activity. This is most effectively accomplished and verified by the documentation of quality assurance activities. The QAREs/CQAs shall document that all requirements in the geosynthetic portions of the project QAP have been addressed and satisfied.

The QAREs/CQAs shall provide the Project Manager/RPR with signed descriptive remarks, data sheets, and checklists to verify that all monitoring activities have been carried out. The QAREs/CQAs shall also maintain at the job site a complete file of the documents which comprise the QAP, including plans and specifications, checklists, test procedures, daily logs, and other pertinent documents.

2.1 Daily Reports

2.1.1 Geosynthetics Quality Assurance. Each Geosynthetic Quality Assurance Monitor shall complete a daily report and/or logs on prescribed forms, outlining all monitoring activities for that day. The precise areas, panel numbers, seams completed, and approved, and measures taken to protect unfinished areas overnight, shall be identified. Failed seams or other panel areas requiring remedial action shall be identified with regard to nature of action, required repair, and precise location. Repairs completed must also be identified. Any problems or concerns with regard to operations on-site should also be noted. This report must be completed at the end of each monitor's shift, prior to leaving the site, and submitted to the Geosynthetic QARE.

The Geosynthetic QARE shall review the daily reports submitted by the Geosynthetic Quality Assurance Monitors, and incorporate a summary of their reports into the Geosynthetic QARE's daily report. Any matters requiring action by the Project Manager/RPR shall be identified. The report shall include a summary of the quantities of the geosynthetics installed that day. This report must be completed daily, summarizing the previous day's activities and a copy submitted to the Project Manager/RPR at the beginning of the work day following the report date.

2.1.2 Construction Quality Assurance. Each Construction Quality Assurance Monitor shall complete a daily report and/or logs on prescribed forms outlining the monitoring activities undertaken for that day. The activities undertaken, materials used, location, time, testing done, samples taken, and test results obtained shall be documented. Failed areas shall be identified and the remedial action taken noted. Any problems and/or concerns regarding failed areas shall be noted. The monitor(s) shall also note the equipment used and the work force provided, and any subcontractors, provided during the daily operations. The report must be completed at the end of each monitor’s shift, prior to leaving the site, and submitted to the Soils QARE/CQA.

The Soils QARE/CQA will review daily reports submitted by the CQA monitor(s) and incorporate a summary of their reports into the Soils QARE’s/CQA’s daily report. Any actions requiring action by the Project Manager/RPR shall be clearly identified. The report shall include a summary of quantities of materials installed that day. This report must be completed daily, summarizing the previous day’s activities and a copy submitted to the Project Manager/RPR at the beginning of the work day following the report date.

2.1.3 Report Forms and Installation Logs. Each Quality Assurance Monitor shall document construction activities as described above on the pertinent forms or logs attached to the end of this Section.

2.2 Testing Reports

2.2.1 Geosynthetics Testing Reports. The destructive test reports from all sources shall be collated by the Geosynthetic QARE. This includes field tests, Installer's laboratory tests (if performed), and Geosynthetic QAL tests. A summary list of test samples pass/fail results shall be prepared by the Geosynthetic QARE on an ongoing basis, and submitted with the weekly progress reports.

2.2.2 Soils Testing Reports. The on-site testing and laboratory test reports from all sources shall be collected by the Soils QARE. This includes field tests and laboratory tests. A summary list of test samples of pass/fail results shall be prepared by the Soils QARE on an ongoing basis, and submitted with weekly progress reports.

2.2.3 Miscellaneous Test Reports. On-site testing of pressure pipe, manholes, pump stations, leachate storage tank, and other equipment shall be reported on a separate Equipment Test Report Log. Test reports shall be submitted to the Soils QARE/CQA as the equipment is tested, and shall be included in weekly progress reports.

2.3 Progress Reports

Progress reports shall be prepared by the CQAs/QAREs and submitted to the Project Manager/RPR. These reports shall be submitted every week, starting the first Friday of construction on-site. This report shall include: test results, submittals and action taken; summary of work progress; upcoming work items for the next two weeks; punchlist items; summary of problems encountered and how the problems were resolved; change order status; and construction stability monitoring results, if applicable. Other items may include delays caused by weather conditions or material shortages. A copy of these progress reports shall be forwarded to the MEDEP within one week after the completion of each construction week.

All CQA's/QARE's daily reports for the period should be appended to each progress report.

2.4 Record Drawings

2.4.1 Geosynthetic Record Drawings. Record drawings shall be prepared by the Geosynthetic QARE. The Record drawings shall include, at a minimum, the following information for geomembrane.

1. Location, as accurate as possible, of each panel relative to the site benchmarks (furnished by the Project Manager/RPR).

2. Identification of the seams and panels with appropriate numbers or identification codes (see Section 5.4.1).

3. Location of patches and repairs.

4. Location of destructive testing samples.

5. Pertinent as-built details, such as penetrations and anchor trenches, etc.

The record drawings shall illustrate each layer of geomembrane, and, if necessary, another drawing shall identify problems or unusual conditions of the geotextile or geonet layers. In addition, applicable cross-sections shall show layouts of geonets, geotextiles or geogrids in sump areas or any other areas which are unusual or differ from the design drawings.

2.4.2 Construction Record Drawings. Record drawings shall be prepared by the CQA Agent. The record drawings will incorporate the record drawings prepared by the Geosynthetic and Soils QARE. The record drawings shall include the above, plus the following information:

1. Horizontal and vertical location of base grades, top of 24-inch clay layer, and top of primary HDPE geomembrane liner, relative to the site benchmarks (furnished by the Project Manager/RPR).

2. The contractor will record changes to pertinent details and supply this information to the Soils QARE/CQA. The Soils QARE/CQA will show changes to the details on the record drawings.

3. Horizontal and vertical location of all external landfill components such as: pump stations; force mains; manholes; roadways; electrical conduits; utilities; gas collection system piping; and leachate storage facilities. These shall be located relative to the site benchmarks.

Record drawings shall be stamped by a Maine registered professional engineer in employ of the CQA.

2.5 Final Certification Report

A final certification report shall be submitted to NEWSME Operations upon completion of the work. This report shall summarize the activities of the project, and document the aspects of the quality assurance program performed. A copy of the report will be submitted to the MEDEP as part of the construction documentation report.

The final certification report shall include, at a minimum, the following information:

1. Parties and personnel involved with the project.

2. Certification, sealed and signed by a registered professional engineer.

3. Record drawings, sealed and signed by a registered professional engineer.

4. A narrative summary of the Geosynthetics QARE observation of the geosynthetic installation and handling activities, including placement of overburden soil.

5. A narrative summary of all phases of the landfill construction and associated facilities.

6. Copies of applicable specifications.

7. Written clarifications and interpretations of the specifications.

8. Change Orders to specifications.

9. Minutes from pertinent construction meetings.

10. Geosynthetics manufacturer's quality control documentation, certifications, warranties, and guarantees.

11. Copies of the following geosynthetics quality assurance records: conformance testing results; certificates of subgrade acceptance; temperature logs; panel deployment logs; trial seam logs; destructive testing results; non-destructive testing logs; repair logs and diagrams; CQA daily and weekly reports.

12. Copies of the following site construction quality assurance records: field compaction testing results; soil material conformance testing results; submittals; pipe testing records; pump station testing records; leachate storage tank leak testing records; and CQA daily and weekly reports.

13. Copies of photographs taken to document the progression of site construction activities.

The report shall include written certification by the Geosynthetic QARE and the Quality Assurance Managing Engineer that the installation was completed in accordance with the project QAP except as noted to the Project Manager/RPR.

3.0 LINING SYSTEM ACCEPTANCE

3.1 Geosynthetic Lining System

Upon written recommendation by the Geosynthetic QARE, the Project Manager/RPR shall consider accepting the geosynthetic lining system. The conditions of acceptance are described below. The Installer and Manufacturer(s) will retain all ownership and responsibility for the geosynthetics in the lining system until acceptance by NEWSME Operations.

The geosynthetic lining system shall be accepted by NEWSME Operations when:

1. The installation of the lining system, or section thereof, is finished.

2. Verification of the adequacy of all seams and repairs, including associated testing, is completed.

3. All documentation of installation is completed.

4. The leak location survey has been performed on the primary geomembrane liner in accordance with ASTM D7007 (Standard Practices for Electrical Methods for Locating Leaks in Geomembranes Covered with Water or Earth Materials), any leaks found have been repaired appropriately by the liner installer and a report documenting the results of the survey has been reviewed and approved by the Geosynthetic CQA Project Manager.

5. The Geosynthetic QARE is able to recommend acceptance.

6. The Owner and/or his Authorized Representative is satisfied that the geomembrane has been installed in accordance with the above Specifications.

The Geosynthetic QARE shall certify that the installation did proceed in accordance with the geosynthetic portions of the project QAP, except as noted to the Project Manager/RPR. This certification shall be provided in the final certification report as outlined in Section 2.5.

3.2 Landfill Facility Systems

Upon written recommendation by the CQA's Quality Assurance Managing Engineer, the Project Manager/RPR shall consider accepting the landfill systems. The conditions of acceptance are described below. The Contractor will retain all ownership and responsibility for the newly constructed landfill and associated facilities until acceptance by NEWSME Operations.

The landfill facility systems shall be accepted by NEWSME Operations when:

1. The installation of the landfill soil and geosynthetic lining system, landfill piping systems, pump stations, leachate transport and storage facilities, or sections thereof, are finished.

2. Verification of the adequacy of the materials placed, equipment supplied, facilities constructed, including all associated testing, is complete.

3. All documentation of installation is completed.

4. The CQA's Quality Assurance Managing Engineer is able to recommend acceptance.

The CQA's Quality Assurance Managing Engineer shall certify that installation has proceeded in accordance with the project QAP except as noted to the Project Manager/RPR. This certification shall be provided in the final certification report as outlined in Section 2.5.

4.0 SOIL COMPONENTS

4.1 Borrow Materials

Clay: For use in the imported soil and clay liner, must meet requirements for gradation, Atterberg limits, remolded hydrogeologic, and moisture content.

Underdrain Sand: For use in the underdrain systems, must meet requirements for gradation and remolded hydraulic conductivity.

Leachate Collection Sand: For use in the leak detection and leachate collection systems, must meet requirements for gradation and remolded hydraulic conductivity specified in Section 02200.

Leachate Collection Sump Stone: For use in the leachate collection system, must meet requirements for gradation.

Drainage Stone: For use in the underdrain and leachate collection systems, must meet requirements for gradation.

Access Road Base and Subbase Material: Must meet requirements for gradation.

Common Borrow: Must meet requirements as defined by Maine Department of Transportation (MDOT) Section 703.18.

Impervious Borrow: (N.I.T.C.) Must have greater than 35 percent by weight passing the No. 200 U.S. standard sieve.

Riprap: Must meet size requirements as defined by the specifications Section 02220.

The soils used in the construction of the Juniper Ridge Landfill (JRL) facility shall meet the CQC requirements outlined in the specifications. Changes in the materials must be approved by the Project Manager/RPR prior to placement.

Clay borrow used for imported soil layer and liner construction will come from an off-site clay pit and/or from any existing clay cover that is to be removed. The Soils QARE/CQA will be required to inspect and approve the use of the clay cover soils for liner construction. The recycled clay must meet the clay borrow requirements as defined by the specifications.

The following quality control procedure is incorporated into the project specifications to assure that the clay borrow source(s) delivered to the site meets the project specifications and provides the data to define quality control acceptance criteria. The procedure is to use the individual moisture density curves and associated hydraulic conductivity test from clay borrow source testing program to guide the clay placement.

4.2 Material Delivery, Storage, and Processing

Material used in the construction of JRL facility will require proper handling in order to assure that the specified design properties are not compromised. The Soils QARE/CQA must observe hauling operations and inspect materials as they are delivered. Loads shall be periodically inspected to assure that contamination is not occurring. Truck dump bodies shall be in clean condition prior to a change of material being handled. Hauling personnel shall be informed by the Contractor of the need to clean dump bodies when appropriate. The Soils QARE/CQA may elect to obtain samples for quality assurance testing. Testing under these circumstances will be at the contractor's expense.

Storage of materials shall be in such a manner so that the material properties will remain uncompromised by contamination until they are used. The Soils QARE/CQA shall instruct the contractor of a storage location. The contractor is responsible for keeping materials from being contaminated. The contractor must take measures to assure stockpiled materials meet the specifications as they are installed. The Soils QARE/CQA may elect to obtain samples for quality assurance testing.

Processing of soil materials at the site may be necessary. Clay material stripped from existing landfill cover and to be used as liner material may require processing. Processing may entail removal of large stones or organic material. Other processing may include homogenization of clay by mixing and stockpiling, and adjustment of clay moisture. The Soils QARE/CQA should monitor these activities and perform quality assurance testing as necessary.

4.3 Subgrade Preparation

Subgrades shall be properly prepared and compacted to the requirements outlined in Part 3.09 of Section 02200 and Part 3.06 of Section 02332 of these specifications. The Soils QARE/CQA is responsible for inspection of the subgrade and identifying areas which will require further compaction. Testing will be done as needed to assure that the subgrade can adequately support the subsequent compacted liner materials. Water should be added or removed to assure optimum compaction effort.

Soil approved to be left in place shall be proofrolled prior to clay layer placement. Proofrolling shall include a minimum of three passes of a heavy vibratory compactor. The type and weight of the compactor shall be approved by a qualified geotechnical engineer.

Unacceptable subgrade material is to be removed and replaced with a suitable compacted common borrow. Common borrow used to fill the subgrade is to be approved by the Soils QARE/CQA. Subgrade below any proposed geosynthetics shall be inspected by the Geosynthetic QARE and the installer. The Geosynthetic QARE will inform the contractor of subgrade deficiencies. The contractor will provide the necessary construction effort to alleviate any subgrade deficiencies. The Geosynthetic QARE and installer will re-inspect the subgrade. The installer will complete a Subgrade Acceptance form which documents the subgrade acceptance.

4.4 Placement, Remolding, and Compaction

4.4.1 Soil Material Placement. Placement of soil into a fill is to be done according to the specifications. For embankment and excavation fills, loose lift thicknesses shall not exceed 12 inches for material compacted by heavy compaction equipment, and not more than 6 inches in loose depth for material compacted by hand-operated tampers. For clay lifts, the loose lift thickness shall not exceed 9 inches. The Soils QARE/CQA will be responsible for implementation of clay placement guidelines as set forth in these specifications. The CQA Monitor(s) will observe the placement of clay in order to verify the clay is placed as required. The contractor is responsible for documentation of loose lift of clay thickness, drainage sand, and drainage stone as directed in the specifications. The contractor will provide this documentation to the Soils QARE/CQA on a weekly basis.

Placement of soil materials to compact will not be allowed in weather conditions that adversely affect material compaction. The contractor is responsible for protection of materials being placed and should take necessary measures to protect the materials.

Clay lifts to be installed up to old or previously placed clay lifts shall be matched-in by stepping the clay layers. Prior to placement of a new lift of soil, the surface of the previous compacted lift of soil liner should be roughened to promote a good bond between new and old lifts. CQA Monitor shall inspect lift surface to assure the surface is properly prepared.

Extreme care is required when placing soil materials over geomembrane liner. Prior to the commencement of sand placement, the contractor must provide the Soils QARE/CQA with a written plan stating the sequence of sand placement over the geomembrane. The plan should state where the site will be accessed; placement of haul roads, how sand is spread, and the personnel and equipment that will be utilized. The minimum thickness of sand over the geomembrane for low ground pressure equipment shall be one foot. Minimum thickness for rubber-tired equipment is three feet. The Soils QARE/CQA will observe the hauling of fully loaded vehicles over three foot sand haul roads and may require additional sand over the liner if conditions warrant. The contractor is responsible for maintaining sand haul roads so that damage does not occur to the geomembrane and/or the underlying clay layer. The CQA Monitor(s) will observe vehicle operation on sand haul roads to assure the operation is smooth (without sudden stops, quick turns, or hopping).

Equipment used for placement of the 12-inch sand blanket over the geomembrane shall have a ground pressure of no more than 5 psi. Equipment shall have no cleats and is not allowed to make tight, sudden turns.

Sand placed on the sideslope shall be placed by working from the toe of the slope to the top of the slope. The CQA Monitor will observe spreading of sand on sideslopes to assure no damage is done to liner materials and that liner materials are not slipping on the sideslopes. Equipment working on the sideslopes is not to make sudden stops, especially in the downhill direction.

Drainage stone placed within limits of the liner system shall be spread so that no damage occurs to the liner system. Stone placed over piping and lifts less than 18 inches in thickness shall be placed with an excavator, bucket loader, or similar type equipment. CQA Monitor will inspect drainage stone for contamination periodically, as lift is placed.

4.4.2 Remolding and Compaction. Proper remolding of soil materials is dependent upon the correct moisture content of the soil and the compaction effort. The Soils QARE/CQA will be responsible for being familiar with the soil properties being used for the soil components at the NEWSME Operations Landfill facility. Soils QARE/CQA shall have soils materials tested when their properties are in doubt. For the clay material the soil moisture/density relationship will be defined from test results from borrow source or conformance testing. The Soils QARE/CQA should be directly involved with the test pad and be knowledgeable of soil materials and workability. The contractor shall be responsible for notifying the Soils QARE/CQA of any changes in soil materials or borrow source. The CQA Monitor shall observe clay or soil placement to assure material meets the requirements set forth by the specifications. The Soils QARE/CQA may elect to perform additional testing, beyond what is required by the specifications, in order to define the soil properties and determine their acceptability.

Prior to compaction of clay soils the contractor will supply the Soils QARE/CQA with a plan detailing where the soil materials will be obtained, how and where they are to be placed within the new phase, method of moisture adjustment, equipment to be used, and measures of protection from the weather elements. Remolding the clay begins after proper scarification of the existing surface, followed by clay placement as described in Section 4.4.1 of this section. The loose clay lift is then compacted. The equipment used and the number of passes required to meet the project specifications shall be determined at the beginning of the clay placement with a minimum of three passes. The CQA Monitor will observe the compactive effort and periodically document the number of passes taken on a given section of soil liner. The surface of the clay will then be sealed by smooth roller. The CQA Monitor will measure the moisture/density by nuclear method, as outlined in Section 4.6.1.2 of this plan. Upon passing test results, the CQA Monitor will sample the clay, if necessary, and inform the contractor of the results. Non-passing test results will immediately be reported to the contractor. The contractor can request the area be retested or the area will be reworked by adjusting soil moisture and compactive effort. The CQA Monitor should observe these adjustment measures to assure that they are applied uniformly throughout the failed area. After reworking the soil lift the CQA Monitor will retest the lift. If the moisture density tests pass, the CQA Monitor may sample the area for hydraulic conductivity if required, and the contractor may resume normal lift construction. Non-passing results on the second trial suggest the material is deficient and must be removed and replaced with other suitable material. The area will be reworked until passing test results are obtained. The CQA Monitor shall document the efforts required to acquire passing test results in deficient areas.

The compaction requirements of other soil and subgrade materials used for construction of the JRL facility are outlined in the specifications. The CQA Monitor must document the testing of these materials. Deficient areas are to be reworked until passing test results are obtained. The CQA monitor will observe the placement of the clay layer to document that previous compacted clay lifts are scarified prior to placement of the next loose lift of clay. The CQA monitor will also document through visual observations that the clay clods are broken up as part of the compaction effort. The CQA will have the authority to require areas of the clay to be reworked if either inadequate lift bonding or lack of break-up, of the clay clods is observed.

4.5 Protection

Soil materials used in the construction of the soil components of the landfill require protection to assure that the quality of the material remains within the specified requirements during and after placement into the system. The CQA shall review the contractor's planned measures to protect the soil components. Protection measures must address soils being stockpiled for subsequent use in the landfill, soils in the process of being placed, and soils already placed. At a minimum, protection measures shall address wet weather conditions, dry weather conditions (desiccation), freezing temperatures, and cross contamination of materials during and after placement. The contractor shall be responsible for implementing protection measures required to assure soil components remain in accordance to the specifications. The CQA shall oversee implementation of the protection measures.

4.6 Quality Assurance Testing and Documentation

4.6.1 Construction Quality Assurance (CQA) Agent Responsibilities. The CQA will direct the sampling and testing of all soil materials to be used in the construction of the landfill components. The CQA will observe and document, borrow site material resources and on-site utilization of the soil materials. Duties shall include but not be limited to sampling, testing, observing, and documenting soil material utilization. The CQA shall also approve methods of construction and the equipment used to place and work the soil materials.

4.6.1.1 Quality Assurance Testing

The CQA shall implement the soil testing plan as defined by the construction specifications and the quality assurance manual. Borrow materials shall be sampled and tested by CQA at the required frequency as defined by the QAQC Plan and construction specifications. Tests performed on each soil component shall be approved by the CQA. Borrow materials from the point source shall be sampled and tested at frequency indicated on Table 2-1 of Section 02200 of the Specifications.

4.6.1.2 Quality Control Testing

Quality control testing of borrow materials to be used in the construction of the landfill shall be sampled and tested at the frequency indicated in Table 3-1 of Section 02200 of the project specification. Samples shall be obtained from stockpiles to be shipped to the site or from shipments arriving on-site. The quality control testing of clay borrow shall be sampled, tested, and approved by the CQA at the required frequency as defined by this QAP and the construction technical specifications (ref. Table 2-1, Specification Section 02200).

4.6.1.3 Standard In-place Testing

The tests indicated on Table 3-2 of Section 02200 of the project Specifications shall be performed to document the clay barrier soil meets the specified in-place properties. This testing shall be performed by the CQA.

4.6.2 Sampling Procedures

4.6.2.1 Borrow Site Characterization

Sampling of a borrow site to be mined for soil components to be utilized in the construction of the landfill shall be overseen by the Soils QARE/CQA. The sample size and location shall be dependent upon the material being mined, the amount to be mined, and the frequency of Construction Quality Control (CQC) testing required. The sample size will be as directed by the CQC testing laboratory and as approved by the Soils QARE/CQA. The frequency of sampling shall be as indicated in Table 2-1 of Section 02200 of the project specifications, or as directed by the Soils QARE/CQA.

4.6.2.2 Construction Testing of Borrow Source Materials

Soil materials processed or unprocessed that are to be used as part of the landfill shall be sampled at stockpiles or from truck loads arriving on-site prior to being installed into the landfill. Samples shall be taken under direction of the Soils QARE/CQA. The frequency of sampling shall be as indicated in Table 3-1 of Section 02200 of the project specifications, or as directed by the Soils QARE/CQA. The sample size will be as directed by the CQC testing laboratory and as approved by the Soils QARE/CQA.

4.6.2.3 In-Place Construction Test Samples

Samples of soil materials placed as components of the landfill shall be sampled as directed by the Soils QARE/CQA. In-place sampling locations shall be chosen by establishing a 75-foot square grid pattern over each lift, sample location shall be randomly picked from the grid nodes. Sample grid patterns shall be staggered in each successive lift so that locations vary from lift to lift.

Should a failing test occur, the Soils QARE/CQA shall define the limit of the failing area by taking additional samples halfway between the passing test and the failing test. This procedure will be continued until the limits of the failing area are defined by passing test results.

Sampling or testing that requires penetration of in-place barrier soils will require the contractor to patch or fill the sample hole. The method of filling or patching of these holes shall be approved by the Soils QARE/CQA. The Soils QARE/CQA shall document such repairs.

4.6.3 Documentation. All observations, results of field tests performed on-site or off-site and laboratory tests, shall be recorded on suitable data record sheets. At a minimum, the inspection data record sheets shall include the following:

1. Description or title of the inspection activity.
2. Location of the inspection activity or location from which the sample was obtained.
3. Type of tests done or to be done.
4. Recorded observation or test date.
5. Results of the inspection or testing with reference to specifications.
6. Person involved in the inspection.
7. Signature of the CQA resident engineer and review by the CQA Project Manager.

Data record sheets to be utilized in the CQA program shall be approved by the CQA Project Manager.

5.0 GEOMEMBRANES

5.1 Quality Control Documentation

Prior to the installation of any geomembrane material, the Manufacturer or Installer shall provide the Project Manager/RPR with the following information:

1. The origin (resin supplier's name and resin production plant), identification (brand name and number), and production date of the resin.

2. Copies of the quality control certificates issued by the resin supplier.

3. Reports on tests conducted by the Manufacturer to verify that the quality of the resin used to manufacture the geomembrane meets the specifications.

4. Reports on quality control tests conducted by the Manufacturer to verify that the geomembrane manufactured for the project meets the project specifications.

5. A statement indicating that the amount of reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness and does not exceed 2 percent by weight.

6. A list of the materials which comprise the geomembrane, expressed in the following categories as percent by weight: polyethylene, carbon black, other additives.

7. A specification for the geomembrane which includes all properties contained in the specifications measured using the appropriate test methods.

8. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.

9. Quality control certificates, signed by a responsible party employed by the Manufacturer. Each quality control certificate shall include roll identification numbers, sampling procedures, and results of quality control tests. At a minimum, results shall be given for those properties stated in Section 02771 Part 1.05.C of the project specifications:

   These quality control tests shall be performed in accordance with the test methods and frequency as specified in GRI GM 13, and Specification Section 02771 Part 1.05.C.

The Manufacturer shall identify all rolls of geomembranes with the following:

1. Manufacturer's name
2. Date of manufacture
3. Resin batch code
4. Product identification
5. Thickness
6. Roll number
7. Roll dimensions

5.1.1 Product Review: The Project Manager/RPR shall verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.

2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.

4. Roll packages are appropriately labeled.

5. Certified minimum properties meet the specifications.

5.2 Conformance Testing

Upon delivery of the rolls of geomembrane, the Geosynthetic QARE shall assure that conformance test samples are obtained for the geomembrane. These samples shall then be forwarded to the Geosynthetic QAL for testing to assure conformance to the specifications.

If the Project Manager/RPR desires, the Geosynthetic QARE or his agent, can perform the third-party conformance test sampling at the manufacturing plant. This may be advantageous in expediting the installation process for very large projects.

The conformance tests shall be performed in accordance with the test methods specified in Section 02771 Part 1.06 of the project specifications and consistent with GRI Test Method GM17.

5.2.1 Sampling Procedures: The rolls to be sampled shall be selected by the Geosynthetic QARE. Samples shall be taken across the entire width of the roll and shall not include the first 3 feet (1 m). Unless otherwise specified, samples shall be 3 feet (1 m) long by the roll width. The Geosynthetic QARE shall mark the machine direction on the samples with an arrow.

A lot consists of a group of materials which is manufactured from a specific batch of raw materials (e.g., HDPE resin, or bentonite clay). The manufacturer shall identify the consecutively numbered rolls of material, which are inclusive within a lot.

Unless otherwise specified, samples shall be obtained at one sample per 50,000 ft^2 (4,650 m^2) of geomembrane. Conformance tests shall be done on the samples and the results of tensile strength properties in the cross machine direction shall be utilized to establish the parent material strength.

5.2.2 Test Results. All conformance test results shall be reviewed and accepted or rejected by the Geosynthetic QARE prior to the deployment of the geomembrane.

The Geosynthetic QARE shall examine all results from laboratory conformance testing and shall report any nonconformance to the Project Manager/RPR. The Geosynthetic QARE shall be responsible for checking that all test results meet or exceed the property values listed in the project specifications.

If the Manufacturer has reason to believe that failing tests may be the result of the Geosynthetic QAL incorrectly conducting the tests, the Manufacturer may request that the sample in question be retested by the Geosynthetic QAL with a technical representative of the Manufacturer present during the testing. This retesting shall be done at the expense of the Manufacturer. Alternatively, the Manufacturer may have the sample retested at two different approved independent laboratories at the expense of the Manufacturer. If both laboratories produce passing results, the material shall be accepted. If both laboratories do not produce passing results, then the original Geosynthetic QAL test results shall be accepted. The use of these procedures for dealing with failed test results is subject to the approval of the Project Manager/RPR.

If a test result is in nonconformance, all material from the lot represented by the failing test should be considered out of specification and rejected. Alternatively, at the option of the Project Manager/RPR, additional conformance test samples may be taken to "bracket" the portion of the lot not meeting specification (note that this procedure is valid only when all rolls in the lot are consecutively produced and numbered from one manufacturing line). To isolate the out of specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If the two additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) shall be rejected. If one or both of the additional tests fail, then the entire lot shall be rejected or the procedure repeated with two additional tests that bracket a greater number of rolls within the lot.

5.3 Subgrade Preparation

5.3.1 Surface Preparation: The earthwork contractor shall be responsible for preparing the supporting soil for placement of the geosynthetic layers. The Geomembrane shall be deployed on top of the Geosynthetic Clay Liner (GCL)

As indicated in Section 5.3.1.1 below the Installer shall certify in writing that the sub-surface on which the geomembrane will be installed is acceptable. A certificate of acceptance shall be given by the Installer to the Geosynthetic QARE prior to commencement of geomembrane deployment in the area under consideration. The Project Manager/RPR shall be given a copy of this certificate by the Geosynthetic QARE.

After the supporting soil has been accepted by the Installer, it is the Installer's responsibility to indicate to the Project Manager/RPR any change in the supporting soil condition that may require repair work. The Project Manager/RPR may consult with the Geosynthetic QARE regarding the need for repairs. If the Geosynthetic QARE concurs with Installer, the Project Manager/RPR shall assure that the supporting soil is repaired.

At any time before or during the geomembrane installation, the Geosynthetic QARE shall indicate to the Project Manager/RPR any locations which may not be adequately prepared for the geomembrane.

5.3.1.1 Clay Liner Surface Preparation

The earthwork contractor shall be responsible for preparing the supporting clay liner soils for placement of the secondary geomembrane.

Before the geomembrane installation begins, the Geosynthetic CQA shall verify that:

1. All lines and grades are verified.

2. An Owner's Representative verify that the supporting soil meets the density specified in the project specifications.

3. The surface to be lined has been rolled, compacted, or handworked so as to be free of irregularities, protrusions, loose soil, and abrupt changes in grade.

4. The surface of the supporting soil does not contain stones which may be damaging to the geosynthetics.

5. There is no area excessively softened by high water content.

The Installer shall certify in writing that the surface on which the geomembrane will be installed is acceptable. A certificate of acceptance shall be given by the Installer to the Geosynthetic CQA prior to commencement of geomembrane deployment in the area under consideration. The Project Manager shall be given a copy of this certificate by the Geosynthetic CQA.

After the supporting soil has been accepted by the Installer, it is the Installer's responsibility to indicate to the Project Manager any change in the supporting soil condition that may require repair work. The Project Manager may consult with the Geosynthetic CQA regarding the need for repairs. If the Geosynthetic CQA concurs with Installer, the Project Manager shall assure that the supporting soil is repaired.

At any time before or during the geomembrane installation, the Geosynthetic CQA shall indicate to the Project Manager any locations which may not be adequately prepared for the geomembrane.

5.3.1.2 GCL Surface Preparation

The geosynthetic installer shall be responsible for preparing the GCL for installation of the overlying primary geomembrane.

Before the geomembrane installation begins, the Geosynthetic CQA shall verify that:

1. All GCL seams have been completed in accordance with the specifications.

2. Repairs to the GCL are complete and secure.

3. The GCL surface is free of large wrinkles and folds and lies flat to the underlying surface.

4. GCL panels have been identified and as-built information collected.

The Installer shall verbally verify with the Geosynthetic CQA that the surface on which the geomembrane will be installed is acceptable. At any time before or during the geomembrane installation, the Geosynthetic CQA shall indicate to the Installer any locations which may not be adequately prepared for the geomembrane. The Installer shall repair said areas to the satisfaction of the Geosynthetic CQA.

5.3.1.3 Gas Collection Sand Surface Preparation (N.I.T.C)

The earthwork contractor shall be responsible for preparing the supporting soil for placement of the secondary sideslope geomembrane.

Before the geomembrane installation begins, the Geosynthetic CQA shall verify that:

1. All lines and grades are verified.

2. An Owner's Representative verify that the supporting soil meets all the gas collection sand requirements specified in the project specifications.

3. The surface to be lined has been machine-graded, or handworked so as to be free of irregularities, protrusions, loose soil, and abrupt changes in grade.

4. The surface of the supporting soil does not contain stones which may be damaging to the geomembrane.

5. There is no area excessively softened by high water content.

The Installer shall certify in writing that the surface on which the geomembrane will be installed is acceptable. A certificate of acceptance shall be given by the Installer to the Geosynthetic CQA prior to commencement of geomembrane deployment in the area under consideration. The Project Manager shall be given a copy of this certificate by the Geosynthetic CQA.

After the supporting soil has been accepted by the Installer, it is the Installer's responsibility to indicate to the Project Manager any change in the supporting soil condition that may require repair work. The Project Manager may consult with the Geosynthetic CQA regarding the need for repairs. If the Geosynthetic CQA concurs with Installer, the Project Manager shall assure that the supporting soil is repaired.

At any time before or during the geomembrane installation, the Geosynthetic CQA shall indicate to the Project Manager any locations which may not be adequately prepared for the geomembrane.

5.3.1.4 Geocomposite Surface Preparation

The geosynthetic installer shall be responsible for preparing the geocomposite surface for installation of the overlying primary geomembrane. The primary geomembrane shall be deployed on top of the geosynthetic clay liner (GCL).

Before the geomembrane installation begins, the Geosynthetic CQA shall verify that:

1. All geocomposite seams have been completed in accordance to the project specifications.

2. Repairs to the geocomposite are complete and secure.

3. The surface of the geocomposite is free of large wrinkles and folds, and lies flat to the underlying secondary geomembrane.

The Installer shall verbally verify with the Geosynthetic CQA that the surface on which the geomembrane will be installed is acceptable.

At any time before or during the geomembrane installation, the Geosynthetic CQA shall indicate to the Installer any locations which may not be adequately prepared for the geomembrane. The Installer shall repair said areas to the satisfaction of the Geosynthetic CQA.

5.3.2 Anchor Trench: The Geosynthetic QARE shall verify that the anchor trench has been constructed according to the design drawings and specifications.

If the anchor trench is excavated in a clay material susceptible to desiccation, the amount of trench open at any time should be minimized. The Geosynthetic QARE shall inform the Project Manager/RPR of any signs of significant desiccation associated with the anchor trench construction.

Slightly rounded corners shall be provided in the trench so as to avoid sharp bends in the geomembrane. Excessive amounts of loose soil shall not be allowed to underlie the geomembrane in the anchor trench.

The anchor trench shall be adequately drained to prevent ponding or softening of the adjacent soils while the trench is open. The anchor trench shall be backfilled and compacted as outlined in the project specifications.

Care shall be taken when backfilling the trenches to prevent any damage to the geosynthetics. The Geosynthetic QARE shall observe the backfilling operation and advise the Project Manager/RPR of any problems. Any problems shall be documented by the Geosynthetic QARE in his daily report.

5.4 Geomembrane Deployment

5.4.1 Panel Nomenclature: A field panel is defined as a unit of geomembrane which is to be seamed in the field, i.e., a field panel is a roll or a portion of roll cut in the field.

It shall be the responsibility of the Geosynthetic QARE to assure that each field panel is given an identification code (number or letter-number) consistent with the layout plan. This identification code shall be agreed upon by the Project Manager/RPR, Installer and Geosynthetic QARE. This field panel identification code shall be as simple and logical as possible. In general, it is not appropriate to identify panels using roll numbers since roll numbers established in the manufacturing plant are usually cumbersome and are not related to location in the field. The Geosynthetic QARE shall establish a table or chart showing correspondence between roll numbers and field panel identification codes. The field panel identification code shall be used for all quality assurance records.

The Geosynthetic QARE shall verify that field panels are installed at the locations indicated on the Installer's layout plan, as approved by the Project Manager/RPR.

5.4.2 Panel Deployment Procedure: The Geosynthetic QARE shall review the panel deployment progress of the Installer (keeping in mind issues relating to wind, rain, geosynthetic clay liner hydration, and other site-specific conditions) and advise the Project Manager/RPR on its compliance with the approved panel layout drawing and its suitability to the actual field conditions. Once approved, only the Project Manager/RPR can authorize changes to the panel deployment procedure. The Geosynthetic QARE shall verify that the condition of the supporting soil does not change detrimentally during installation.

The Geosynthetic QARE shall record the identification code, location, and date of installation of each field panel.

5.4.3 Deployment Weather Conditions: Geomembrane deployment shall not proceed at an ambient temperature below 32^{o}F (0^{o}C) or above 104^{o}F (40^{o}C) unless otherwise authorized, in writing, by the Project Manager/RPR. Geomembrane placement shall not be performed during any precipitation, in the presence of excessive moisture (e.g., fog, dew), in an area of ponded water, or in the presence of excessive winds. Geomembrane deployment shall not be undertaken if weather conditions will preclude material seaming following deployment.

The Geosynthetic QARE shall verify that the above conditions are fulfilled. Ambient temperature shall be measured by the Geosynthetic QARE in the area in which the panels are to be deployed. The Geosynthetic QARE shall inform the Project Manager/RPR of any weather related problems which may not allow geomembrane placement to proceed.

5.4.4 Method of Deployment: Before the geomembrane is handled on site, the Geosynthetic QARE shall verify that handling equipment to be used on the site is adequate and does not pose risk of damage to the geomembrane or underlying geosynthetic clay liner. During handling, the Geosynthetic QARE shall observe and verify that the Installer's personnel handle the geomembrane and geosynthetic clay liner with care.

The Geosynthetic QARE shall verify the following:

1. Any equipment used does not damage the geomembrane or geosynthetic clay liner by handling, trafficking, excessive heat, leakage of hydrocarbons, or other means.

2. The prepared surface underlying the geomembrane or geosynthetic clay liner has not deteriorated since previous acceptance, and is still acceptable immediately prior to geomembrane placement.

3. Any geosynthetic elements immediately underlying the geomembrane or geosynthetic clay liner are clean and free of debris.

4. All personnel do not smoke or wear damaging shoes while working on the geomembrane or geosynthetic clay liner, or engage in other activities which could damage the geomembrane or geosynthetic clay liner.

5. The method used to unroll the panels does not cause excessive scratches or crimps in the geomembrane and does not damage the supporting soil or geosynthetic clay liner.

6. The geomembrane or geosynthetic clay liner is not dragged over surfaces that may damage the panels. Rub sheets shall be used to prevent damage to geosynthetic materials if dragging is necessary.

7. The method used to place the panels minimizes wrinkles (especially differential wrinkles between adjacent panels).

8. Adequate temporary loading and/or anchoring (e.g., sand bags, tires), not likely to damage the geomembrane, has been placed to prevent uplift by wind. In case of high winds, continuous loading, e.g., by sand bags, is recommended along edges of panels to minimize risk of wind flow under the panels.

9. Direct contact with the geomembrane is minimized, and the geomembrane is protected by geotextiles, extra geomembrane, or other suitable materials, in areas where excessive traffic may be expected.

10. The method used to place the panels does not result in bridging.

11. That adequate field trial seams be run on seams that connect new geomembrane with existing geomembrane.

The Geosynthetic QARE shall inform the Project Manager/RPR if the above conditions are not fulfilled.

5.4.5 Damage and Defects: Upon delivery to the site, the Geosynthetic QARE shall conduct a surface observation of all rolls for defects and for damage. This inspection shall be conducted without unrolling rolls unless defects or damages are found or suspected. The Geosynthetic QARE shall advise the Project Manager/RPR, in writing, of any rolls or portions of rolls which should be rejected and removed from the site because they have severe flaws, and/or minor repairable flaws.

The Geosynthetic QARE shall inspect each panel, after placement and prior to seaming, for damage and/or defects. The Geosynthetic QARE shall advise the Project Manager/RPR which panels, or portions of panels, should be rejected, repaired, or accepted. Damaged panels, or portions of damaged panels, which have been rejected shall be marked and their removal from the work area recorded by the Geosynthetic QARE. Repairs shall be made using procedures described in Section 5.8.

5.4.6 Writing on the Liner: To avoid confusion, the Installer and the Geosynthetic QARE shall each use different colored markers that are readily visible for writing on the geomembrane. The markers used must be semi-permanent and compatible with the geomembrane.

5.5 Field Seaming

5.5.1 Seam Layout: Before installation begins, the Installer must provide the Project Manager/RPR and the Geosynthetic QARE with a panel layout drawing, i.e., a drawing of the facility to be lined showing all expected seams. The Geosynthetic QARE shall review the panel layout drawing and verify that it is consistent with accepted state-of-practice. No panels may be seamed without the written approval of the panel layout drawing by the Project Manager/RPR. In addition, panels not specifically shown on the panel layout drawing may not be used without the Project Manager/RPR's prior approval.

In general, seams should be oriented parallel to the line of maximum slope, i.e., oriented along, not across, the slope. In corners and odd-shaped geometric locations, the number of seams should be minimized. No horizontal seam should be less than 5 feet (1.5 m) from the toe of the slope, or areas of potential stress concentrations, unless otherwise authorized by the Project Manager/RPR. No horizontal seam shall be located on the sideslopes

A seam numbering system compatible with the panel numbering system shall be used by the Geosynthetic QARE.

5.5.2 Accepted Seaming Methods: Approved processes for field seaming are extrusion welding and fusion welding. Proposed alternate processes shall be documented and submitted by the Installer to the Project Manager/RPR for approval. Only apparatus which have been specifically approved by make and model shall be used. The Project Manager/RPR shall submit all documentation regarding seaming methods to be used to the Geosynthetic QARE for review.

5.5.2.1 Extrusion Process

The Geosynthetic QARE shall log ambient, seaming apparatus, and geomembrane surface temperatures at appropriate intervals and report any noncompliances to the Project Manager/RPR.

The Geosynthetic QARE shall verify that:

1. The Installer maintains on-site the number of spare operable seaming apparatus decided upon at the pre-construction meeting.

2. Equipment used for seaming is not likely to damage the geomembrane.

3. Prior to beginning a seam, the extruder is purged until all heat-degraded extrudate has been removed from the barrel.

4. Clean and dry welding rods or extrudate pellets are used.

5. The electric generator is placed on a smooth base such that no damage occurs to the geomembrane.

6. Grinding shall be completed no more than 1 hour prior to seaming.

7. A smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage.

8. The geomembrane is protected from damage in heavily trafficked areas.

9. Exposed grinding marks adjacent to an extrusion weld shall be minimized. In no instance shall exposed grinding marks extend more than 1/4 inch from the seamed area.

10. In general, the geomembrane panels are aligned to have a nominal overlap of 3 inch (75 mm) for extrusion welding. In any event, the final overlap shall be sufficient to allow peel tests to be performed on the seam.

11. No solvent or adhesive is used unless the product is approved in writing by the Project Manager/RPR prior to use (samples shall be submitted to the Project Manager/RPR for testing and evaluation).

12. The procedure used to temporarily bond adjacent panels together does not damage the geomembrane; in particular, the temperature of hot air at the nozzle of any temporary welding apparatus is controlled such that the geomembrane is not damaged or degraded.

5.5.2.2 Fusion Process

The Geosynthetic QARE shall log ambient, seaming apparatus, and geomembrane surface temperatures at appropriate intervals and report any noncompliances to the Project Manager/RPR.

The Geosynthetic QARE shall also verify that:

1. The Installer maintains on-site the number of spare operable seaming apparatus decided upon at the pre-construction meeting.

2. Equipment used for seaming is not likely to damage the geomembrane.

3. For cross seams, the edge of the cross seam is ground to an incline prior to welding.

4. The electric generator is placed on a smooth base such that no damage occurs to the geomembrane.

5. A smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage.

6. The geomembrane is protected from damage in heavily trafficked areas.

7. A movable protective layer is used as required by the Installer directly below each overlap of geomembrane that is to be seamed to prevent buildup of moisture between the sheets and prevent debris from collecting around the pressure rollers.

8. In general, the geomembrane panels are aligned to have a nominal overlap of 6 inches (152 mm) for fusion welding. In any event, the final overlap shall be sufficient to allow peel tests to be performed on the seam.

9. No solvent or adhesive is used unless the product is approved in writing by the Project Manager/RPR prior to use (samples shall be submitted to the Project Manager/RPR for testing and evaluation).

5.5.3 Seam Preparation: The Geosynthetic QARE shall verify that prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris or foreign material of any kind. If seam overlap grinding is required, the Geosynthetic QARE must assure that the process is completed according to the Manufacturer's instructions within one hour of the seaming operation, and in a way that does not damage the geomembrane. The Geosynthetic QARE shall also verify that seams are aligned with the fewest possible number of wrinkles and "fishmouths".

Seaming operations to existing liner systems shall be done with care. Existing liner system shall be prepared by pressure wash to assure clean surface. Trial seams shall be done on all seams connecting new liner with existing liner systems.

5.5.4 Trial Seams: Trial seams shall be made on fragment pieces of geomembrane liner used on the project to verify that conditions are adequate for production seaming. Such trial seams shall be made at the beginning of each seaming period, or at least once each four hours, for each production seaming apparatus used that day. Also, an additional trial seam is required after any change of operator, equipment shutdown, significant idle time, or significant weather change. Additional trial seams due to idle time or weather are at the discretion of the Geosynthetic QARE. Each seamer shall make at least one trial seam each day. Trial seams shall be made under the same conditions as actual seams.

Trial seams shall be made on liner that will be used in production seaming, such as: new textured liner to new textured liner; new textured liner to existing textured liner; smooth liner to textured liner.

The trial seam shall be at least 4 feet (1.2 m) long by 1 foot (0.3 m) wide (after seaming) with the seam centered lengthwise. Seam overlap shall be as indicated in Section 5.5.2.

Five samples shall be cut from the trial seam with a 1 inch (25 mm) wide die. The samples shall be cut by the Installer at locations selected randomly along the trial seam. The samples shall be tested in peel and shear using a field tensiometer. The tensiometer shall be capable of maintaining a constant jaw separation rate of two inches per minute. If a sample fails, the entire operation shall be repeated. If the additional sample fails, the seaming apparatus and seamer shall not be accepted and shall not be used for seaming until the deficiencies are corrected and two consecutive successful trial seams are achieved. The Geosynthetic QARE shall observe all trial seam procedures.

5.5.5 General Seaming Procedures: During general seaming, the Geosynthetic QARE shall be cognizant of the following:

1. For fusion welding, it may be necessary to place a movable protective layer of plastic directly below each overlap of geomembrane that is to be seamed. This is to prevent any moisture buildup between the sheets to be welded and prevent debris from collecting around the pressure rollers.

2. If required, a firm substrate shall be provided by using a flat board, a conveyor belt, or similar hard surface directly under the seam overlap to achieve proper support.

3. Fishmouths or wrinkles at the seam overlaps shall be cut along the ridge of the wrinkle in order to achieve a flat overlap. The cut fishmouths or wrinkles shall be seamed and any portion where the overlap is inadequate shall then be patched with an oval or round patch of the same geomembrane extending a minimum of 6 inches (152 mm) beyond the cut in all directions.

4. If seaming operations are carried out at night, adequate illumination shall be provided.

5. Seaming shall extend to the outside edge of panels placed in the anchor trench.

6. All cross seam tees should be extrusion welded to a minimum distance of 4 inches on each side of the tee.

7. No field seaming shall take place without the Master Seamer being present.

The Geosynthetic QARE shall verify that the above seaming procedures (or any other procedures agreed upon and indicated in the project manual) are followed, and shall inform the Project Manager/RPR of any nonconformance.

5.5.6 Seaming Weather Conditions:

5.5.6.1 Normal Weather Conditions

The normal required weather conditions for seaming are as follows:

1. Ambient temperature between 32^{o}F (0^{o}C) and 104^{o}F (40^{o}C).

2. Dry conditions, i.e., no precipitation or other excessive moisture, such as fog or dew.

3. No excessive winds.

The Geosynthetic QARE shall verify that these weather conditions are fulfilled and notify the Project Manager/RPR in writing if they are not. Ambient temperature shall be measured by the Geosynthetic QARE in the area in which the panels are to be placed. The Project Manager/RPR will then decide if the installation is to be stopped or special procedures used.

5.5.6.2 Cold Weather Conditions

To assure a quality installation, if seaming is conducted when the ambient temperature is below 32^{o}F (0^{o}C), the following conditions must be met:

1. Geomembrane surface temperatures shall be determined by the Geosynthetic QARE at intervals of at least once per 100 foot of seam length to determine if preheating is required. For extrusion welding, preheating is required if the surface temperature of the geomembrane is below 32^{o}F (0^{o}C).

2. Preheating may be waived by the Project Manager/RPR based on a recommendation from the Geosynthetic QARE, if the Installer demonstrates to the Geosynthetic QARE's satisfaction that welds of equivalent quality may be obtained without preheating at the expected temperature of installation.

3. If preheating is required, the Geosynthetic QARE shall inspect all areas of geomembrane that have been preheated by a hot air device prior to seaming, to assure that they have not been overheated.

4. Care shall be taken to confirm that the surface temperatures are not lowered below the minimum surface temperatures specified for welding due to winds or other adverse conditions. It may be necessary to provide wind protection for the seam area.

5. All preheating devices shall be approved prior to use by the Project Manager/RPR.

6. Additional destructive tests (as described in Section 5.7) shall be taken at an interval between 500 feet and 250 feet of seam length, at the discretion of the Geosynthetic QARE.

7. Sheet grinding may be performed before preheating, if applicable.

8. Trial seaming, as described in Section 5.5.4, shall be conducted under the same ambient temperature and preheating conditions as the actual seams. Under cold weather conditions, new trial seams shall be conducted if the ambient temperature drops by more than 5°F from the initial trial seam test conditions.

9. All snow and ice shall be removed from the liner using plastic shovels. The Geosynthetic QARE will also have authority to suspend installation activities during severe weather conditions.

5.5.6.3 Warm Weather Conditions

At ambient temperatures above 104°F, no seaming of the geomembrane shall be permitted unless the Installer can demonstrate to the satisfaction of the Project Manager/RPR that geomembrane seam quality is not compromised.

Trial seaming, as described in Section 5.5.4, shall be conducted under the same ambient temperature conditions as the actual seams.

At the option of the Geosynthetic QARE, additional destructive tests (as described in Section 5.4) may be required for any suspect areas.

5.6 Nondestructive Seam Testing

5.6.1 Concept. The Installer shall nondestructively test field seams over their full length using a vacuum test unit, air pressure test (for double fusion seams only), or other approved method. Vacuum testing and air pressure testing are described in Sections 5.6.2 and 5.6.3, respectively. The purpose of nondestructive tests is to check the continuity of seams. It does not provide quantitative information on seam strength. Nondestructive testing shall be carried out as the seaming work progresses, not at the completion of field seaming. At a minimum of once every 4 hours of seaming and when operation has been suspended for greater than one hour or if breakdown of seaming equipment occurs, weld samples will be tested in peel and shear in accordance with the destructive test requirements of the contract documents.

For all seams, the Geosynthetic QARE shall:

1. Observe nondestructive testing procedures.

2. Record location, data, test unit number, name of tester, and outcome of all testing.

3. Inform the Installer and Project Manager/RPR of any required repairs.

5.6.2 Vacuum Testing. The following procedures are applicable to vacuum testing.

1. The equipment shall consist of the following:

    a. A vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, a porthole or valve assembly, and a vacuum gauge.

    b. A pump assembly equipped with a pressure controller and pipe connections.

    c. A rubber pressure/vacuum hose with fittings and connections.

    d. A sudsy solution of soap and water.

    e. A bucket and wide paint brush, or other means of applying the sudsy solution.

2. The following procedures shall be followed:

    a. Energize the vacuum pump and reduce the tank pressure to approximately 3.0 to 8.0 psi gauge.

    b. Wet a strip of geomembrane approximately 12 inches by 48 inches (0.3 m x 1.2 m) with the soapy distilled water solution.

    c. Place the box over the wetted area.

    d. Close the bleed valve and open the vacuum valve.

    e. Assure that a leak-tight seal is created.

f. For a period of not less than 15 seconds, apply vacuum and examine the geomembrane through the viewing window for the presence of soap bubbles.

g. If no bubble appears after 15 seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3 inches (76 mm) overlap, and repeat the process.

h. All areas where soap bubbles appear shall be marked and repaired in accordance with Section 5.8.

<u>5.6.3 Air Pressure Testing</u>. The following procedures are applicable to double fusion welding which produces a double seam with an enclosed space.

1. The equipment shall consist of the following:

    a. An air pump (manual or motor driven), equipped with pressure gauge capable of generating and sustaining a pressure between 30 and 35 psi (206 and 241 kPa) and mounted on a cushion to protect the geomembrane.

    b. A rubber hose with fittings and connections.

    c. A sharp hollow needle, or other approved pressure feed device.

2. The following procedures shall be followed:

    a. Seal both ends of the seam to be tested.

    b. Insert needle or other approved pressure feed device into the air channel created by the fusion weld.

    c. Insert a protective cushion between the air pump and the geomembrane.

    d. Energize the air pump to a pressure between 30 and 35 psi (206 and 241 kPa), close valve, allow 2 minutes for pressure to stabilize, and sustain pressure for at least 5 minutes.

    e. If loss of pressure exceeds 2 psi (15 kPa) or does not stabilize, locate faulty area and repair in accordance with Section 5.8.

    f. Cut opposite end of tested seam area once testing is completed to verify continuity of the air channel. If air does not escape, locate blockage and retest unpressurized area. Seal the cut end of the air channel.

    g. Remove needle or other approved pressure feed device and seal.

<u>5.6.4 Test Failure Procedures</u>. The Installer shall complete any required repairs in accordance with Section 5.8. For repairs, the Geosynthetic QARE shall:

1. Observe the repair and testing of the repair.

2. Mark on the geomembrane that the repair has been made.

3. Document the repair procedures and test results.

5.7 Destructive Seam Testing

5.7.1 Concept. Destructive seam tests shall be performed at selected locations. The purpose of these tests is to evaluate seam strength. Seam strength testing shall be done as the seaming work progresses, not at the completion of all field seaming.

5.7.2 Location and Frequency. The Geosynthetic QARE shall select locations where seam samples will be cut out for laboratory testing. Those locations shall be established as follows:

1. A minimum frequency of one test location per 1,000 feet of seam length performed by each welder. This minimum frequency is to be determined as an average taken throughout the entire facility.

2. Test locations shall be determined during seaming at the Geosynthetic QARE's discretion. Selection of such locations may be prompted by suspicion of overheating, contamination, offset welds, or any other potential cause of imperfect welding.

The Installer shall not be informed in advance of the locations where the seam samples will be taken.

5.7.3 Sampling Procedures. Samples shall be cut by the Installer at locations chosen by the Geosynthetic QARE as the seaming progresses so that laboratory test results are available before the geomembrane is covered by another material. The Geosynthetic QARE shall:

1. Observe sample cutting.

2. Assign a number to each sample, and mark it accordingly.

3. Record sample location on layout drawing.

4. Record reason for taking the sample at this location (e.g., statistical routine, suspicious feature of the geomembrane).

All holes in the geomembrane resulting from destructive seam sampling shall be immediately repaired in accordance with repair procedures described in Section 5.8. The continuity of the new seams in the repaired area shall be tested according to the project specifications.

5.7.4 Sample Dimensions. At a given sampling location, one sample will be taken by the installer with minimum dimensions of seam width by 48 inches in length. Field seam testing shall be performed in accordance with Section 02771 on a total of five coupons. Two coupons shall be cut one end and three from the opposite end using a 1-inch wide die with the seam centered parallel to the width of the coupon. If all coupons pass the field seam testing requirements as specified in Section 02771, a sample for laboratory testing is cut from the sample. The sample is cut into three parts and distributed as follows.

1. One portion to the Installer for optional laboratory testing, 12 inches x 12 inches (0.3 m x 0.3 m).

2. One portion for Geosynthetic QAL testing, 12 inches x 18 inches (0.3 m x 0.5 m) and

3. One portion to the Project Manager/RPR for archive storage, 12 inches x 12 inches (0.3 m x 0.3 m).

Final determination of the sample sizes shall be made at the pre-construction meeting.

<u>5.7.5 Field Testing</u>. The five 1 inch (25 mm) wide coupons mentioned in Section 5.7.4 shall be tested in the field using a tensiometer and shall not fail according to the criteria in the project specifications. The tensiometer shall be capable of maintaining a constant jaw separation rate of two inches per minute. If the test passes in accordance with this section, the sample qualifies for testing in the laboratory. If it fails, the seam should be repaired in accordance with Section 5.8. Final judgment regarding seam acceptability, based on the failure criteria, rests with the Geosynthetic QARE. Both tracks are to be tested in peel.

The Geosynthetic QARE shall witness all field tests and mark all samples and portions with their number. The Geosynthetic QARE shall also log the date and time, ambient temperature, number of seaming unit, name of seamer, welding apparatus temperatures and pressures, and pass or fail description, and attach a copy to each sample portion.

<u>5.7.6 Laboratory Testing</u>. Destructive test samples shall be packaged and shipped, if necessary, under the responsibility of the Geosynthetic QAL in a manner which will not damage the test sample. The Project Manager/RPR will be responsible for storing the archive samples. Test samples shall be tested by the Geosynthetic QAL.

Testing shall include properties as defined in the specifications. The minimum acceptable values to be obtained in these tests are indicated in the specifications. At least 5 specimens shall be tested in each shear and peel. Specimens shall be selected alternately by test from the samples (i.e. peel, shear, peel, shear...).

The Geosynthetic QAL shall provide verbal test results no more than 24 hours after they receive the samples. The Geosynthetic QARE shall review laboratory test results as soon as they become available, and make appropriate recommendations to the Project Manager/RPR.

<u>5.7.7 Destructive Test Failure Procedures</u>. The following procedures shall apply whenever a sample fails a destructive test, whether that test is conducted by the Geosynthetic QAL, or by field tensiometer. The Installer has two options:

1. The Installer can repair the seam between any two passing test locations.

2. The Installer can trace the welding path to an intermediate location (at 10 feet (3 m) minimum from the point of the failed test in each direction) and take a sample with a 1 inch (25 mm) wide die for an additional field test at each location. If these additional samples pass the test, then full laboratory samples are taken. If these laboratory samples pass the tests, then the seam is repaired between these locations. If either sample fails, then the process is repeated to establish the zone in which the seam should be repaired.

All acceptable repaired seams shall be bound by two locations from which samples passing laboratory destructive tests have been taken. Passing laboratory destructive tests of trial seam samples taken as indicated in Section 5.5.4 may be used as a boundary for the failing seam. In cases exceeding 150 feet (50 m) of repaired seam, a sample taken from the zone in which the seam has been repaired must pass destructive testing. Repairs shall be made in accordance with Section 5.8.

The Geosynthetic QARE shall document all actions taken in conjunction with destructive test failures.

5.8 Defects and Repairs

5.8.1 Identification. All seams and non-seam areas of the geomembrane shall be examined by the Geosynthetic QARE for identification of defects, holes, blisters, undispersed raw materials, and any sign of contamination by foreign matter. Because light reflected by the geomembrane helps to detect defects, the surface of the geomembrane shall be clean at the time of examination. The geomembrane surface shall be cleaned by the Installer if the amount of dust or mud inhibits examination.

5.8.2 Evaluation. Each suspect location both in seam and non-seam areas shall be nondestructively tested using the methods described in Section 5.6 as appropriate. Each location which fails the nondestructive testing shall be marked by the Geosynthetic QARE and repaired by the Installer. Work shall not proceed with any materials which will cover locations which have been repaired until appropriate nondestructive and laboratory test results with passing values are available.

5.8.3 Repair Procedures. Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, shall be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure shall be agreed upon between the Project Manager/RPR, Installer, and Geosynthetic QARE.

1. The repair procedures available include:

    a. Patching, used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter.

    b. Spot welding or seaming, used to repair small tears, pinholes, or other minor, localized flaws.

    c. Capping, used to repair large lengths of failed seams at the discretion of the Geosynthetic CQA.

    d. Removing bad seam and replacing with a strip of new material welded into place.

2. For any repair method, the following provisions shall be satisfied:

    a. Surfaces of the geomembrane which are to be repaired using extrusion methods shall be abraded no more than one hour prior to the repair. The extrudate shall cover all the grinding area.

    b. All surfaces shall be clean and dry at the time of the repair.

    c. All seaming equipment used in repairing procedures shall meet the requirements of the QAP.

d. Patches or caps shall extend at least 6 inches (150 mm) beyond the edge of the defect, and all corners of patches shall be rounded with a radius of approximately 3 inches (75 mm).

5.8.4 Repair Verification. Each repair shall be numbered and logged. Each repair shall be nondestructively tested using the methods described in Section 5.6 as appropriate. Repairs which pass the nondestructive test shall be taken as an indication of an adequate repair. Repairs more than 150 feet long may be of sufficient extent to require destructive test sampling, at the discretion of the Geosynthetic QARE. Failed tests indicate that the repair shall be redone and retested until a passing test results. The Geosynthetic QARE shall observe all nondestructive testing of repairs and shall record the number of each repair, date, and test outcome.

5.8.5 Large Wrinkles. When seaming of the geomembrane is completed, and prior to placing overlying materials, the Geosynthetic QARE shall indicate to the Project Manager/RPR which wrinkles should be cut and reseamed by the Installer. The number of wrinkles to be repaired should be kept to an absolute minimum. Therefore, wrinkles should be located during the coldest part of the installation process, while keeping in mind the forecasted weather to which the uncovered geomembrane may be exposed. Wrinkles are considered to be large when the geomembrane can be folded over onto itself. This is generally the case for a wrinkle that extends 12 inches from the subgrade. Seams produced while repairing wrinkles shall be tested as outlined above.

When placing overlying material on the geomembrane, every effort must be made to minimize wrinkle development. If possible, cover should be placed during the coolest weather available. In addition, small wrinkles should be isolated and covered as quickly as possible to prevent their growth. The placement of cover materials shall be observed by the Geosynthetic QARE to assure that wrinkle formation is minimized.

5.9 Geomembrane Protection

The quality assurance procedures indicated in this Section are intended only to assure that the installation of adjacent materials does not damage the geomembrane.

5.9.1 Soils. A copy of the specifications prepared by the Designer for placement of soils shall be given to the Geosynthetic QARE by the Project Manager/RPR. The Geosynthetic QARE shall verify that these specifications are consistent with the state-of-practice such as:

1. Placement of soils on the geomembrane shall not proceed at an ambient temperature below 32^{o}F (0^{o}C) nor above 104^{o}F (40^{o}C) unless otherwise specified.

2. Placement of soil on the geomembrane should be done during the coolest part of the day to minimize the development of wrinkles in the geomembrane.

3. A geotextile or other cushion approved by the Designer is generally required between aggregate and the geomembrane.

4. Equipment used for placing soil shall not be driven directly on the geomembrane.

5. A minimum thickness of 1 foot (0.3 m) of soil is specified between a light dozer (ground pressure of 5 psi (35 kPa) or lighter) and the geomembrane.

6. In any areas traversed by any vehicles other than low ground pressure vehicles approved by the Project Manager/RPR, the soil layer shall have a minimum thickness of 3 feet (0.9 m). This requirement may be waived if provisions are made to protect the geomembrane through an engineered design. Drivers shall proceed with caution when on the overlying soil and prevent spinning of tires or sharp turns.

The Geosynthetic QARE shall measure soil thickness and verify that the required thicknesses are present. The Geosynthetic QARE must also verify that final thicknesses are consistent with the design and verify that placement of the soil is done in such a manner that geomembrane damage is unlikely. The Geosynthetic QARE shall inform the Project Manager/RPR if the above conditions are not fulfilled.

<u>5.9.2 Sumps and Appurtenances</u>. A copy of the plans and specifications prepared by the Designer for appurtenances shall be given by the Project Manager/RPR to the Geosynthetic QARE. The Geosynthetic QARE shall review these plans and verify that:

1. Installation of the geomembrane in appurtenant areas, and connection of geomembrane to appurtenances have been made according to specifications.

2. Extreme care is taken while welding around appurtenances since neither non-destructive nor destructive testing may be feasible in these areas.

3. The geomembrane has not been visibly damaged while making connections to appurtenances.

The Geosynthetic QARE shall inform the Project Manager/RPR in writing if the above conditions are not fulfilled.

<u>5.10 Leak Location Survey:</u>

The work shall be performed by a qualified independent testing firm that has performed surveys during the previous three years of at least 10,000,000 square feet of geomembrane covered by earth materials. This shall include at least three large scale projects consisting of at least two surveys of more than 875,000 square feet and one survey of 1,000,000 square feet. Leak Location Services, Inc. of San Antonio, Texas (Daren L. Laine, contact) is approved and is a qualified provider of these services.

<u>5.10.1 Preparation:</u> The following must be complete prior to the leak location service arriving on site.

- Install electrodes, if required, under the GCL prior to deployment
- Uniformly wet the leachate collection sand with water to field capacity.
- Expose or leave exposed, the geomembrane located at the top of the anchor trench or along tie-in seams with adjacent cells for the duration of the leak survey

<u>5.10.2 Performance</u>. The leak location equipment will be tested/calibrated to document the leak detection sensitivity. A 0.25-inch diameter hole will be used as an artificial leak to test equipment sensitivity. Once the leachate collection sand has been placed a hole shall be dug through the sand and a hole made in the drainage geocomposite. Then a drill or other suitable instrument shall be used to remove rather than displace the material (geomembrane) in the hole location. The geocomposite shall be replaced and the location marked and/or located relative to site benchmarks.

6.0 GEOTEXTILES

6.1 Quality Control Documentation

Prior to the installation of any geotextile, the Manufacturer or Installer shall provide the Project Manager/RPR with the following information:

1. The origin (resin supplier's name and resin production plant) and identification (brand name and number) of the resin used to manufacture the geotextile.

2. Copies of the quality control certificates issued by the resin supplier.

3. Reports on tests conducted by the Manufacturer to verify that the quality of the resin used to manufacture the geotextile meets the Manufacturer's resin specifications.

4. Reports on quality control tests conducted by the Manufacturer to verify that the geotextile manufactured for the project meets the project specifications.

5. A statement indicating that the reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness.

6. A list of the materials which comprise the geotextile, expressed in the following categories as percent by weight: base polymer, carbon black, other additives.

7. A specification for the geotextile which includes all properties contained in the project specifications measured using the appropriate test methods.

8. Written certification that minimum average roll values given in the specification are guaranteed by the Manufacturer.

9. For non-woven geotextiles, written certification that the Manufacturer has continuously inspected the geotextile for the presence of needles and found the geotextile to be needle free.

10. Quality control certificates, signed by a responsible party employed by the Manufacturer. The quality control certificates shall include roll identification numbers, sampling procedures and results of quality control tests. At a minimum, results shall be given for those properties stated in Section 02272 Part 1.05(C) of the project specifications.

    Quality control tests shall be performed in accordance with the test methods specified in the project specifications for at least every 100,000 ft^2 of geotextile produced.

The Manufacturer shall identify all rolls of geotextiles with the following:

1. Manufacturer's name
2. Product identification
3. Roll number
4. Roll dimensions

6.1.1 Product Review: The Project Manager/RPR shall verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.

2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.

4. Roll packages are appropriately labeled.

5. Certified minimum average roll properties meet the project specifications.

6.2 Conformance Testing

Upon delivery of the rolls of geotextiles, the Geosynthetic QARE shall assure that conformance test samples are obtained for the geotextile. These samples shall then be forwarded to the Geosynthetic QAL for testing to assure conformance to the project specifications.

If the Project Manager/RPR desires, the Geosynthetic QARE or his agent, can perform the conformance test sampling at the manufacturing plant. This may be advantageous in expediting the installation process for very large projects.

The conformance tests shall be performed in accordance with the test methods indicated in Section 02272 Part 2.01(B) of the project specifications. Other conformance tests may be required by the Project Manager/RPR.

6.2.1 Sampling Procedures: The rolls to be sampled shall be selected by the Geosynthetic QARE. Samples shall be taken across the entire width of the roll and shall not include the first complete revolution of fabric on the roll. Samples shall not be taken from any portion of a roll which has been subjected to excess pressure or stretching. Unless otherwise specified, samples shall be 3 ft (1 m) long by the roll width. The Geosynthetic QARE shall mark the machine direction on the samples with an arrow. All lots of material and the particular test sample that represents each lot should be defined before the samples are taken.

A lot shall be defined as a group of consecutively numbered rolls from the same manufacturing line. Alternatively, a lot may be designated by the Geosynthetic QARE based on a review of all roll information including quality control documentation and manufacturing records.

Unless otherwise specified, samples shall be taken at a rate of one per lot, not to exceed one conformance test per 100,000 ft^2 (10,000 m^2) of geotextile.

6.2.2 Test Results. All conformance test results shall be reviewed and accepted or rejected by the Geosynthetic QARE prior to the deployment of the geotextile.

The Geosynthetic QARE shall be responsible for checking that all test results meet or exceed the property values listed in the project specifications.

If the Manufacturer has reason to believe that failing tests may be the result of the Geosynthetic QAL incorrectly conducting the tests, the Manufacturer may request that the sample in question be retested by the Geosynthetic QAL with a technical representative of the Manufacturer present during the testing. This retesting shall be done at the expense of the Manufacturer. Alternatively, the Manufacturer may have the sample retested at two different approved independent laboratories at the expense of the Manufacturer. If both laboratories produce passing results, the material shall be accepted. If both laboratories do not produce passing results, then the original Geosynthetic QAL test results shall be accepted. The use of these procedures for dealing with failed test results is subject to the approval of the Project Manager/RPR.

If a test result is in nonconformance, all material from the lot represented by the failing test should be considered out of specification and rejected. Alternatively, at the option of the Project Manager/RPR, additional conformance test samples may be taken to "bracket" the portion of the lot not meeting specification (note that this procedure is valid only when all rolls in the lot are consecutively produced and numbered from one manufacturing line). To isolate the out of specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) shall be rejected. If one or both of the additional tests fail, then the entire lot shall be rejected or the procedure repeated with two additional tests that bracket a greater number of rolls within the lot.

6.3 Geotextile Deployment

During shipment and storage, the geotextile shall be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions. Geotextile rolls shall be shipped and stored in relatively opaque and watertight wrappings. Wrappings shall be removed shortly before deployment.

The Geosynthetic QARE shall observe rolls upon delivery at the site and any deviation from the above requirements shall be reported to the Project Manager/RPR.

The Installer shall handle all geotextiles in such a manner as to assure they are not damaged in any way, and the following shall be complied with:

1. On slopes, the geotextiles shall be securely anchored and then rolled down the slope in such a manner as to continually keep the geotextile sheet in tension.

2. In the presence of wind, all geotextiles shall be weighted with sandbags or the equivalent. Such sandbags shall be installed during deployment and shall remain until replaced with cover material.

3. Geotextiles shall be cut using a geotextile cutter (hook blade) only. If in place, special care shall be taken to protect other materials from damage which could be caused by the cutting of the geotextiles.

4. The Installer shall take any necessary precautions to prevent damage to underlying layers during placement of the geotextile.

5. During placement of geotextiles, care shall be taken not to entrap, in or beneath the geotextile, stones, excessive dust, or moisture that could damage the geomembrane, cause clogging of drains or filters, or hamper subsequent seaming.

6. A visual examination of the geotextile shall be carried out over the entire surface, after installation, to assure that no potentially harmful foreign objects, such as needles, are present.

The Geosynthetic QAL shall note any noncompliance and report it to the Project Manager/RPR.

6.4 Seaming Procedures

On slopes steeper than 10 horizontal:1 vertical, all geotextiles shall be continuously sewn (i.e., spot sewing is not allowed). Geotextiles shall be overlapped a minimum of 3 inches (75 mm) prior to seaming. In general, no horizontal seams shall be allowed on sideslopes (i.e., seams shall be along, not across, the slope), except as part of a patch.

On bottoms and slopes shallower than 10 (horizontal):1 (vertical), geotextiles shall be seamed as indicated above (preferred), or continuously thermally bonded with the written approval of the Project Manager/RPR.

The Installer shall pay particular attention at seams to assure that no earth cover material could be inadvertently inserted beneath the geotextile.

Any sewing shall be done using polymeric thread with chemical and ultraviolet light resistance properties equal to or exceeding those of the geotextile. Sewing shall be done using machinery and stitch types specified in the project specifications or as approved in writing by the Project Manager/RPR.

6.5 Defects and Repairs

Any holes or tears in the geotextile shall be repaired as follows:

On slopes, a patch made from the same geotextile shall be sewn into place in accordance with the project specifications. Should any tear exceed 10 percent of the width of the roll, that roll shall be removed from the slope and replaced.

Care shall be taken to remove any soil or other material which may have penetrated the torn geotextile.

The Geosynthetic QARE shall observe any repair and report any noncompliance with the above requirements in writing to the Project Manager/RPR.

6.6 Geotextile Protection

All soil materials located on top of a geotextile shall be deployed in such a manner as to assure:

1. The geotextile and underlying lining materials are not damaged.

2. Minimal slippage of the geotextile on underlying layers occurs.

3. No excess tensile stresses occur in the geotextile.

Unless otherwise specified by the Geosynthetic QARE, all lifts of soil material shall be in conformance with the guidelines given in Section 4.9.1.

7.0 GEONETS (DRAINAGE GEOCOMPOSITE)

7.1 Quality Control Documentation

Prior to the installation of any geonet, the Manufacturer or Installer shall provide the Project Manager/RPR with the following information:

1. The origin (resin supplier's name and resin production plant), identification (brand name and number), and production date of the resin.

2. Copies of the quality control certificates issued by the resin supplier.

3. Reports on tests conducted by the Manufacturer to verify that the quality of the resin used to manufacture the geonet meets the specifications.

4. Reports on quality control tests conducted by the Manufacturer to verify that the geonet and drainage geocomposite manufactured for the project meets the project specifications.

5. A statement indicating that the amount of reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness and does not exceed 2 percent by weight.

6. A list of the materials which comprise the geonet, expressed in the following categories as percent by weight: polyethylene, carbon black, other additives.

7. A specification for the geonet which includes all properties contained in the specifications measured using the appropriate test methods.

8. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.

9. Quality control certificates, signed by a responsible party employed by the Manufacturer. The quality control certificates shall include roll identification numbers, sampling procedures and results of quality control tests. At a minimum, results shall be given for those properties stated in Section 02272 Part 1.05 of the project specifications.

   Quality control tests shall be performed in accordance with the test methods specified in the specifications, for every 100,000 ft^2 of geonet produced.

The Manufacturer shall identify all rolls of drainage geocomposite with the following:

1. Manufacturer's name
2. Product identification
3. Roll number
5. Roll dimensions

The Geosynthetic QARE shall review these documents and shall report any discrepancies with the above requirements to the Project Manager/RPR. The Geosynthetic QARE shall verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.

2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.

4. Roll packages are appropriately labeled.

5. Certified minimum properties meet the specifications.

7.2 Conformance Testing

Upon delivery of the rolls of geonet, the Geosynthetic QARE shall assure that conformance test samples are obtained for the geonet. These samples shall then be forwarded to the Geosynthetic QAL for testing to assure conformance to the specifications.

If the Project Manager/RPR desires, the Geosynthetic QARE or his agent, can perform the conformance test sampling at the manufacturing plant. This may be advantageous in expediting the installation process for very large projects.

The conformance tests shall be performed in accordance with the test methods indicated in Section 02272 Part 2.01(C)(5) of the project specifications. Other conformance tests may be required by the Project Manager/RPR.

7.2.1 Sampling Procedures: The rolls to be sampled shall be selected by the Geosynthetic QARE. Samples shall be taken across the entire width of the roll, and shall not be taken from any portion of a roll which has been subjected to excess pressure or stretching. Unless otherwise specified, samples shall be 3 ft (1 m) long by the roll width. The Geosynthetic QARE shall mark the machine direction on the samples with an arrow. All lots of material and the particular test sample that represents each lot should be defined before the samples are taken.

If the Project Manager/RPR desires, the Geosynthetic QARE or his agent or his agent, can perform the third-party conformance test sampling at the manufacturing plant. This may be advantageous in expediting the installation process for very large projects.

A lot consists of a group of materials which is manufactured from a specific batch of raw materials (e.g., HDPE resin, or bentonite clay). The manufacturer shall identify the consecutively numbered rolls of material, which are inclusive within a lot.

Unless otherwise specified, samples shall be taken at a rate of one per lot, not less than one conformance test per 100,000 ft^2 (10,000 m^2) of geonet.

7.2.2 Test Results. All conformance test results shall be reviewed and accepted or rejected by the Geosynthetic QARE prior to the deployment of the geotextile.

The Geosynthetic QARE shall be responsible for checking that all test results meet or exceed the property values listed in the project specifications.

If the Manufacturer has reason to believe that failing tests may be the result of the Geosynthetic QAL incorrectly conducting the tests, the Manufacturer may request that the sample in question be retested by the Geosynthetic QAL with a technical representative of the Manufacturer present during the testing. This retesting shall be done at the expense of the Manufacturer. Alternatively, the Manufacturer may have the sample retested at two different approved independent laboratories at the expense of the Manufacturer. If both laboratories produce passing results, the material shall be accepted. If both laboratories do not produce passing results, then the original Geosynthetic QAL test results shall be accepted. The use of these procedures for dealing with failed test results is subject to the approval of the Project Manager/RPR.

If a test result is in nonconformance, all material from the lot represented by the failing test should be considered out of specification and rejected. Alternatively, at the option of the Project Manager/RPR, additional conformance test samples may be taken to "bracket" the portion of the lot not meeting specification (note that this procedure is valid only when all rolls in the lot are consecutively produced and numbered from one manufacturing line). To isolate the out of specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) shall be rejected. If one or both of the additional tests fail, then the entire lot shall be rejected or the procedure repeated with two additional tests that bracket a greater number of rolls within the lot.

7.3 Geocomposite Deployment

During shipment and storage, the geocomposite shall be protected from inundation, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions.

The Geosynthetic QARE shall observe rolls upon delivery at the site and any deviation from the above requirements shall be reported to the Project Manager/RPR.

The Installer shall handle all geocomposite in such a manner as to assure they are not damaged in any way, and the following shall be complied with:

1. On slopes, the geocomposite shall be securely anchored and then rolled down the slope in such a manner as to continually keep the geocomposite sheet in tension.

2. In the presence of wind, all geocomposite shall be weighted with sandbags or the equivalent. Such sandbags shall be installed during deployment and shall remain until replaced with cover material.

3. Geocomposite shall be cut using a hook blade only. If in place, special care shall be taken to protect other materials from damage which could be caused by the cutting of the geocomposite.

4. The Installer shall take any necessary precautions to prevent damage to underlying layers during placement of the geocomposite.

5. During placement of geocomposite, care shall be taken not to entrap, in or beneath the geocomposite, stones, excessive dust, soil, or moisture that could damage the geomembrane, cause clogging of drains or filters, or hamper subsequent seaming.

6. A visual examination of the geocomposite shall be carried out over the entire surface, after installation, to assure that no potentially harmful foreign objects are present.

The Geosynthetic QAL shall note any noncompliance and report it to the Project Manager/RPR.

7.4 Seaming Procedures

Geonets shall be overlapped a minimum of 4 inches (75 mm) prior to tying. In general, no horizontal seams shall be allowed on sideslopes (i.e., seams shall be along, not across, the slope), except as part of a patch.

7.5 Seams and Overlaps

Adjacent geonet shall be joined according to construction drawings and specifications. At a minimum, the following requirements shall be met:

1. Adjacent rolls shall be overlapped by at least 4 inches (100 mm).

2. Overlaps shall be secured by tying.

3. Tying can be achieved by plastic fasteners or polymer braid. Tying devices shall be white or yellow for easy inspection. Metallic devices are not allowed.

4. Tying shall be every 5 feet (1.5 m) along the slope, every 6 inches (0.15 m) in the anchor trench, and every 6 inches (0.15 m) along end-to-end seams on the base of the landfill.

5. In general, no horizontal seams shall be allowed on sideslopes.

6. In the corners of the sideslopes of rectangular landfills, where overlaps between perpendicular geonet strips are required, an extra layer of geonets. shall be unrolled along the slope, on top of the previously installed geonet, from top to bottom of the slope.

7. When more than one layer of geonet is installed, joints shall be staggered.

8. The geotextiles that will have direct contact with soil shall be heat-sealed at geonet overlaps.

The Geosynthetic QARE shall note any noncompliance and report it to the Project Manager/RPR.

When several layers of geonet are stacked, care shall be taken to prevent strands of one layer from penetrating the channels of the next layer, thereby significantly reducing the transmissivity. This cannot happen if stacked geonet are placed in the same direction. A stacked geonet shall never be laid in perpendicular directions to the underlying geonet (unless otherwise specified by the Designer).

7.6 Defects and Repairs

Any holes or tears in the geocomposite shall be repaired by placing a patch extending 1 foot (0.3 m) beyond the edges of the hole or tear. The patch shall be secured to the original geonet by tying every 6 inches (0.15 m). Tying devices shall be as indicated in Section 7.5. If the hole or tear width across the roll is more than 50 percent of the width of the roll, the damaged area shall be repaired as follows:

1. On the base of the landfill, the damaged area shall be cut out and the two portions of the geocomposite shall be joined as indicated in Section 7.5.

2. On sideslopes, the damaged geocomposite shall be removed and replaced.

7.7 Geonet Protection

Soil should never be placed in direct contact with geocomposite. Soil materials near the geocomposite shall be placed in such a manner as to assure:

1. The geocomposite and underlying lining materials are not damaged.

2. Minimal slippage of the geocomposite on underlying layers occurs.

3. No excess tensile stresses occur in the geonet.

Unless otherwise specified by the Designer, all lifts of soil material shall be in conformance with the guidelines given in Section 4.0.

Any noncompliance shall be noted by the Geosynthetic QARE and reported to the Project Manager/RPR.

8.0 GEOSYNTHETIC CLAY LINERS (GCL)

8.1 Quality Control Documentation

Prior to the installation of any GCL, the Manufacturer or Installer shall provide the Project Manager/RPR with the following information:

1. The origin (bentonite and geotextile supplier's name and bentonite and geotextile production plant) and identification (brand name and number) of the bentonite and geotextile used to manufacture the GCL.

2. Copies of the quality control certificates issued by the bentonite and geotextile supplier.

3. Reports on tests conducted by the Manufacturer to verify that the quality of the bentonite and geotextile used to manufacture the GCL meets the Manufacturer's bentonite and geotextile specifications.

4. Reports on quality control tests conducted by the Manufacturer to verify that the GCL manufactured for the project meets the project specifications.

5. A specification for the GCL which includes all properties contained in the project specifications measured using the appropriate test methods.

6. Written certification that minimum average roll values given in the specification are guaranteed by the Manufacturer.

7. For non-woven geotextiles, written certification that the Manufacturer has continuously inspected the geotextile for the presence of needles and found the geotextile to be needle free.

8. Quality control certificates, signed by a responsible party employed by the Manufacturer. The quality control certificates shall include roll identification numbers, sampling procedures and results of quality control tests. At a minimum, results shall be given for those properties stated in Section 02275 Part 2.01(A) of the project specifications.

   Quality control tests shall be performed in accordance with the test methods specified in the project specifications and consistent with GRI-GCL3, Standard Specification for Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs).

The Manufacturer shall identify all rolls of GCLs with the following:

1. Manufacturer's name
2. Product identification
3. Roll number
4. Roll dimensions

8.1.1 Product Review: The Project Manager/RPR shall verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.

2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.

4. Roll packages are appropriately labeled.

5. Certified minimum average roll properties meet the project specifications.

8.2 Conformance Testing

Upon delivery of the rolls of GCLs, the Project Manager/RPR shall assure that conformance test samples are obtained for the GCL. Samples shall be obtained for conformance testing at a frequency consistent with GRI-GCL3 Standards, with a minimum of 1 per lot. These samples shall then be forwarded to an independent laboratory for testing to assure conformance to the project specifications.

If the Project Manager/RPR desires, the Geosynthetic QARE or his agent, can perform the conformance test sampling at the manufacturing plant. This may be advantageous in expediting the installation process for very large projects.

The conformance tests shall be performed in accordance with the test methods indicated in Section 02275 Part 1.05. Other conformance tests may be required by the Project Manager/RPR.

8.2.1 Sampling Procedures: The rolls to be sampled shall be selected by the Project Manager/RPR. Samples shall be taken across the entire width of the roll. Samples shall not be taken from any portion of a roll which has been subjected to excess pressure or stretching. Unless otherwise specified, samples shall be 3 ft (1 m) long by the roll width. All lots of material and the particular test sample that represents each lot should be defined before the samples are taken.

A lot consists of a group of materials which is manufactured from a specific batch of raw materials (e.g., HDPE resin, or bentonite clay). The manufacturer shall identify the consecutively numbered rolls of material, which are inclusive within a lot.

Unless otherwise specified, samples shall be taken at a rate of one per lot, not less than one conformance test per 50,000 ft^2 (4,645 m^2) of GCL.

8.2.2 Test Results. All conformance test results shall be reviewed and accepted or rejected by the Project Manager/RPR prior to the deployment of the GCL.

The Project Manager/RPR shall be responsible for checking that all test results meet or exceed the property values listed in the project specifications.

If the Manufacturer has reason to believe that failing tests may be the result of the independent laboratory incorrectly conducting the tests, the Manufacturer may request that the sample in question be retested by the independent laboratory with a technical representative of the Manufacturer present during the testing. This retesting shall be done at the expense of the Manufacturer. Alternatively, the Manufacturer may have the sample retested at two different approved independent laboratories at the expense of the Manufacturer. If both laboratories produce passing results, the material shall be accepted. If both laboratories do not produce passing results, then the original independent laboratory test results shall be accepted. The use of these procedures for dealing with failed test results is subject to the approval of the Project Manager/RPR.

If a test result is in nonconformance, all material from the lot represented by the failing test should be considered out of specification and rejected. Alternatively, at the option of the Project Manager/RPR, additional conformance test samples may be taken to "bracket" the portion of the lot not meeting specification (note that this procedure is valid only when all rolls in the lot are consecutively produced and numbered from one manufacturing line). To isolate the out of specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) shall be rejected. If one or both of the additional tests fail, then the entire lot shall be rejected or the procedure repeated with two additional tests that bracket a greater number of rolls within the lot.

8.3 Subgrade Preparation

8.3.1 Surface Preparation: The earthwork contractor shall be responsible for preparing the supporting soil for GCL placement.

Before the GCL installation begins, the Geosynthetic QARE shall verify that:

1. All lines and grades are verified.

2. An Owner's Representative verify that the supporting soil meets the density specified in the project specifications.

3. The surface to be lined has been rolled, compacted, or handworked so as to be free of irregularities, protrusions, loose soil, and abrupt changes in grade.

4. The surface of the supporting soil does not contain stones which may be damaging to the geosynthetics.

5. There is no area excessively softened by high water content.

The Installer shall certify in writing that the surface on which the GCL will be installed is acceptable. A certificate of acceptance shall be given by the Installer to the Geosynthetic QARE prior to commencement of GCL deployment in the area under consideration. The Project Manager/RPR shall be given a copy of this certificate by the Geosynthetic QARE.

After the supporting soil has been accepted by the Installer, it is the Installer's responsibility to indicate to the Project Manager/RPR any change in the supporting soil condition that may require repair work. The Project Manager/RPR may consult with the Geosynthetic QARE regarding the need for repairs. If the Geosynthetic QARE concurs with Installer, the Project Manager/RPR shall assure that the supporting soil is repaired.

At any time before or during the GCL installation, the Geosynthetic QARE shall indicate to the Project Manager/RPR any locations which may not be adequately prepared for the GCL.

8.4 GCL Deployment

During shipment and storage, the GCL shall be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions. GCL rolls shall be shipped and stored in relatively opaque and watertight wrappings. Wrappings shall be removed shortly before deployment.

The Geosynthetic QARE shall observe rolls upon delivery at the site and any deviation from the above requirements shall be reported to the Project Manager/RPR.

The Installer shall handle all GCLs in such a manner as to assure they are not damaged in any way, and the following shall be complied with:

1. On slopes, the GCLs shall be securely anchored and then rolled down the slope in such a manner as to continually keep the GCLs fully relaxed (but not wrinkled).

2. GCLs shall be cut using a utility knife or special (manufacturer) cutter only. If in place, special care shall be taken to protect other materials from damage which could be caused by the cutting of the GCLs.

3. The Installer shall take any necessary precautions to prevent damage to underlying layers during placement of the GCL.

4. A visual examination of the GCL shall be carried out over the entire surface, after installation, to assure that no potentially harmful foreign objects, such as needles, are present.

5. The Installer shall sign and submit subgrade surface acceptance certificates for each area to be covered by the GCL.

6. The Installer shall deploy no more GCL material than can be covered with geomembrane by the end of that working day; this shall be verified by the Project Manager/RPR.

7. The Installer shall use a core pipe to lift the GCL during deployment which does not bend excessively creating unacceptable tension in the GCL, this shall be verified by the Project Manager/RPR.

8. To prevent damage, the GCL panels should not be dragged along the subgrade surface.

8.5 Seaming Procedures

GCLs shall be overlapped a minimum of 6 inches prior to seaming. In general, no horizontal seams shall be allowed on sideslopes (i.e., seams shall be along, not across, the slope), except as part of a patch. The GCL shall be seamed with a minimum of ¼-pound (115 g/lf) of bentonite. Seams at the base of the slope shall be a minimum of 5 feet away from the toe of the slope.

The Geosynthetic QARE shall pay particular attention at seams to assure that no earth cover material could be inadvertently inserted beneath the GCL.

8.6 Defects and Repairs

Any holes or tears in the GCL shall be repaired as follows:

Place a bead of granular bentonite at the minimum rate of one-quarter pound per linear foot around the damaged area, cut a batch of new GCL to fit over the damaged area and extending a minimum of 1 foot beyond it, and carefully backfill.

Care shall be taken to remove any soil or other material which may have penetrated the damaged GCL.

8.7 GCL Protection

All soil materials located on top of a GCL shall be deployed in such a manner as to assure:

1. The GCL is not damaged.
2. Minimal slippage of the GCL on underlying layers occurs.
3. No excess tensile stresses occur in the GCL.

Unless otherwise specified by the Geosynthetic QARE, all lifts of soil material shall be in conformance with the guidelines given in Section 4.0.

8.8 Installation Documentation

The Project Manager/RPR shall prepare and submit the following information as part of the project documentation plan:

1. All conformance testing results.
2. All daily reports detailing the GCL deployment.
3. Subgrade surface acceptance certifications signed by the responsible parties.
4. A compilation of all CQA checklists completed during the installation.
5. All manufacturer's certifications and accompanying test data.
6. A description of deviations, if any, made to the original CQA plan during the installation.

7. The Geosynthetic QARE will accept the GCL prior to placement.

9.0 HDPE PIPE, FITTINGS AND APPURTENANCES

9.1 Quality Control Documentation

Prior to the installation of any pipe, fittings, and appurtenances at the landfill facilities, the contractor shall provide the Engineer with material submittals on the materials to be used in construction of the landfill facilities. Submittals shall be prepared according to the project specifications. The following information shall be available and will be provided to the Engineer or CQA upon request:

1. The origin (resin supplier's name and resin production plant), identification (brand name and number), and production date of the resin.

2. Copies of the quality control certificates issued by the resin supplier.

3. Reports on tests conducted by the Manufacturer to verify that the quality of the resin used to manufacture the HDPE pipe and fittings, meets the specifications.

4. Reports on quality control tests conducted by the Manufacturer to verify that the HDPE pipe and fittings manufactured for the project meets the project specifications.

5. A statement indicating that the amount of reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness and does not exceed 2 percent by weight.

6. A list of the materials which comprise the HDPE pipe and fittings, expressed in the following categories as percent by weight: polyethylene, carbon black, other additives.

7. A specification for the pipe and fittings which includes all properties contained in the specifications measured using the appropriate test methods.

8. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.

9. Quality control certificates, signed by a responsible party employed by the Manufacturer. The quality control certificates shall include pipe identification numbers, sampling procedures and results of quality control tests. At a minimum, results shall be given for:

   a. Density
   b. Melt flow index
   c. Carbon black content

   Quality control tests shall be performed in accordance with the test methods specified in the specifications, for every 1,000 ft of pipe produced.

The Manufacturer shall identify all pipe according to ASTM D 1248 and ASTM F 714.

The CQA shall review these documents, as requested, and shall report any discrepancies with the above requirements to the Project Manager/RPR. The CQA shall verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.

2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

3. Quality control certificates have been provided at the specified frequency for pipe produced, and that each certificate identifies the pipe related to it.

4. Pipe is appropriately labeled.

5. Certified minimum properties meet the specifications.

9.2 Conformance Testing

Upon delivery of the HDPE pipe, the CQA shall inspect the pipe and based on the pipe condition and review of the manufacturer's certification documentation, may elect to sample the pipe for conformance testing. Conformance test samples shall be identified in a manner appropriate for the ASTM standard for HDPE pipe. These samples shall then be forwarded to the QAL for testing to assure conformance to the specifications.

If elected, the following conformance tests shall be performed on the pipe:

1. Physical dimensions by ASTM D 2122
2. Density by ASTM D 1505
3. Plate bearing test by ASTM D 2412
4. Impact resistance by ASTM D 2444

These conformance tests shall be performed in accordance with the test methods specified. Other conformance tests may be required by the Project Manager/RPR.

9.2.1 Sampling Procedures: A lot shall be defined as a group of consecutively numbered pipes from the same manufacturing line. Alternatively, a lot may be designated by the CQA based on a review of all pipe information including quality control documentation and manufacturing records.

Unless otherwise specified, samples shall be taken at a rate of one per lot, not less than one conformance test per 1,000 ft of HDPE pipe.

9.2.2 Test Results. All conformance test results must be reviewed and accepted or rejected by the CQA prior to installation of the pipe.

The CQA shall examine all results from laboratory conformance testing and shall report any nonconformance to the Project Manager/RPR. The CQA shall be responsible for checking that all test results meet or exceed the minimum property values listed in project specifications.

If the Manufacturer has reason to believe that failing tests may be the result of the QAL incorrectly conducting the tests, the Manufacturer may request that the sample in question be retested by the QAL with a technical representative of the Manufacturer present during the testing. This retesting shall be done at the expense of the Manufacturer. Alternatively, the Manufacturer may have the sample retested at two different NEWSME Operations approved independent laboratories at the expense of the manufacturer. If both laboratories produce passing results, the material shall be accepted. If both laboratories do not produce passing results, then the original QAL's test results shall be accepted. The use of these procedures for dealing with failed test results is subject to the approval of the Project Manager/RPR.

If a test result is in nonconformance, all material from the lot represented by the failing test shall be considered out of specification and rejected. Alternatively, at the option of the Project Manager/RPR, additional conformance test samples may be taken to "bracket" the portion of the lot not meeting specification (note that this procedure is valid only when all pipes in the lot are consecutively produced and numbered from one manufacturing line). To isolate the out of specification material, additional samples must be taken from pipes that have pipe numbers immediately adjacent to the pipe that was sampled and failed. If the two additional tests pass, the pipe that represents the initial failed test and the pipe manufactured immediately after that pipe (next larger pipe number) shall be rejected. If one or both of the additional tests fail, then the entire lot shall be rejected or the procedure repeated with two additional tests that bracket a greater number of pipes within the lot.

9.3 Pipe and Fitting Placement

The Installer shall handle all pipe in such a manner as to ensure it is not damaged in any way, and the following shall be complied with:

1. HDPE pipe, fittings, and appurtenances shall be placed as shown in the Contract Drawings, specifications, and/or as directed by the CQA.

2. Piping placed within a trench shall be placed according to the contract drawings. Pipe bedding and backfill shall be done according to the specifications.

3. Pipe fittings shall be installed as shown on the contract drawings and as recommended by the pipe manufacturer and CQA.

4. Solid wall HDPE transport pipe and associated fittings shall be pressure leak tested according to general industry standards. The CQA is responsible for observing and documenting these tests. The installer is responsible for locating and repairing all leaks in such a manner that is acceptable to the CQA.

9.4 Pipe Seams and Fusion Techniques

Butt fusion welding shall be used to seam pipe lengths together and install pipe fittings. Welding shall be done by qualified personnel and according to manufacturer’s instructions. The installer shall provide the CQA with documentation of the pipe welder’s experience. The CQA shall observe and document welding operations as necessary to assure welding techniques are highest quality.

9.5 Repairs

Any damaged pipe shall be discarded. No repairs shall be allowed.

9.6 Manholes, Valves and Other Appurtenances

Manholes, valves and other appurtenances associated with the piping systems shall be constructed according to the contract drawings and specifications. All connections to piping and/or associated equipment shall be according to the contract drawings and the manufacturer's recommendations. All manholes, valves and other pipe connections shall be leak tested according to general industry standards. The installer is responsible for repair of faulty material and repairs required to provide a leakproof installation. The CQA is responsible for observing and documenting installation and testing necessary to assure installation techniques are highest quality.

9.7 Soil Materials Placement

All soil materials located on top of a pipe shall be placed in such a manner as to ensure:

1. The pipe and underlying materials are not damaged or dislocated.

2. The pipe structural integrity is not compromised.

Unless otherwise specified by the Designer, all lifts of soil material shall be in conformance with the guidelines given in Section 4.0.

Any noncompliance shall be noted by the CQA and reported to the Project Manager/RPR.

SME 6

VOLUME III APPENDIX F
SUPPLEMENTAL GEOTECHNICAL EVALUATIONS

- **SUPPLEMENTAL STABILITY EVALUATION FOR SECTION A-A’**
- **PEAK AND LARGE DISPLACEMENT LINER STRENGTH ENVELOPES**
- **SLOPE STABILITY SENSITIVITY ANALYSIS**

SUPPLEMENTAL STABILITY EVALUATION FOR SECTION A-A’

SUMMARY OF MINIMUM CALCULATED SLOPE STABILITY FACTORS OF SAFETY

Comparison of values in Table 3-9 of Volume III of the Application to 1H:1V Slope detail in Figures E-2 and E-3 of the Application

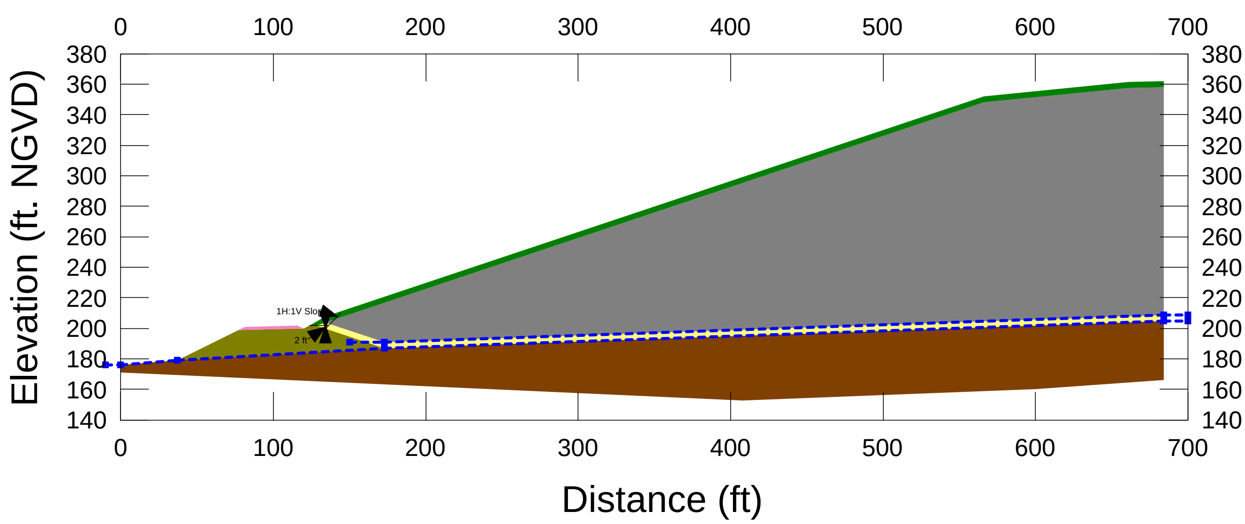
Construction and Operations				
		Static Factors of Safety		
Slip Surface Location	Surface Shape	MEDEP Required Minimum	A-A' in Application	A-A' with 1H:1V Toe
Waste	Shallow Surficial	1.3	1.91	1.88
Liner	Block		1.73	1.73
Foundation	Circular		2.65	2.65
		Seismic Factors of Safety		
Slip Surface Location	Surface Shape	MEDEP Required Minimum	A-A' in Application	A-A' with 1H:1V Toe
Waste	Shallow Surficial	1.1	1.54	1.51
Liner	Block		1.37	1.37
Foundation	Circular		2.14	2.14

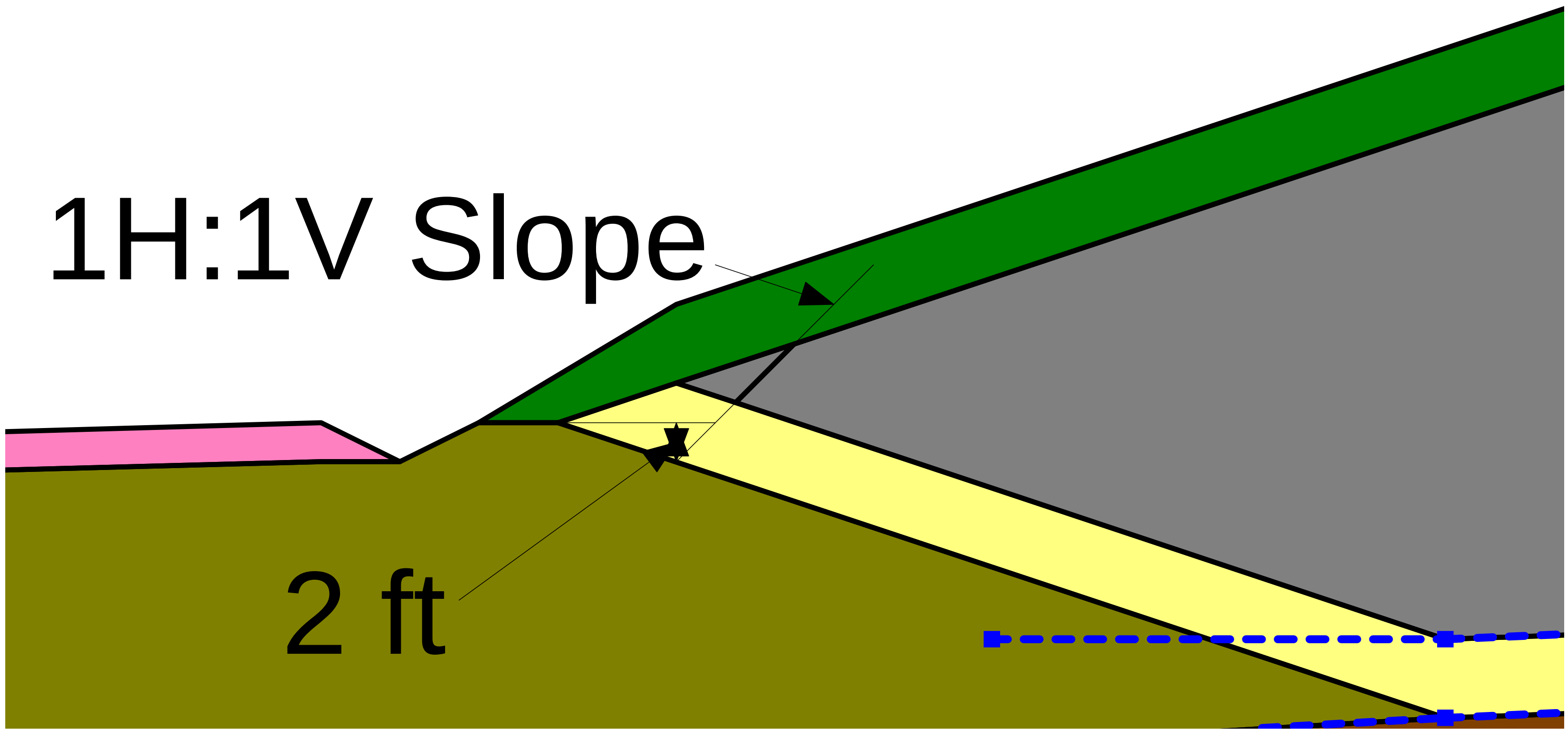
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: For Appendix F-1
Analysis Name: 0 Cross-Section A-A'
Date: 1/27/2016

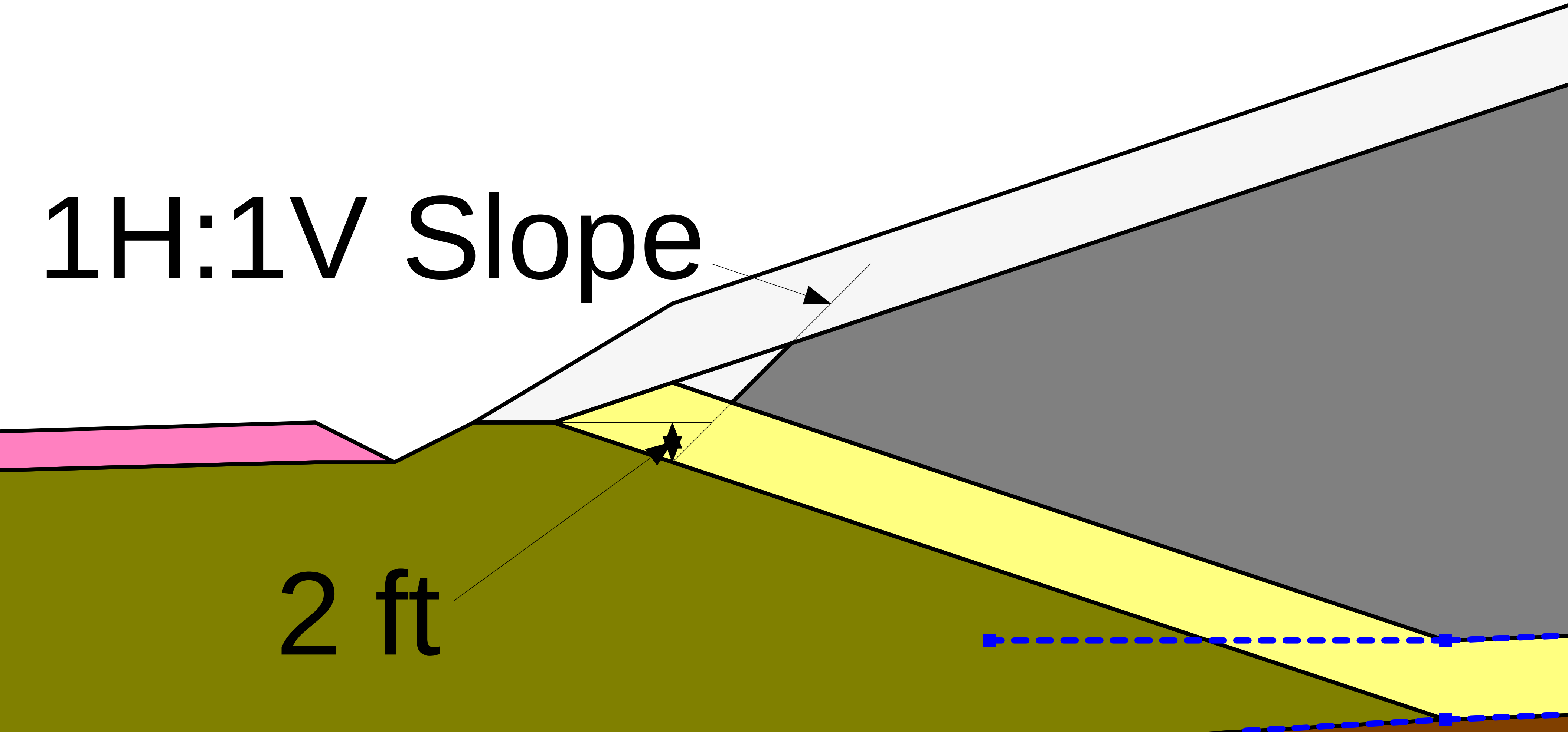
Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1
Name: Gravel Roads Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 0 Phi: 34 Phi-B: 0 Piezometric Line: 2
Name: Foundation Soils (Glacial Till) Model: Mohr-Coulomb Unit Weight: 132 Cohesion: 1000 Phi: 38 Phi-B: 0 Piezometric Line: 2
Name: Till Road Base Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 250 Phi: 34 Phi-B: 0 Piezometric Line: 2

Method: Bishop, Ordinary and Janbu
Analysis Name: 0 Cross-Section A-A'
Seismic Coefficient (ks): 0

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555




1H:1V Slope
2 ft

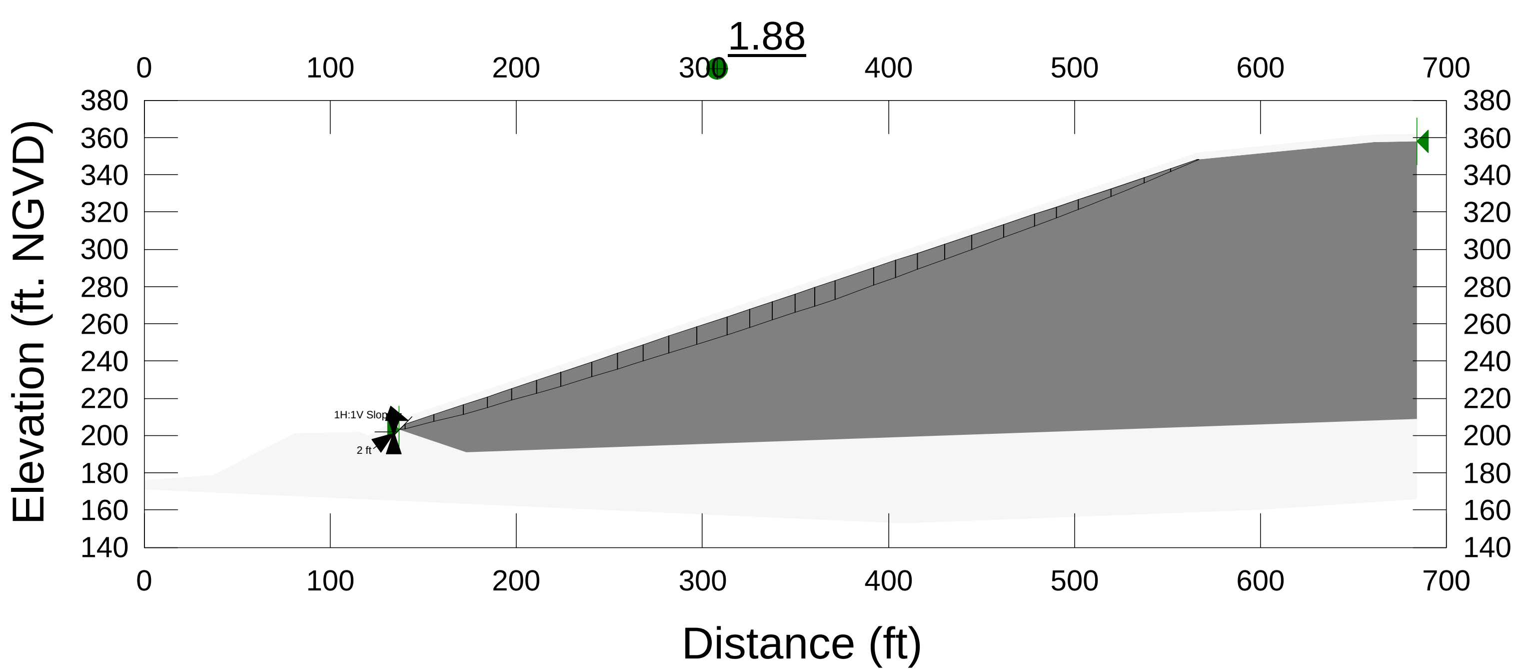

1H:1V Slope
2 ft

Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Waste Operational with the 1H:1V slope cut at toe
Analysis Name: 6b Waste Ops 1H:1V
Date: 1/27/2016

Minimum Factor of Safety = 1.88

Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0

Method: Spencer
Analysis Name: 6b Waste Ops 1H:1V
Seismic Coefficient (ks): 0



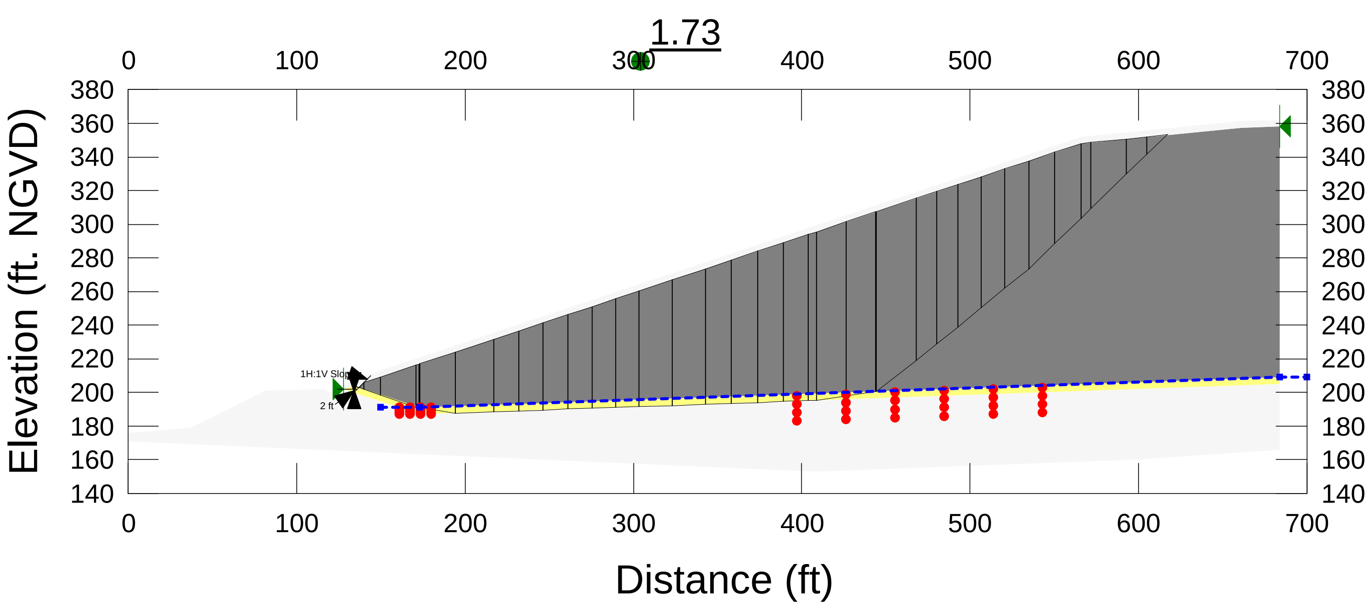
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Liner Operational
Analysis Name: 4b Liner Ops 1H:1V
Date: 1/27/2016

Minimum Factor of Safety = 1.73

Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1

Method: Spencer
Analysis Name: 4b Liner Ops 1H:1V
Seismic Coefficient (ks): 0

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555



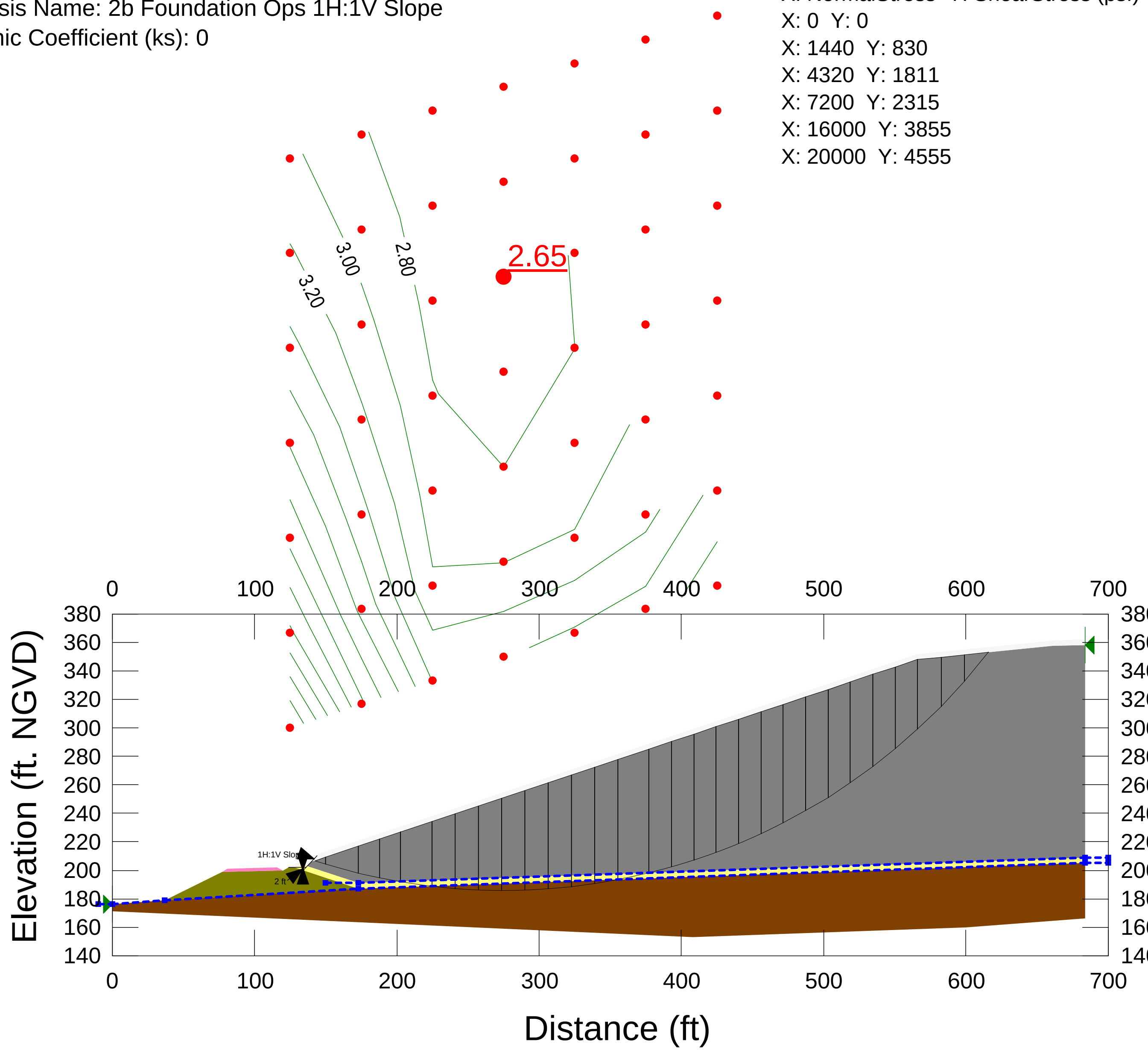
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Foundation Operational
Analysis Name: 2b Foundation Ops 1H:1V Slope
Date: 1/27/2016

Minimum Factor of Safety = 2.65

Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1
Name: Gravel Roads Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 0 Phi: 34 Phi-B: 0 Piezometric Line: 2
Name: Foundation Soils (Glacial Till) Model: Mohr-Coulomb Unit Weight: 132 Cohesion: 1000 Phi: 38 Phi-B: 0 Piezometric Line: 2
Name: Till Road Base Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 250 Phi: 34 Phi-B: 0 Piezometric Line: 2

Method: Bishop, Ordinary and Janbu
Analysis Name: 2b Foundation Ops 1H:1V Slope
Seismic Coefficient (ks): 0

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555

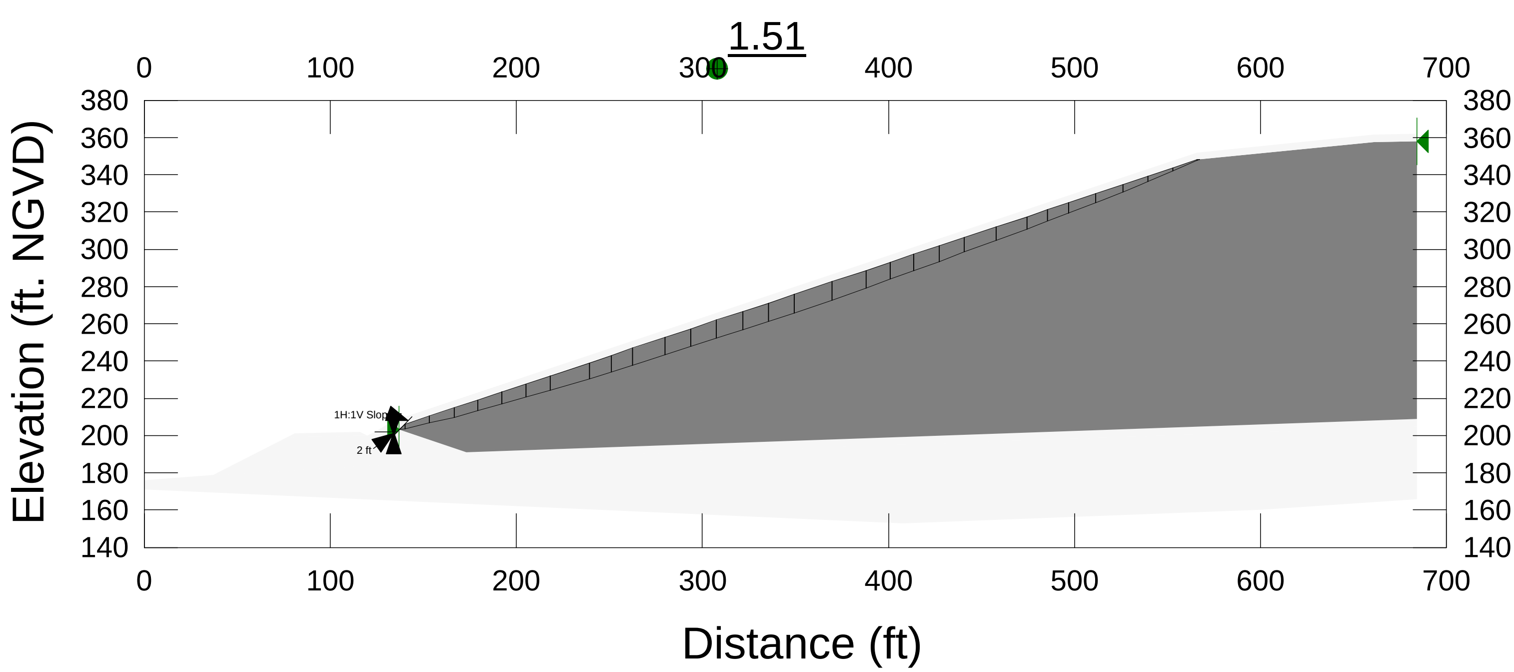


Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Waste Operational with the 1H:1V slope cut at toe, seismic
Analysis Name: 6bs Waste Ops 1H:1V s
Date: 1/27/2016

Minimum Factor of Safety = 1.51

Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0

Method: Spencer
Analysis Name: 6bs Waste Ops 1H:1V s
Seismic Coefficient (ks): 0.07



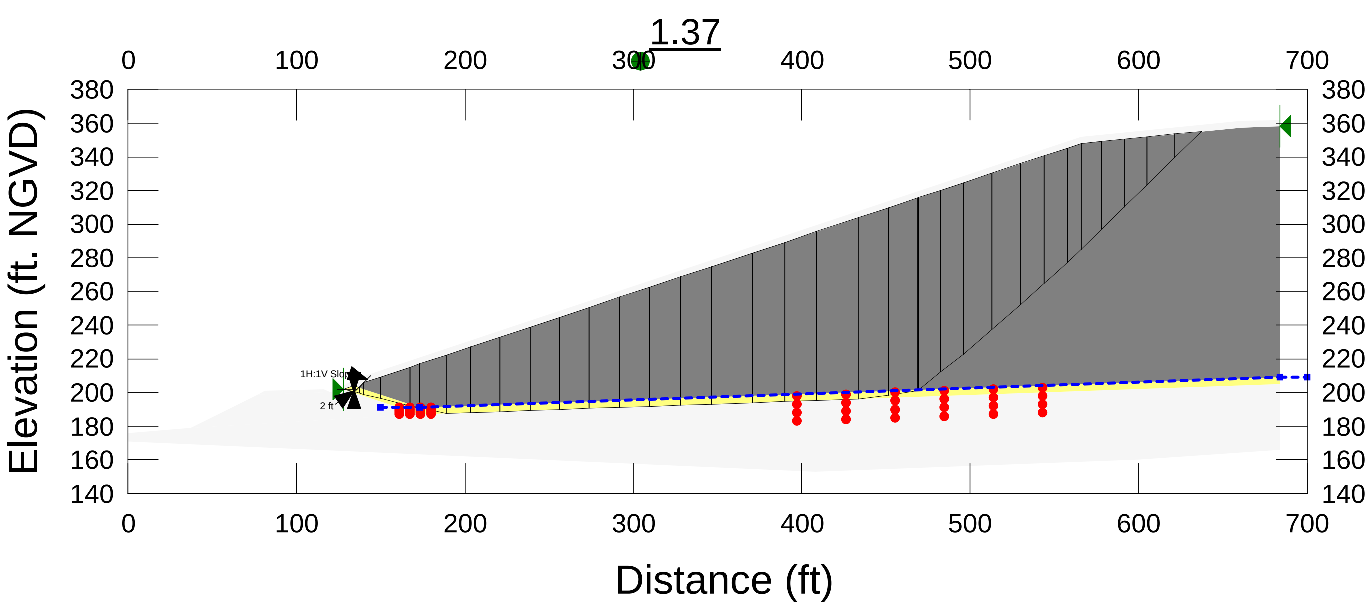
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Liner Operational
Analysis Name: 4b Liner Ops 1H:1V s
Date: 1/27/2016

Minimum Factor of Safety = 1.37

Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1

Method: Spencer
Analysis Name: 4b Liner Ops 1H:1V s
Seismic Coefficient (ks): 0.07

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555



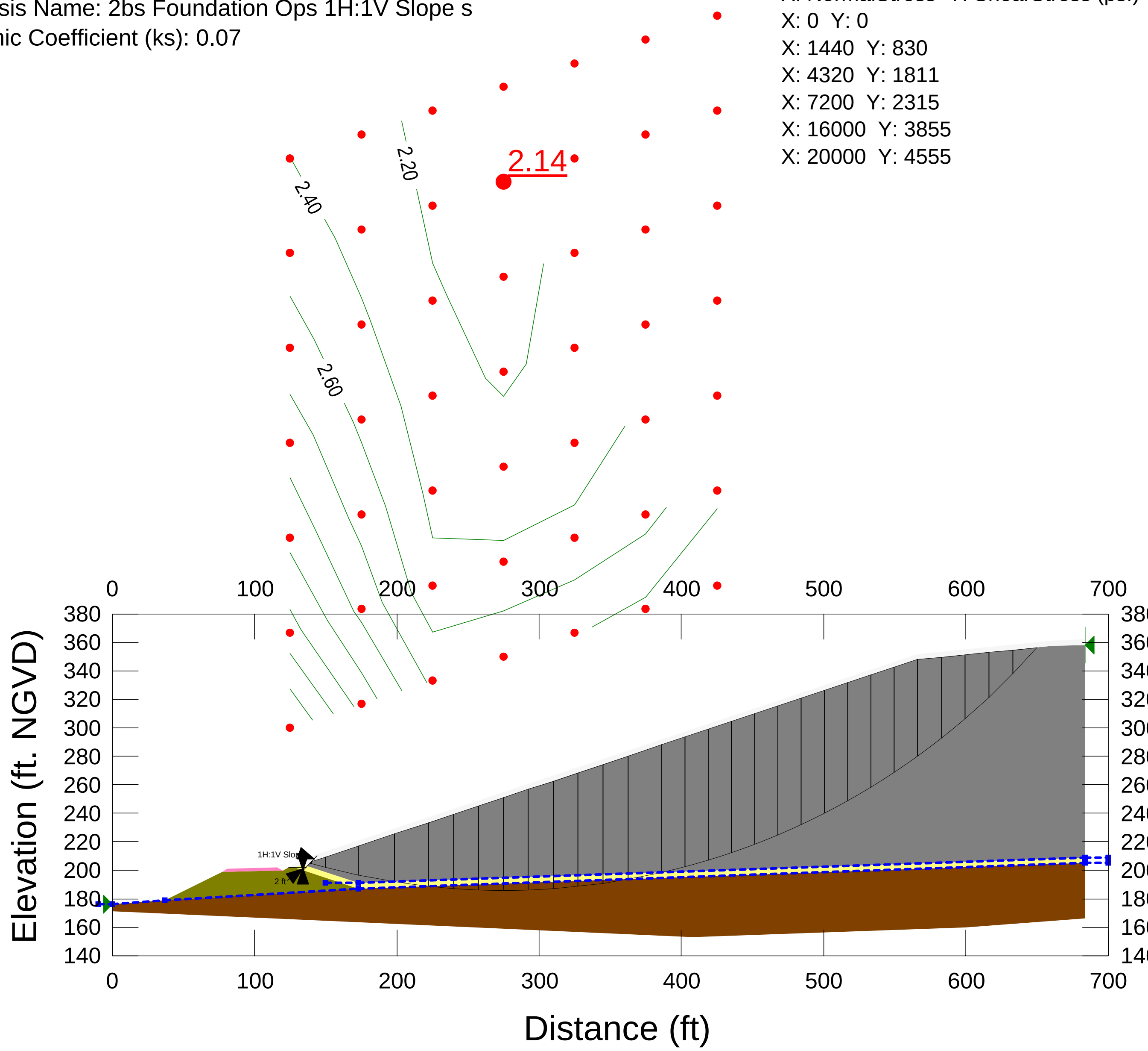
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\Stability\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Foundation Operational
Analysis Name: 2bs Foundation Ops 1H:1V Slope s
Date: 1/27/2016

Minimum Factor of Safety = 2.14

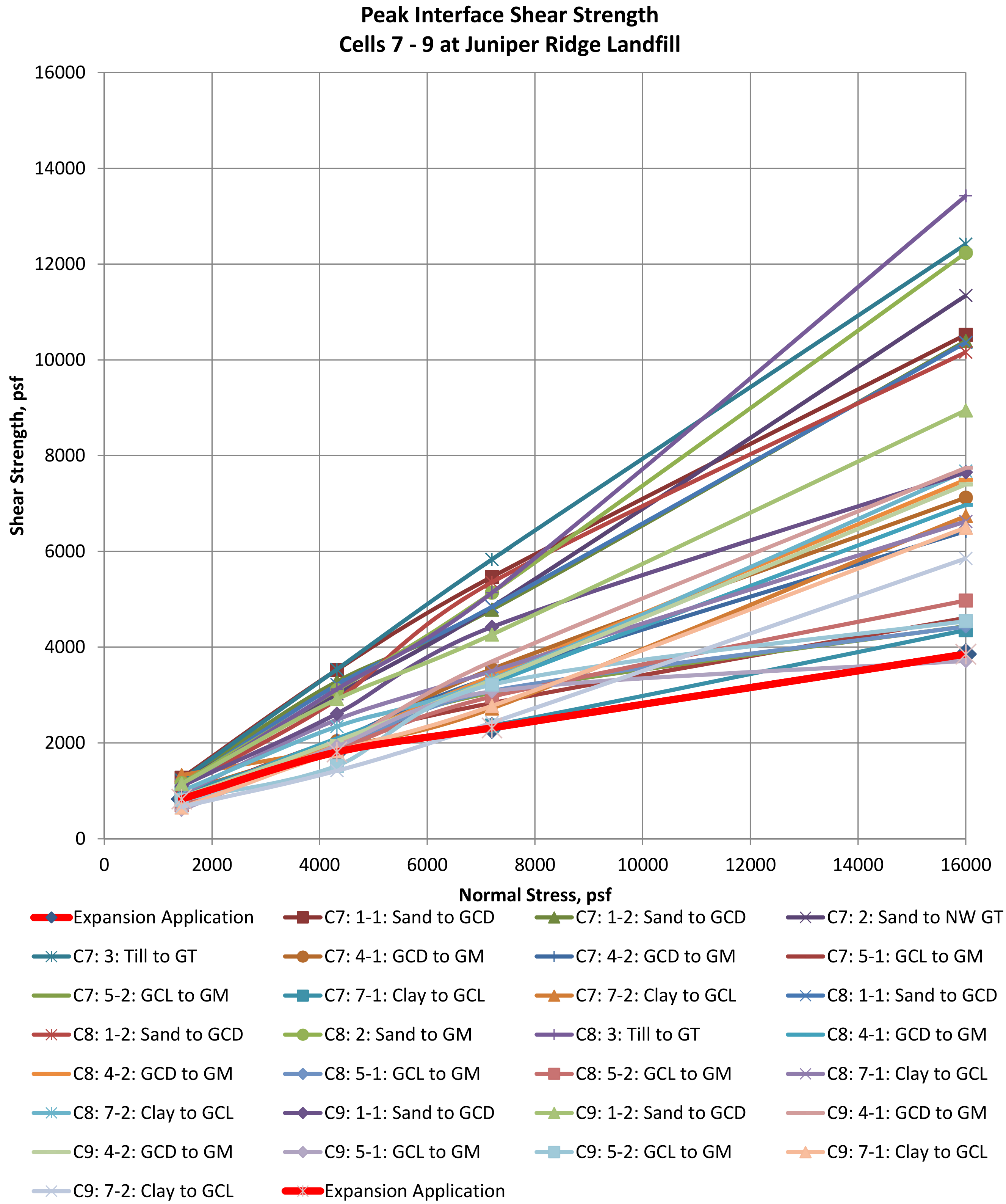
Name: Solid Waste   Model: Mohr-Coulomb   Unit Weight: 74   Cohesion: 0   Phi: 32   Phi-B: 0   Piezometric Line: 1
Name: Liner Peak Spec   Model: Shear/Normal Fn.   Unit Weight: 120   Strength Function: Liner Peak Spec   Phi-B: 0   Piezometric Line: 1
Name: Gravel Roads   Model: Mohr-Coulomb   Unit Weight: 128   Cohesion: 0   Phi: 34   Phi-B: 0   Piezometric Line: 2
Name: Foundation Soils (Glacial Till)   Model: Mohr-Coulomb   Unit Weight: 132   Cohesion: 1000   Phi: 38   Phi-B: 0   Piezometric Line: 2
Name: Till Road Base   Model: Mohr-Coulomb   Unit Weight: 128   Cohesion: 250   Phi: 34   Phi-B: 0   Piezometric Line: 2

Method: Bishop, Ordinary and Janbu
Analysis Name: 2bs Foundation Ops 1H:1V Slope s
Seismic Coefficient (ks): 0.07

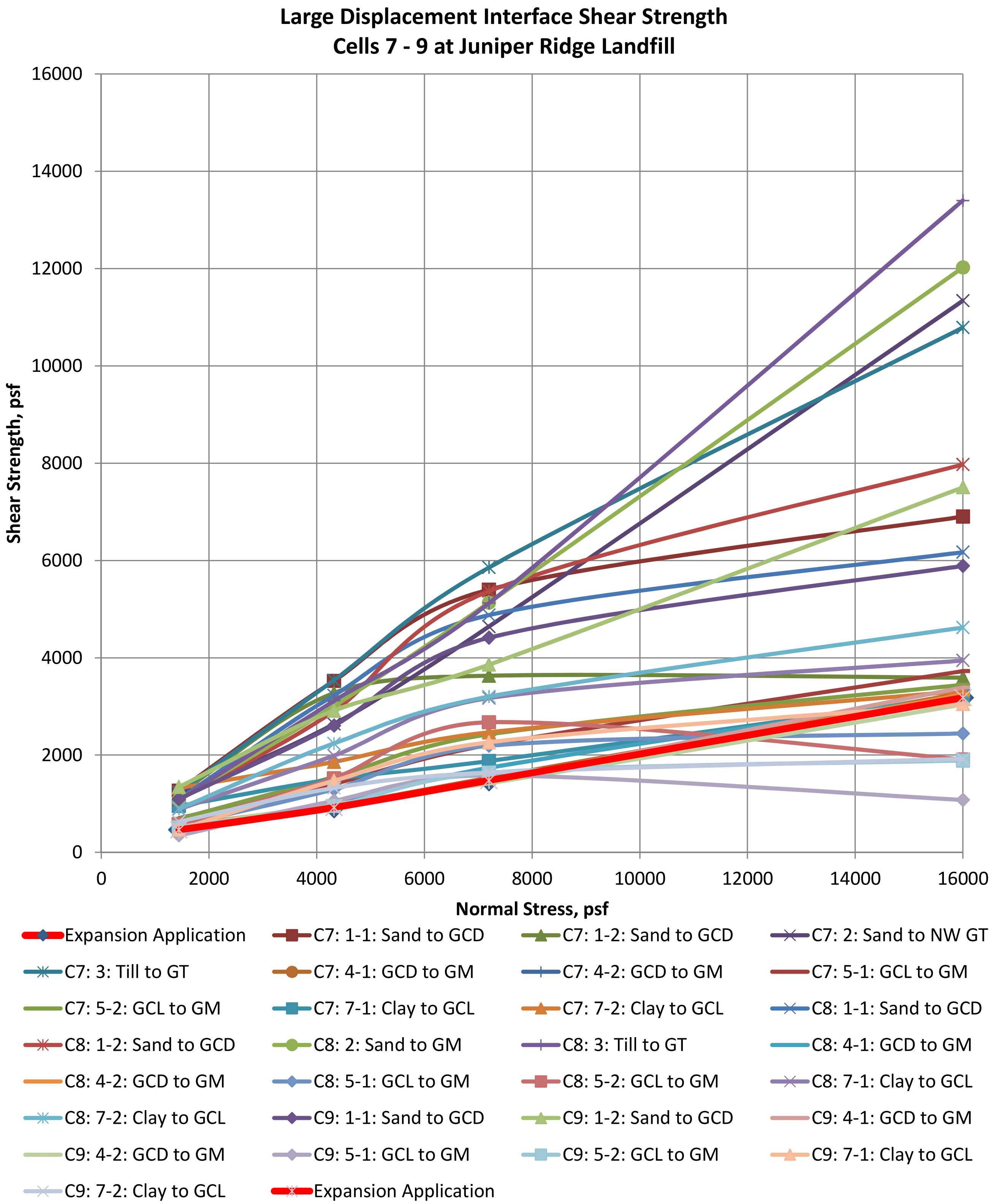
Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555



PEAK AND LARGE DISPLACEMENT LINER STRENGTH ENVELOPES



\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\Interface Summary



\\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\Interface Summary

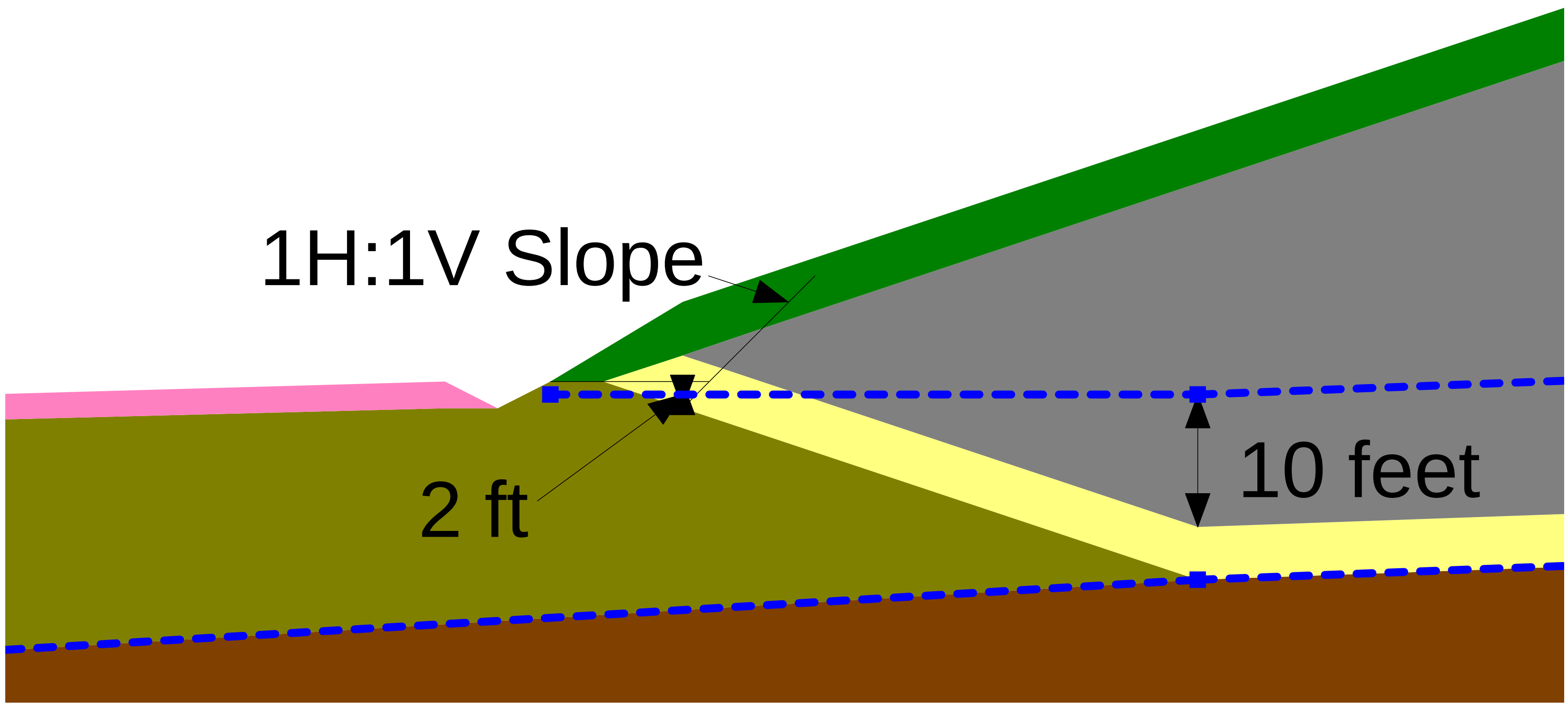
SLOPE STABILITY SENSITIVITY ANALYSIS

SUMMARY OF MINIMUM CALCULATED SLOPE STABILITY FACTORS OF SAFETY

Comparison of values in Table 3-9 of Volume III of the Application to a Sensitivity Analysis of a hypothetical increase in leachate head (10 foot increase above top of liner system).

Post Closure				
		Static Factors of Safety		
Slip Surface Location	Surface Shape	MEDEP Required Minimum	A-A' in Application (1 foot of head in Leachate Drainage Sand)	A-A' Sensitivity Analysis (11 feet of head in Leachate Drainage Sand)
Waste	Shallow Surficial	1.5	1.81	1.80
Liner	Block		1.72	1.65
Foundation	Circular		2.65	2.61
Slip Surface Location	Surface Shape	MEDEP Required Minimum	A-A' in Application (1 foot head in Leachate Drainage Sand)	A-A' Sensitivity Analysis (11 feet of head in Leachate Drainage Sand)
Waste	Shallow Surficial	1.0	1.11	1.11
Liner	Block		1.00	0.95
Foundation	Circular		1.62	1.60

Note: MEDEP Required Minimum values do not apply to the results of the sensitivity analysis.



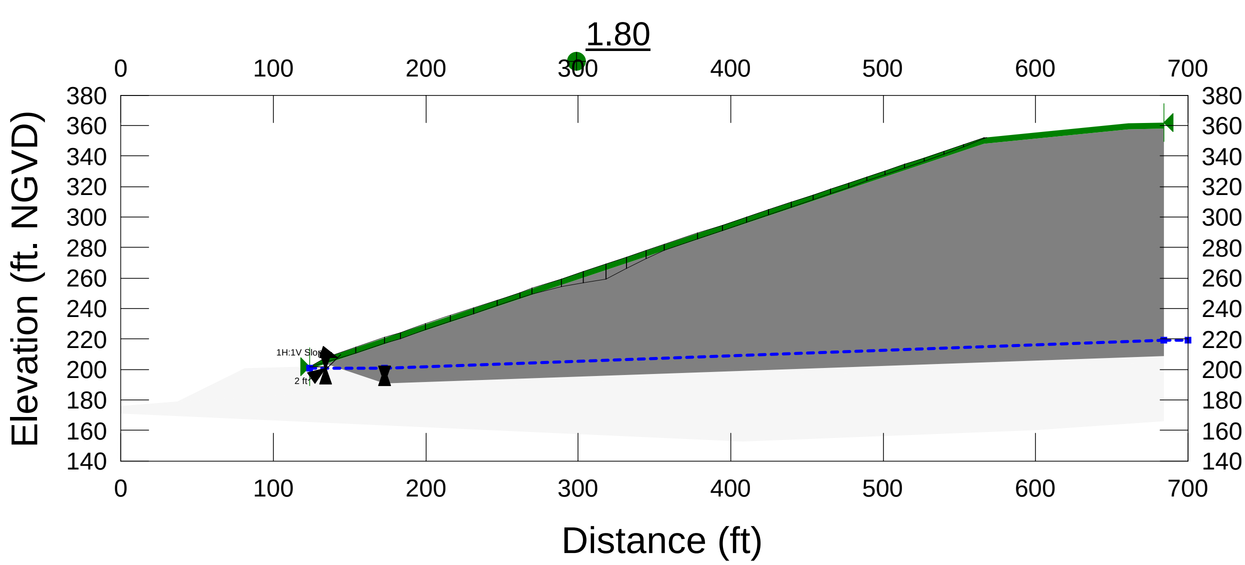
1H:1V Slope
2 ft
10 feet

Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Waste Closed; Sensitivity with 10 added ft of head on leachate sand.
Analysis Name: 5sen leachate Waste Closed
Date: 1/28/2016

Minimum Factor of Safety = 1.80

Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1

Method: Bishop, Ordinary and Janbu
Analysis Name: 5sen leachate Waste Closed
Seismic Coefficient (ks): 0



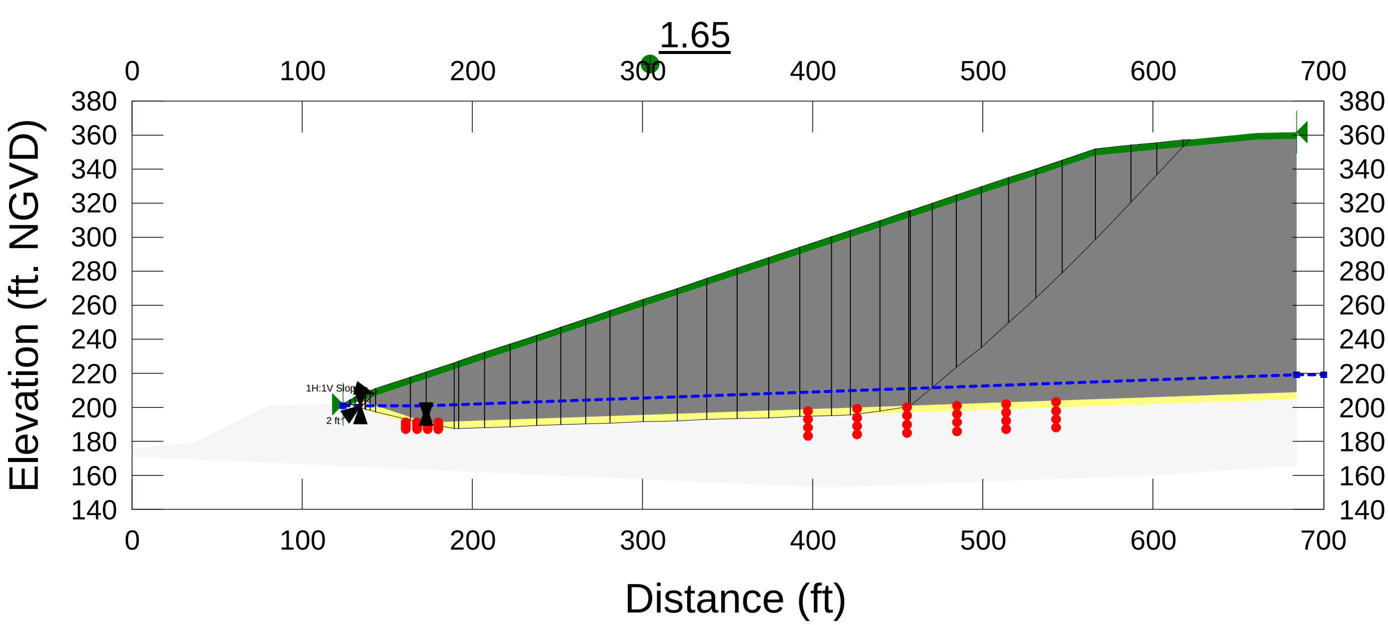
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Liner Closed; Sensitivity with 10 ft added head on leachate sand.
Analysis Name: 3sen_leachate Liner Closed
Date: 1/28/2016

Minimum Factor of Safety = 1.65

Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1

Method: Spencer
Analysis Name: 3sen_leachate Liner Closed
Seismic Coefficient (ks): 0

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555



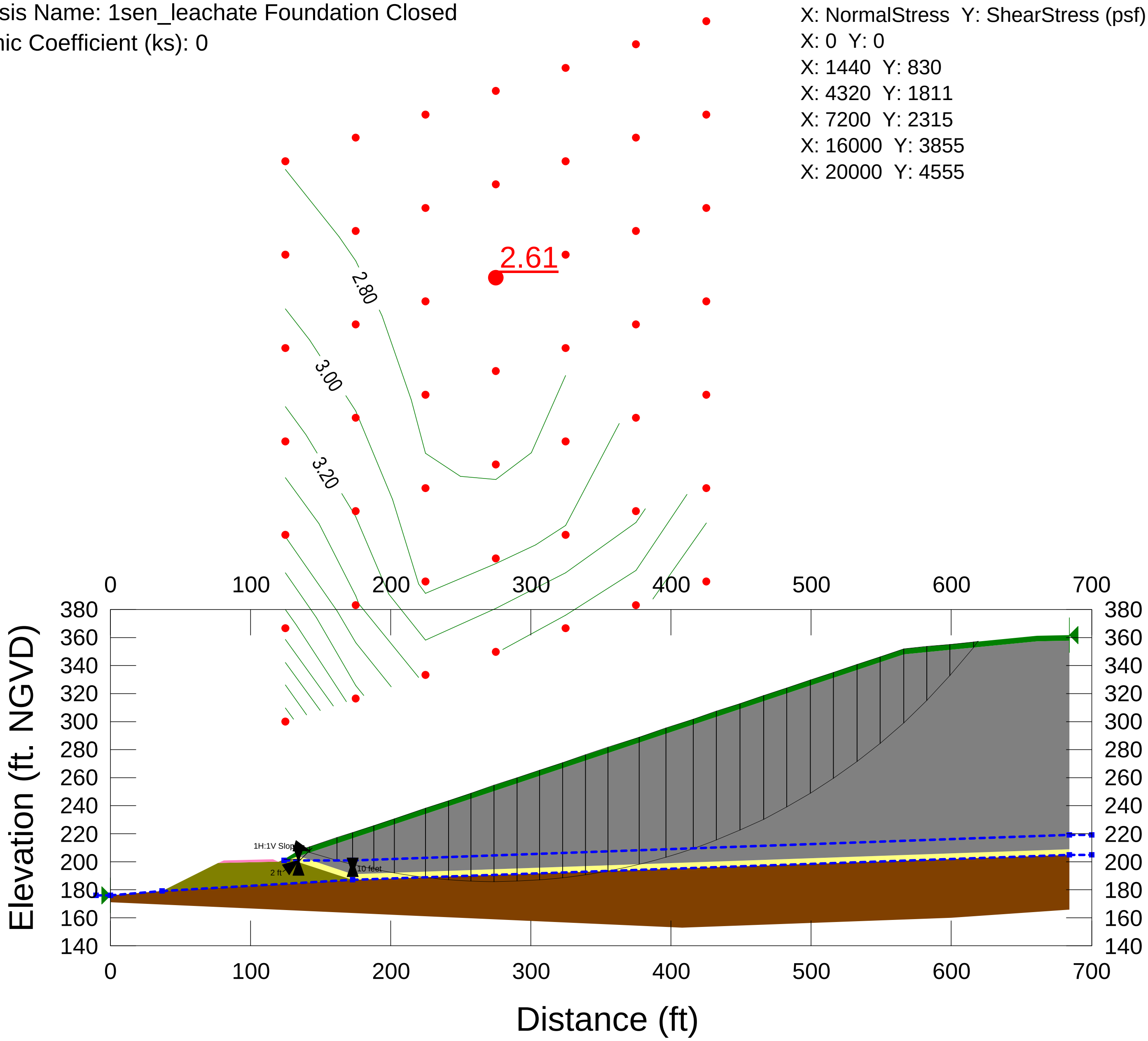
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Foundation Closed; Sensitivity to leachate levels.
Analysis Name: 1sen_leachate Foundation Closed
Date: 1/28/2016

Minimum Factor of Safety = 2.61

Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1
Name: Gravel Roads Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 0 Phi: 34 Phi-B: 0 Piezometric Line: 2
Name: Foundation Soils (Glacial Till) Model: Mohr-Coulomb Unit Weight: 132 Cohesion: 1000 Phi: 38 Phi-B: 0 Piezometric Line: 2
Name: Till Road Base Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 250 Phi: 34 Phi-B: 0 Piezometric Line: 2

Method: Bishop, Ordinary and Janbu
Analysis Name: 1sen_leachate Foundation Closed
Seismic Coefficient (ks): 0

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555

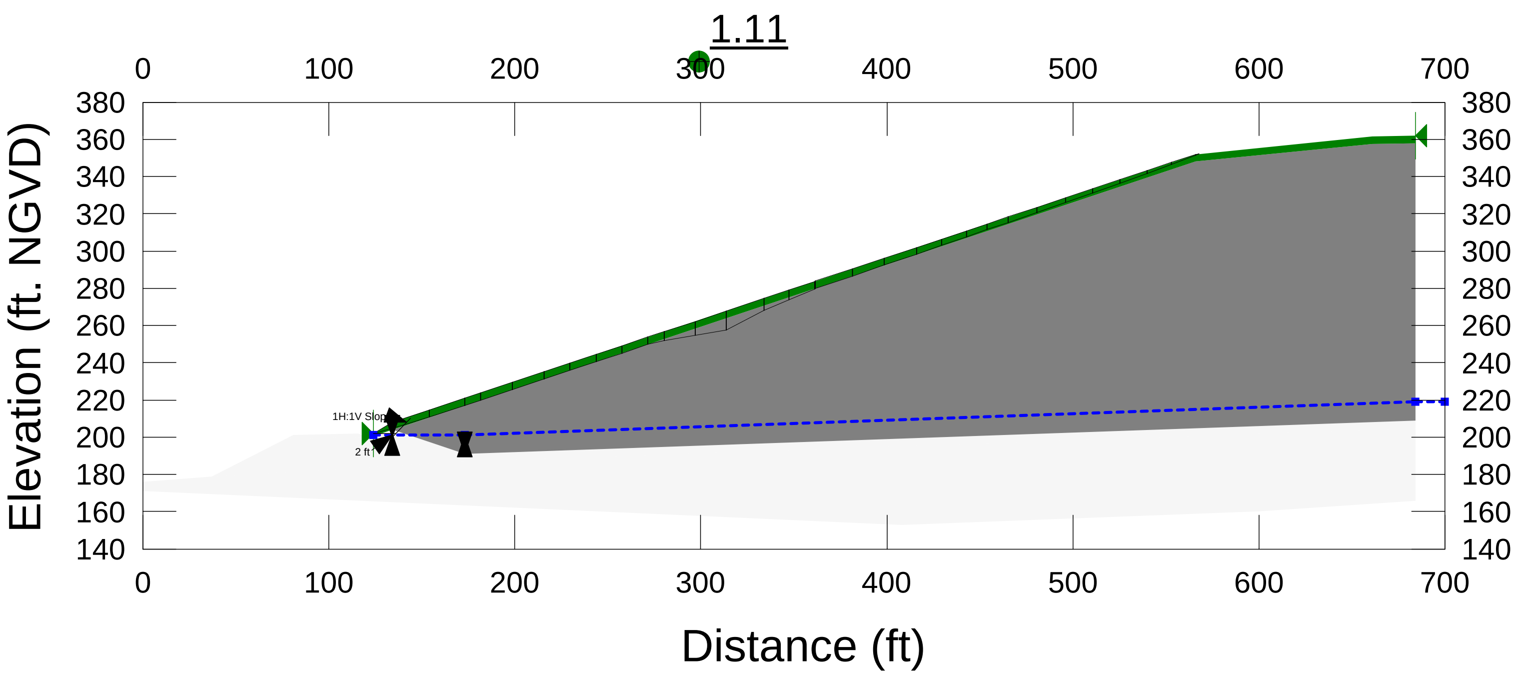


Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Waste Closed; Sensitivity with 10 added ft of head on leachate sand; seismic.
Analysis Name: 5sen_s_leachate Waste Closed
Date: 1/28/2016

Minimum Factor of Safety = 1.11

Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1

Method: Bishop, Ordinary and Janbu
Analysis Name: 5sen_s_leachate Waste Closed
Seismic Coefficient (ks): 0.18



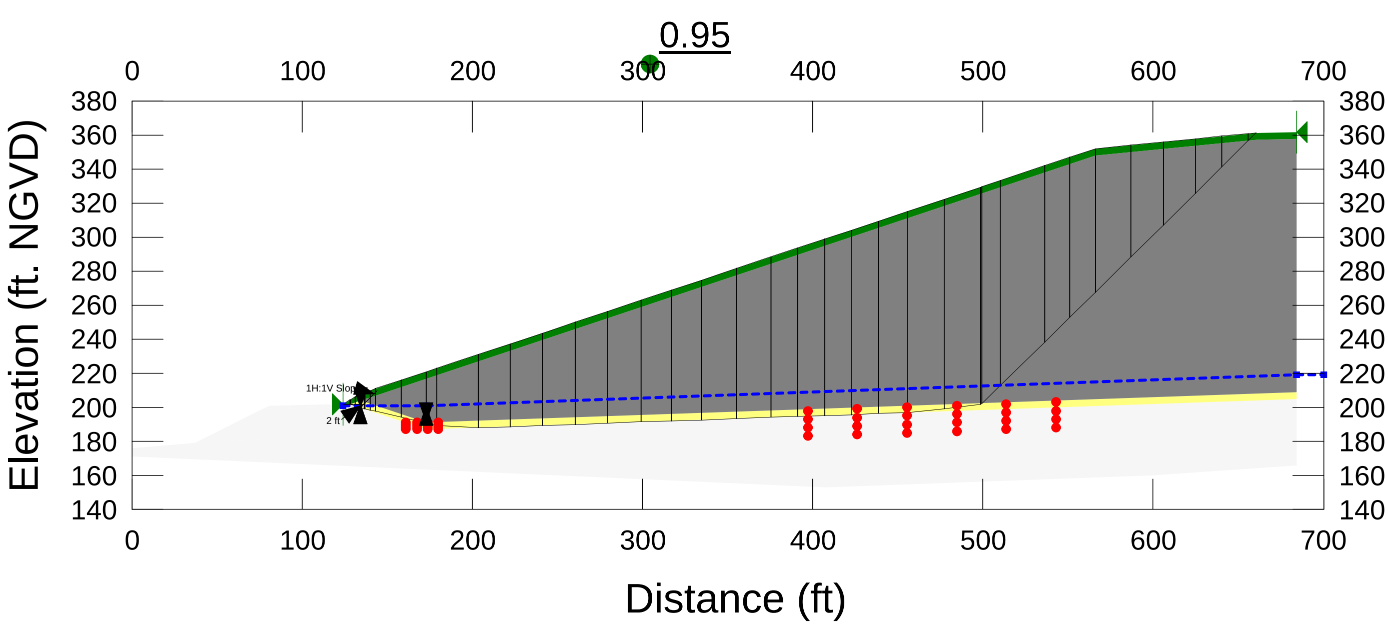
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Liner Closed; Sensitivity with 10 ft added head on leachate sand; seismic.
Analysis Name: 3sen_s_leachate Liner Closed (2)
Date: 1/28/2016

Minimum Factor of Safety = 0.95

Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1

Method: Spencer
Analysis Name: 3sen_s_leachate Liner Closed (2)
Seismic Coefficient (ks): 0.18

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555



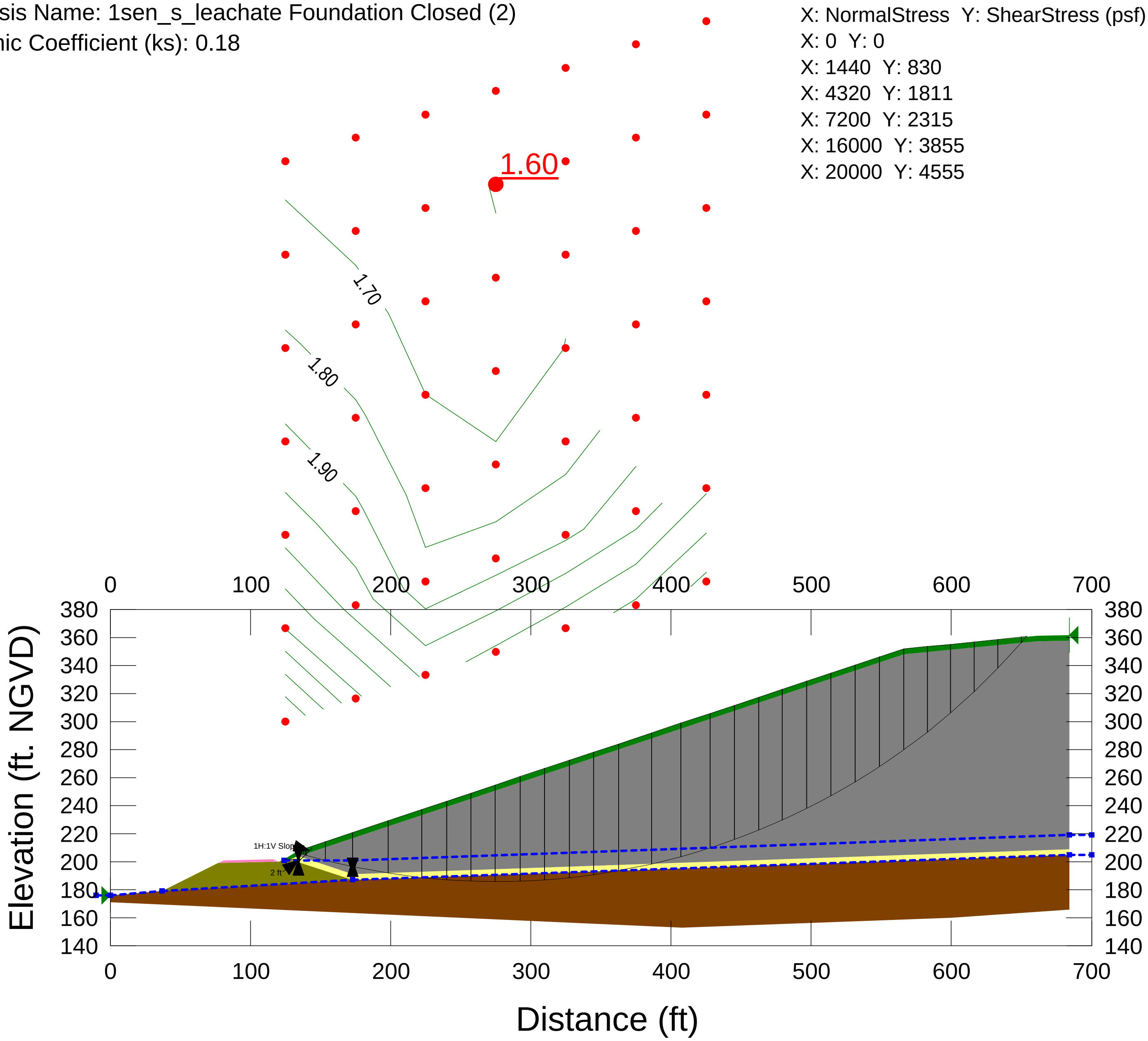
Directory: \\nserver\CFS\Casella\OldTownLandfill\Expansion\9.35MCY-Expansion\Geotech\RTC 01-2016\
File Name: Cross-Section A-A' RTC 01-2016.gsz
Description: Foundation Closed; Sensitivity to leachate levels; seismic.
Analysis Name: 1sen_s_leachate Foundation Closed (2)
Date: 1/28/2016

Minimum Factor of Safety = 1.60

Name: Cover System Model: Mohr-Coulomb Unit Weight: 125 Cohesion: 0 Phi: 30 Phi-B: 0
Name: Solid Waste Model: Mohr-Coulomb Unit Weight: 74 Cohesion: 0 Phi: 32 Phi-B: 0 Piezometric Line: 1
Name: Liner Peak Spec Model: Shear/Normal Fn. Unit Weight: 120 Strength Function: Liner Peak Spec Phi-B: 0 Piezometric Line: 1
Name: Gravel Roads Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 0 Phi: 34 Phi-B: 0 Piezometric Line: 2
Name: Foundation Soils (Glacial Till) Model: Mohr-Coulomb Unit Weight: 132 Cohesion: 1000 Phi: 38 Phi-B: 0 Piezometric Line: 2
Name: Till Road Base Model: Mohr-Coulomb Unit Weight: 128 Cohesion: 250 Phi: 34 Phi-B: 0 Piezometric Line: 2

Method: Bishop, Ordinary and Janbu
Analysis Name: 1sen_s_leachate Foundation Closed (2)
Seismic Coefficient (ks): 0.18

Name: Liner Peak Spec
X: NormalStress Y: ShearStress (psf)
X: 0 Y: 0
X: 1440 Y: 830
X: 4320 Y: 1811
X: 7200 Y: 2315
X: 16000 Y: 3855
X: 20000 Y: 4555



SME 7

VOLUME IV APPENDIX P
SUPPLEMENTAL EXAMPLE OF
LINER LEAKAGE ACTION PLAN CALCULATIONS

Juniper Ridge Landfill Expansion Example of Liner Leakage Action Plan Calculation

CELL 11			
	ALR -1	ALR-II	
Leakage per acre (gpd)	4.60	92	
Cell 11 area (acres)	9.5	9.5	
A) Liner leakage action flow rate (gpd)	44	874	
B) Base flow rate to underdrain (gpd)	1,414	1,421	Based recorded flow data from (UD-1)
C) Baseline specific conductance (umhos/cm)	310	310	based on average Cond from LF-UD-1 as of 2013 N=77
D) Leachate specific conductance (umhos/cm)	23,000	23,000	average value measured in 2015 at JRL
Leak Detection Action Level-UAL (umhos/cm)	990	8,951	
((BxC) +(DxA))/(A+B)			
2015 Maximum specific conductance (umhos/cm)	425		

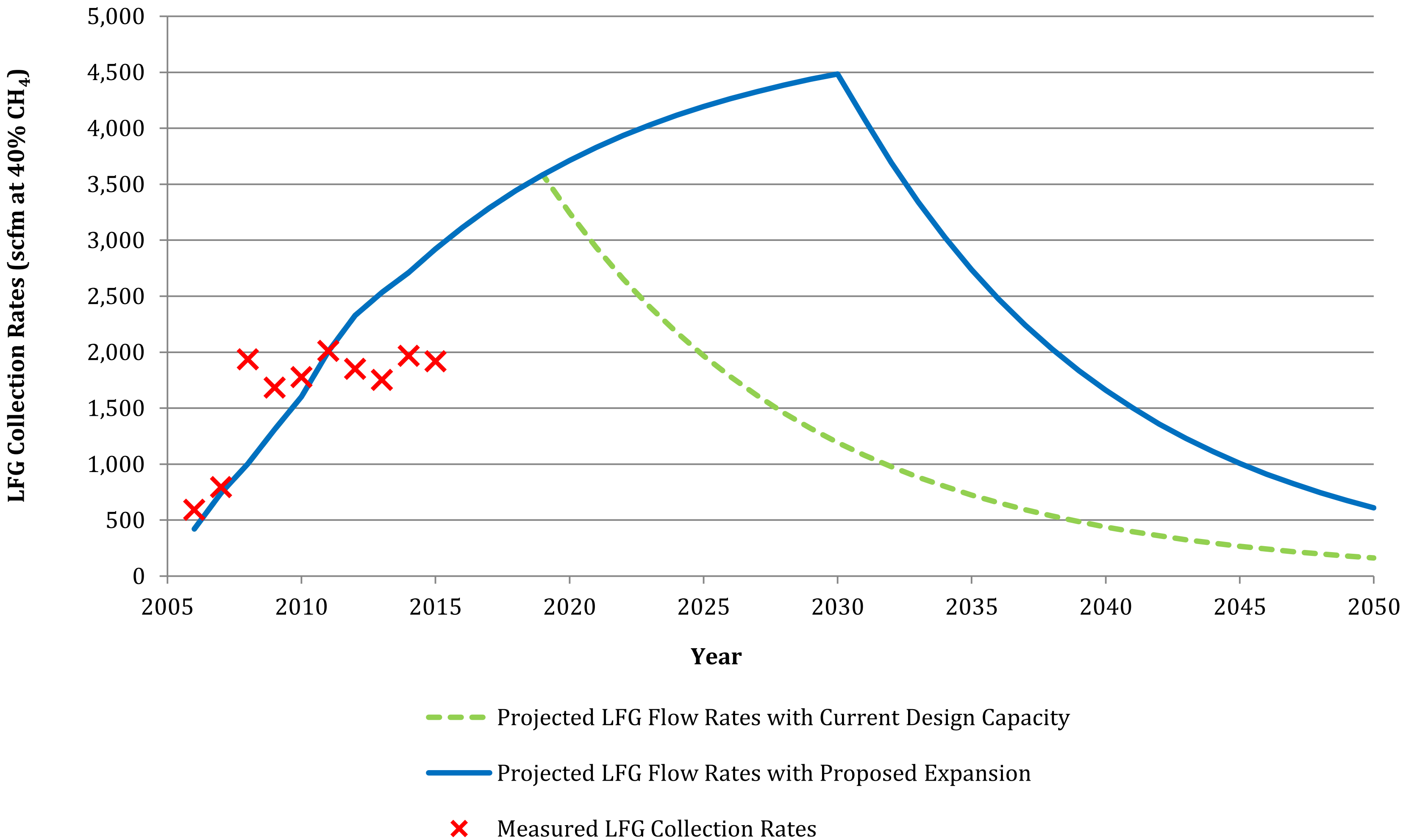
1,0 Calculation done for Cell 11 of the Expansion using data from Juniper Ridge Landfill existing site operations

SH-1

VOLUME III - APPENDIX I
FIGURE 2
PROJECTED LFG COLLECTION RATES WITH AND WITHOUT PROPOSED EXPANSION
(REVISED FEBRUARY 2016)

Figure 2 (Revised)
Projected LFG Collection Rates
With and Without Proposed Expansion

Juniper Ridge Landfill
Old Town, Maine



Sanborn, Head & Associates, Inc.

SH-2

VOLUME III - APPENDIX I
SECTION 5.2 OF THE
LFG SYSTEM EXPANSION DESIGN REPORT
(REVISED FEBRUARY 2016)

SANBORN HEAD

5.2 Gas Collection Trenches

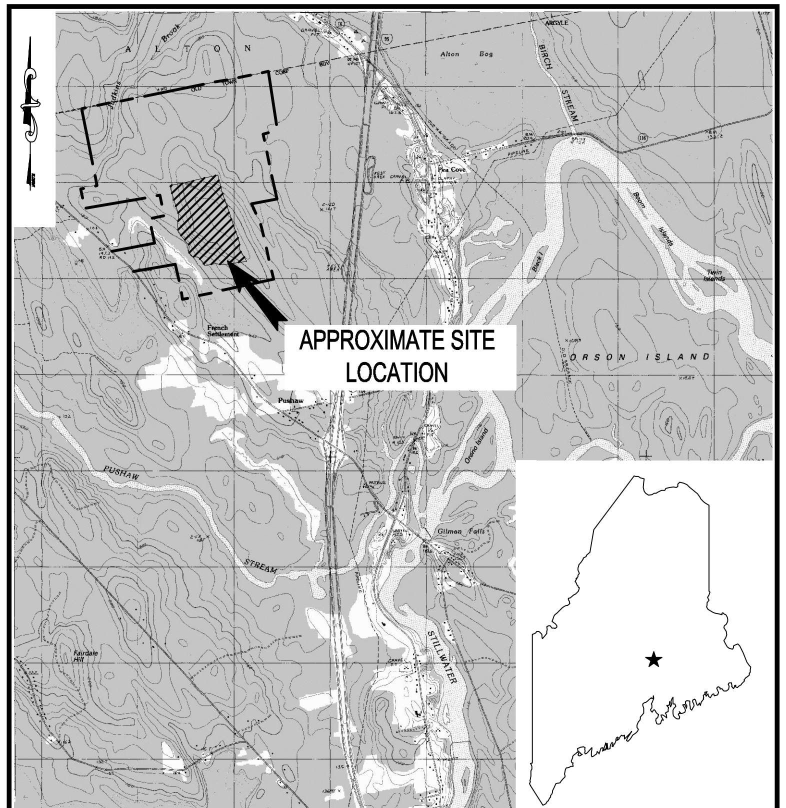
The GCTs will be constructed as an interim method of gas extraction until vertical extraction wells can be installed at final grades. The GCTs are designed assuming an elliptical effective radius of influence with a vertical radius of 20 feet and horizontal reach of 100 feet. The 4-foot wide by 5-foot deep collection trenches will be constructed within the waste mass during active filling to capture LFG early in its generation phase and reduce the potential for LFG migration and associated odors. Each GCT will consist of a 6-inch diameter perforated high-density polyethylene (HDPE) pipe embedded within a coarse aggregate (e.g., ballast stone or tire chips). ~~Intermediate cover will be placed above the trenches.~~ The trench construction detail is presented on Sheet 13 of the Engineering Drawings (Appendix B). For clarity the Engineering Drawings do not show the locations of interim GCCS infrastructure (including GCT's) to be installed within each cell. The actual locations of the GCT installations will be positioned based on the effective radius of influence described above.

SH-3

VOLUME III - APPENDIX I
LANDFILL GAS SYSTEM EXPANSION DRAWINGS
(REVISED FEBRUARY 2016)

LANDFILL GAS SYSTEM EXPANSION DRAWINGS

JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE
JUNE 2015
(REVISED FEBRUARY 2016)

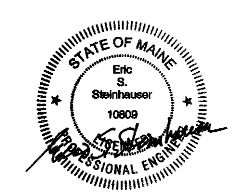


FROM: USGS TOPOGRAPHIC MAP- OLD TOWN, MAINE (1988)

LOCUS PLAN

SCALE: 1"=1000'

SHEET INDEX



REVISION TABLE

NO.	DATE	DESCRIPTION	BY
⚠	02/25/16	REVISED BASED ON MEDEP COMMENTS.	RLC

PREPARED FOR:

NEW ENGLAND WASTE SERVICES

NEWSME LANDFILL OPERATIONS, LLC

JUNIPER RIDGE LANDFILL

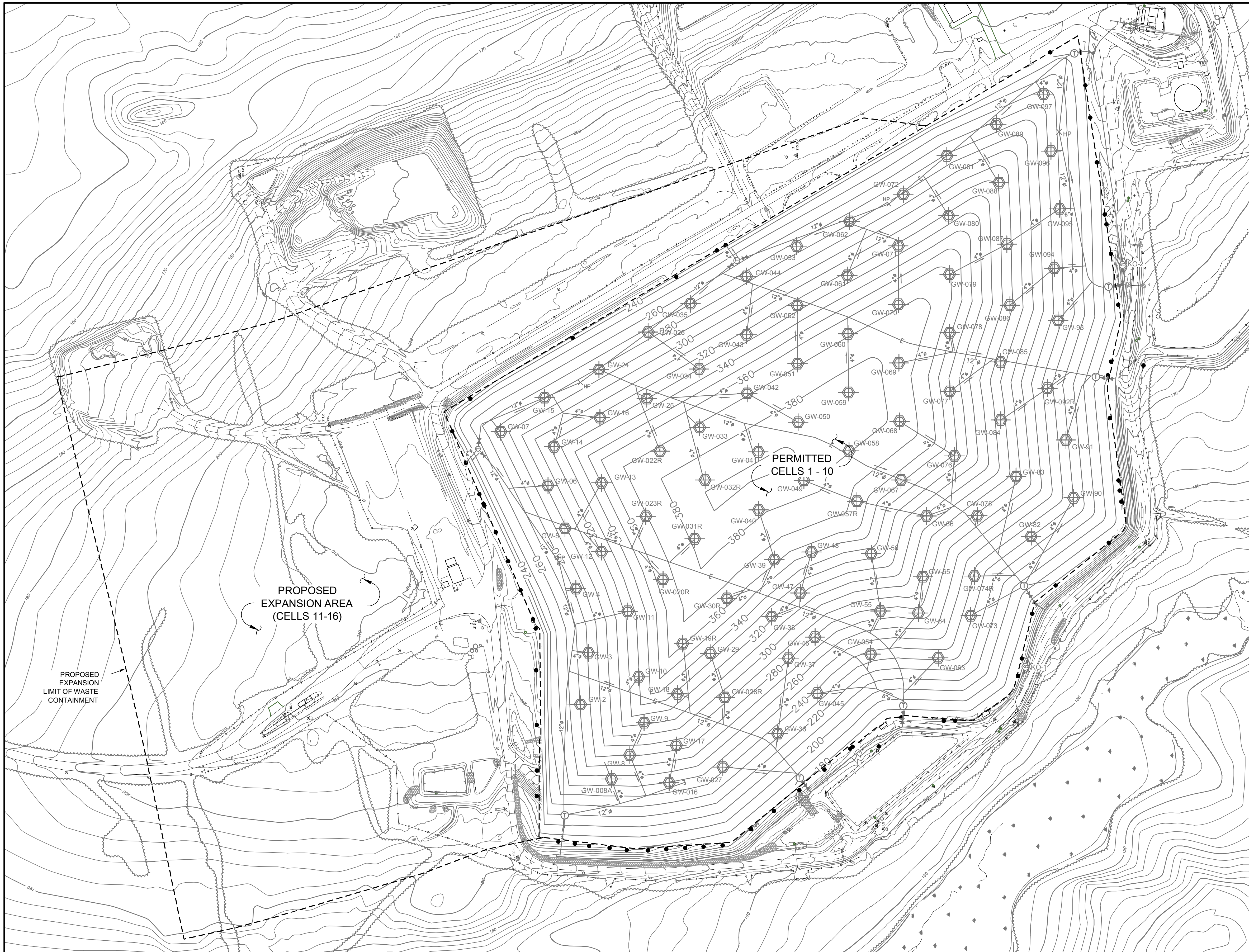
OLD TOWN, MAINE

PREPARED BY:

SANBORN HEAD

20 FOUNDRY STREET, CONCORD, NEW HAMPSHIRE 03301
(603) 229-1900 FAX (603) 229-1919

PROJ.#: 2536.27 "LANDFILL GAS SYSTEM EXPANSION DRAWINGS, JUNIPER RIDGE LANDFILL, OLD TOWN, MAINE" REVISED FEBRUARY 2016.



NOTES:

1. THE TOPOGRAPHY AND SITE FEATURES SHOWN OUTSIDE THE LIMIT OF WASTE CONTAINMENT WERE PREPARED BY AERIAL SURVEY & PHOTO INC., OF NORRIDGEWOCK, MAINE. PHOTO DATE JULY 31, 2014. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE.

3. THE GRADES SHOWN INSIDE THE LIMIT OF WASTE CONTAINMENT ARE PERMITTED FINAL GRADES FOR CELLS 1 - 10 OF THE JUNIPER RIDGE LANDFILL (JRL), AND WERE PROVIDED TO SANBORN HEAD BY SEVEE & MAHER ENGINEERS, INC. OF CUMBERLAND, MAINE.

4. THIS DRAWING SHOWS THE LANDFILL GAS (LFG) SYSTEM DESIGN FOR CELLS 1-10 PRIOR TO THE PROPOSED EXPANSION FOR CELLS 11-16. THIS IS NOT AN AS-BUILT DRAWING. EXISTING GRADES AND ACTUAL LOCATIONS OF LFG SYSTEM INFRASTRUCTURE MAY BE DIFFERENT THAN SHOWN AT THE TIME OF CONSTRUCTION OF THE EXPANSION. AN UPDATED LFG SYSTEM AS-BUILT DRAWING SHOULD BE USED DURING CONSTRUCTION.

LEGEND:

Symbol	Description
180	10 FOOT CONTOUR
	2 FOOT CONTOUR
GW-11	VERTICAL LFG WELL
GW-19R	REPLACEMENT VERTICAL LFG WELL
12"ø	LFG CONVEYANCE PIPE (SIZE AND SLOPE DIRECTION)
T	CONDENSATE TRAP
	CONTROL VALVE
KO-1	CONDENSATE KNOCKOUT
	LEACHATE COLLECTION CLEANOUT
	VERTICAL RISER
	EDGE OF ROAD
HP	HIGH POINT
	TEMPORARY PIPE TERMINATION
	LIMIT OF WASTE CONTAINMENT

SANBORN HEAD

GRAPHICAL SCALE
150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	ADDED THIS SHEET.	RLC

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

CELLS 1 - 10
PROJECTED DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 1 OF 14

NOTES:

1. SEE SHEET 1 FOR ADDITIONAL NOTES.
2. PROPOSED EXPANSION GRADES SHOWN WERE PROVIDED TO SANBORN HEAD BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE.
3. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
4. THE EXISTING LANDFILL GAS EXTRACTION SYSTEM INFRASTRUCTURE FEATURES SHOWN ARE BASED ON A COMBINATION OF DESIGN AND AS-BUILT DOCUMENTATION AVAILABLE TO SANBORN, HEAD & ASSOCIATES, INC. (SANBORN HEAD). ACTUAL LOCATIONS OF INDIVIDUAL FEATURES MAY BE DIFFERENT THAN SHOWN.
5. THE LOCATIONS OF MANY OF THE LANDFILL DESIGN COMPONENTS SHOWN ON THIS PLAN, SUCH AS LEACHATE CLEANOUTS, STORMWATER MANAGEMENT FEATURES, AND UTILITIES, ARE BASED ON PROPOSED LOCATIONS PROVIDED TO SANBORN HEAD BY SEVEE & MAHER ENGINEERS, INC. OF CUMBERLAND, MAINE.
6. THIS PLAN IS INTENDED TO ILLUSTRATE THE PROPOSED LAYOUT OF THE LANDFILL GAS (LFG) EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
7. SOLID LANDFILL GAS CONVEYANCE PIPE LOCATED INSIDE THE LIMIT OF WASTE CONTAINMENT SHALL BE INSTALLED AT A MINIMUM SLOPE OF 5 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
8. HDPE PIPE AND FITTINGS SHALL BE SDR-17.
9. EXISTING VERTICAL LFG WELLS SHALL BE REPLACED OR EXTENDED AS NECESSARY PRIOR TO INSTALLATION OF FINAL CAP SYSTEM

LEGEND:

EXISTING		PROPOSED
180	10 FOOT CONTOUR	370
	2 FOOT CONTOUR	
GW-11	VERTICAL LFG WELL	GW-35
GW-19R	REPLACEMENT VERTICAL LFG WELL	
12"ø	LFG CONVEYANCE PIPE (SIZE AND SLOPE DIRECTION)	12"ø
T	CONDENSATE TRAP	T
	CONTROL VALVE	
KO KO-1	CONDENSATE KNOCKOUT	KO KO-4
	LEACHATE COLLECTION CLEANOUT	
	VERTICAL RISER	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
HP	HIGH POINT	HP
	RIPRAP-LINED DOWNCHUTE	
	TEMPORARY PIPE TERMINATION	

SANBORN | HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	REVISED BASED ON MEDEP COMMENTS.	RLC

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN

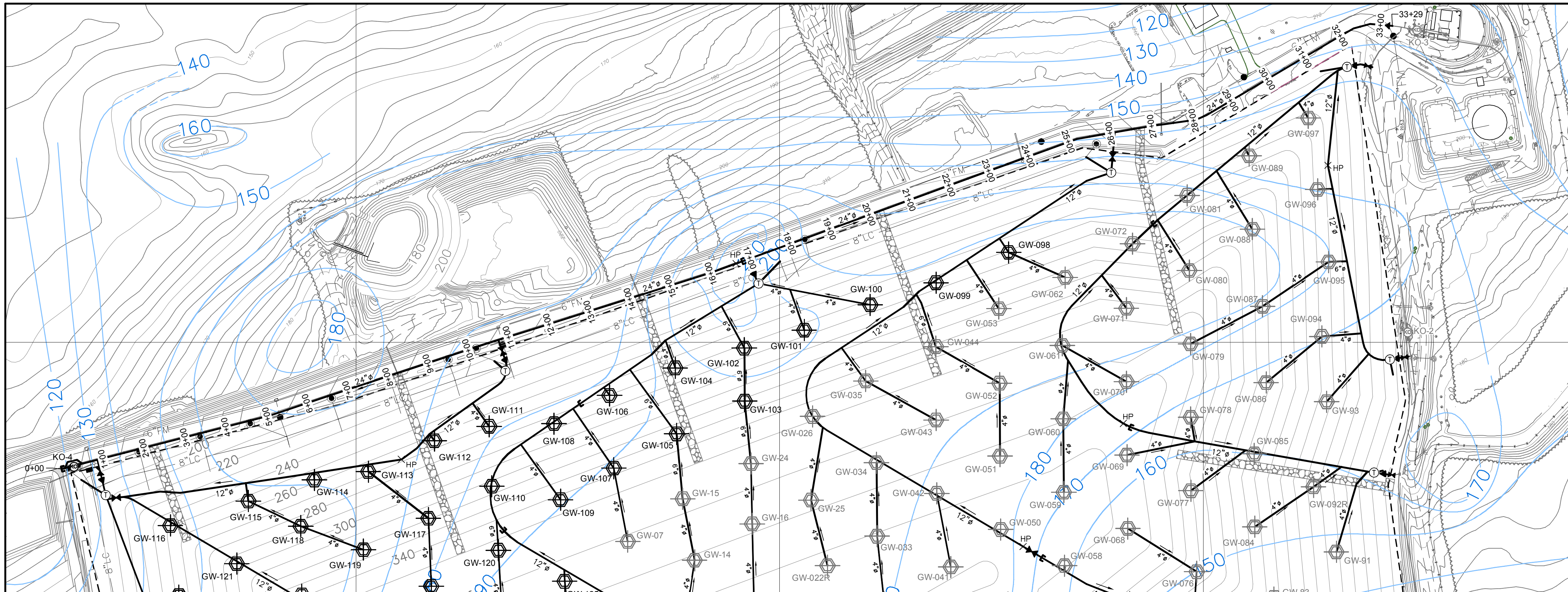
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

LANDFILL GAS EXTRACTION SYSTEM PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 2 OF 14

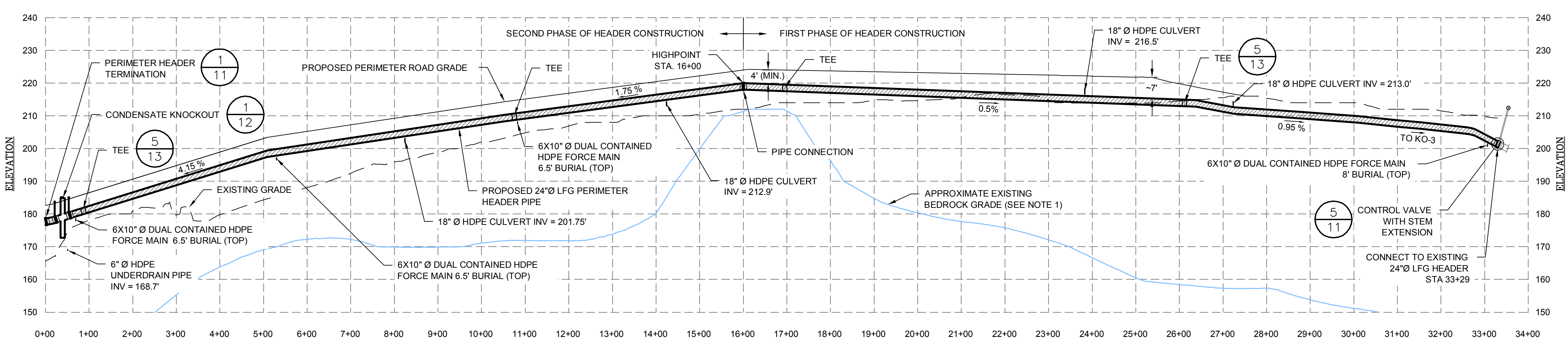


PLAN

NOT TO SCALE

GRAPHICAL SCALE

150' 75' 0' 150' 300'



PROFILE

NOT TO SCALE

VERTICAL SCALE

20' 10' 0' 20' 40'

150' 75' 0' 150' 300'

HORIZONTAL SCALE

NOTE:

1. EXISTING BEDROCK CONTOURS WERE PROVIDED TO SANBORN HEAD BY SME ON NOVEMBER 14, 2014. THESE GRADES ARE PRELIMINARY AND SHOULD BE CONSIDERED APPROXIMATE.
2. FORCE MAIN AND CULVERT CROSSINGS SHOWN ON PROFILE WERE PROVIDED BY SME ON APRIL 6, 2015.

210 BEDROCK CONTOUR

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE Eric S. Steinhauser 10809 LICENSED PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN

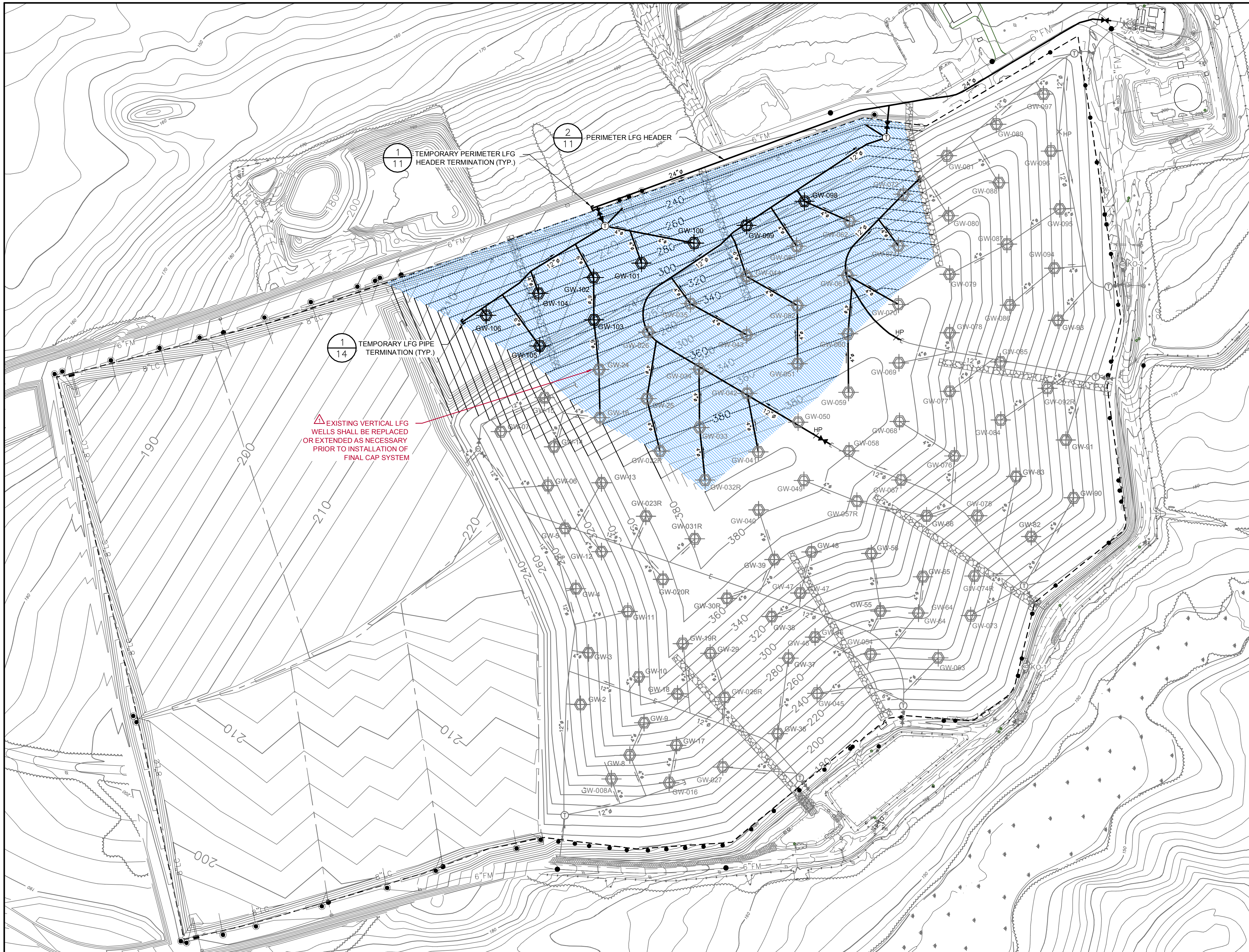
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

PERIMETER LFG HEADER PIPE PROFILE

PROJECT NUMBER: 2536.27

SHEET NUMBER: 3 OF 14



NOTES:

1. SHEETS 4 THRU 9 OF THIS DRAWING SET SHOW THE SEQUENTIAL CONSTRUCTION OF THE LANDFILL GAS SYSTEM FOR CELLS 11-16. ACTUAL LOCATIONS OF INDIVIDUAL FEATURES AND TIMING OF INSTALLATION MAY CHANGE DUE TO FUTURE PLANNING OR CONSTRAINTS DURING CONSTRUCTION.
2. GAS COLLECTION TRENCHES WILL BE INSTALLED AS FILLING PROGRESSES IN EACH CELL AND ARE NOT SHOWN FOR CLARITY. GAS COLLECTION TRENCHES ARE INTENDED TO BE A TEMPORARY MEASURE OF GAS COLLECTION UNTIL FINAL GRADES ARE REACHED AND VERTICAL WELLS ARE INSTALLED.
3. SEE SHEETS 1 AND 2 FOR ADDITIONAL NOTES AND LEGEND.

= AREA EXPECTED TO REACH FINAL GRADE

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE Eric S. Steinhauser 10809 PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	REVISED BASED ON MEDEP COMMENTS.	RLC

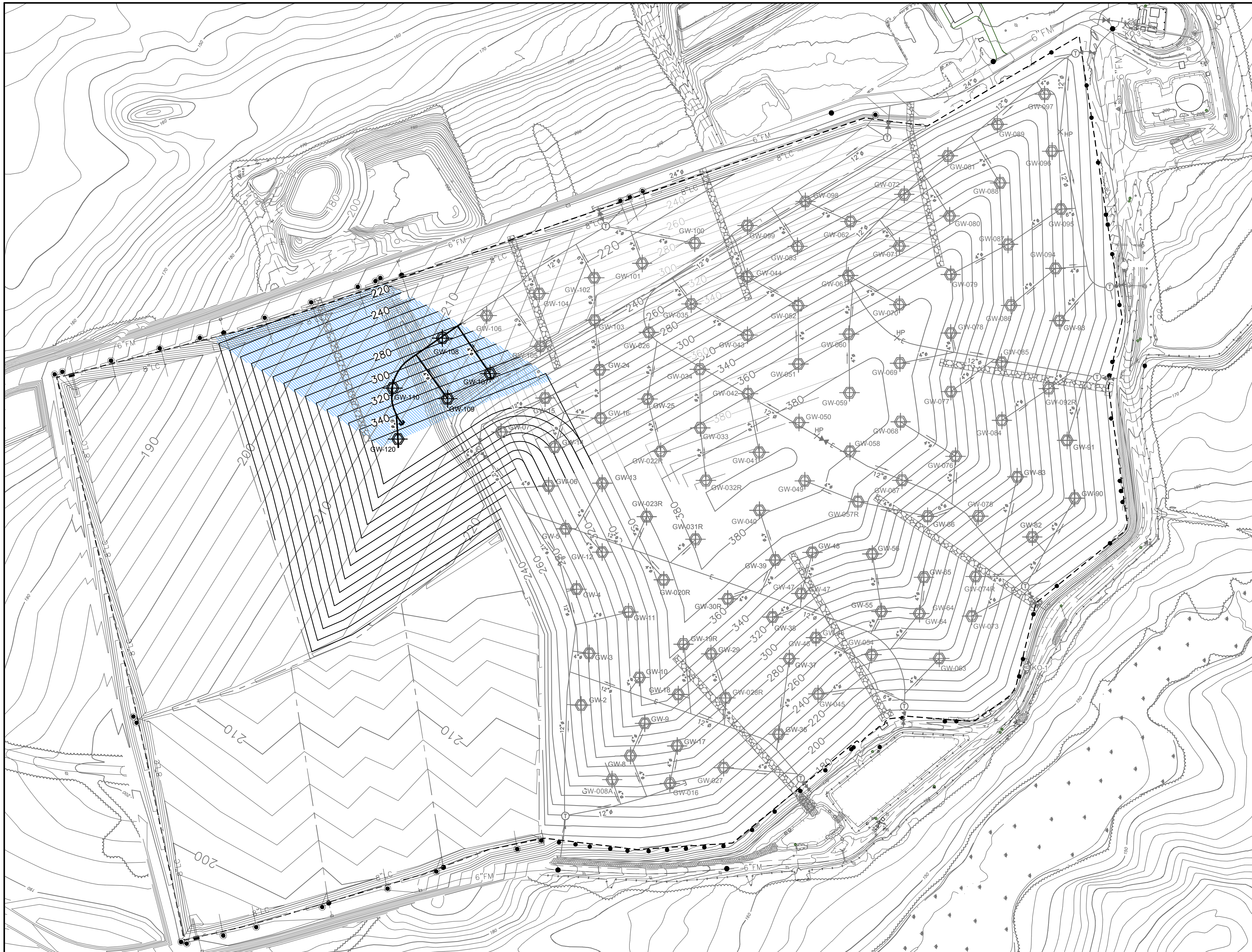
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

CELL 11 LFG INFRASTRUCTURE DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 4 OF 14



NOTES:

1. SHEETS 4 THRU 9 OF THIS DRAWING SET SHOW THE SEQUENTIAL CONSTRUCTION OF THE LANDFILL GAS SYSTEM FOR CELLS 11-16. ACTUAL LOCATIONS OF INDIVIDUAL FEATURES AND TIMING OF INSTALLATION MAY CHANGE DUE TO FUTURE PLANNING OR CONSTRAINTS DURING CONSTRUCTION.

2. GAS COLLECTION TRENCHES WILL BE INSTALLED AS FILLING PROGRESSES IN EACH CELL AND ARE NOT SHOWN FOR CLARITY. GAS COLLECTION TRENCHES ARE INTENDED TO BE A TEMPORARY MEASURE OF GAS COLLECTION UNTIL FINAL GRADES ARE REACHED AND VERTICAL WELLS ARE INSTALLED.

3. SEE SHEETS 1 AND 2 FOR ADDITIONAL NOTES AND LEGEND.

= AREA EXPECTED TO REACH FINAL GRADE

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN

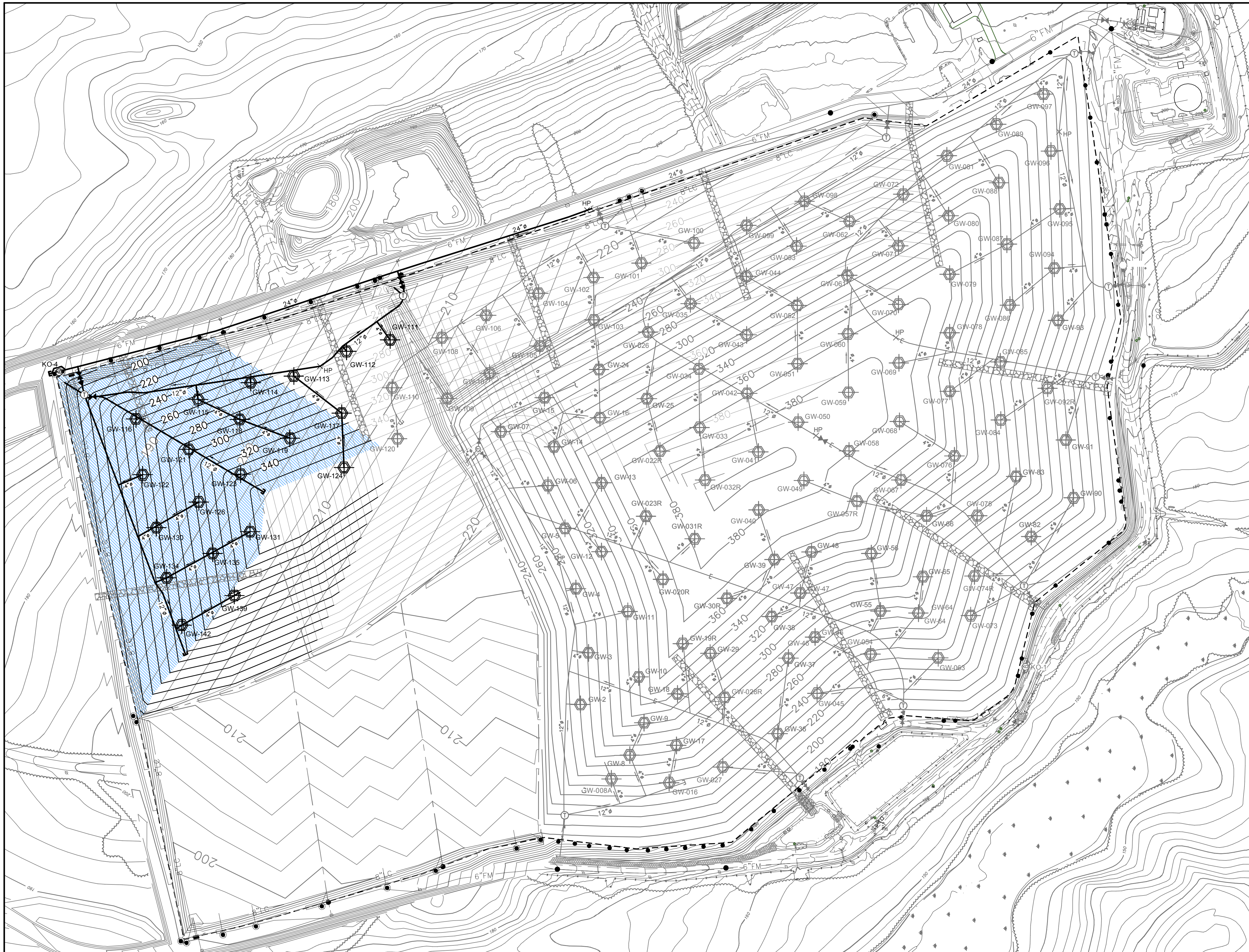
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

CELL 12 LFG INFRASTRUCTURE DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 5 OF 14



NOTES:

1. SHEETS 4 THRU 9 OF THIS DRAWING SET SHOW THE SEQUENTIAL CONSTRUCTION OF THE LANDFILL GAS SYSTEM FOR CELLS 11-16. ACTUAL LOCATIONS OF INDIVIDUAL FEATURES AND TIMING OF INSTALLATION MAY CHANGE DUE TO FUTURE PLANNING OR CONSTRAINTS DURING CONSTRUCTION.

2. GAS COLLECTION TRENCHES WILL BE INSTALLED AS FILLING PROGRESSES IN EACH CELL AND ARE NOT SHOWN FOR CLARITY. GAS COLLECTION TRENCHES ARE INTENDED TO BE A TEMPORARY MEASURE OF GAS COLLECTION UNTIL FINAL GRADES ARE REACHED AND VERTICAL WELLS ARE INSTALLED.

3. SEE SHEETS 1 AND 2 FOR ADDITIONAL NOTES AND LEGEND.

= AREA EXPECTED TO REACH FINAL GRADE

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN

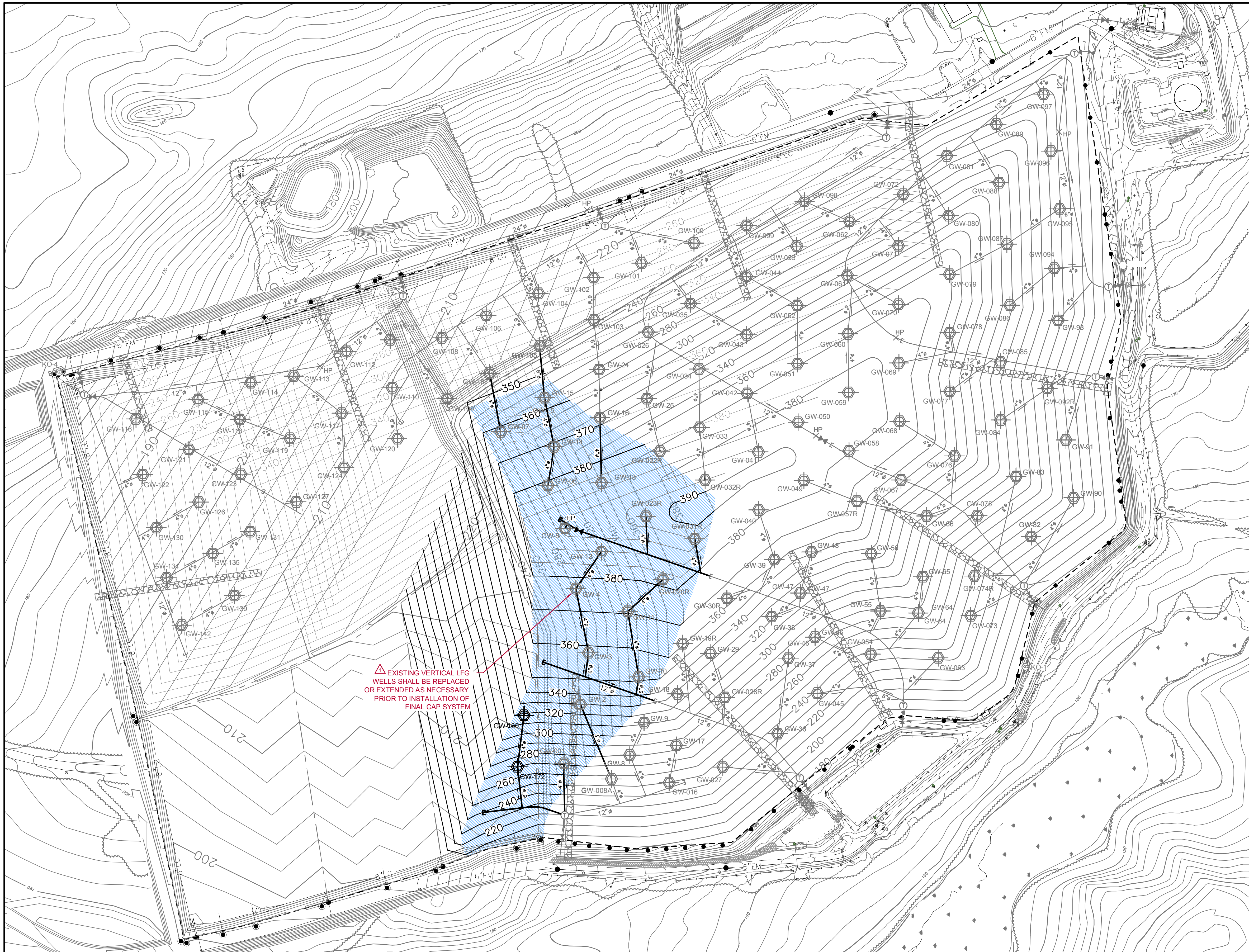
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

CELL 13 LFG INFRASTRUCTURE DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 6 OF 14



NOTES:

1. SHEETS 4 THRU 9 OF THIS DRAWING SET SHOW THE SEQUENTIAL CONSTRUCTION OF THE LANDFILL GAS SYSTEM FOR CELLS 11-16. ACTUAL LOCATIONS OF INDIVIDUAL FEATURES AND TIMING OF INSTALLATION MAY CHANGE DUE TO FUTURE PLANNING OR CONSTRAINTS DURING CONSTRUCTION.
2. GAS COLLECTION TRENCHES WILL BE INSTALLED AS FILLING PROGRESSES IN EACH CELL AND ARE NOT SHOWN FOR CLARITY. GAS COLLECTION TRENCHES ARE INTENDED TO BE A TEMPORARY MEASURE OF GAS COLLECTION UNTIL FINAL GRADES ARE REACHED AND VERTICAL WELLS ARE INSTALLED.
3. SEE SHEETS 1 AND 2 FOR ADDITIONAL NOTES AND LEGEND.

= AREA EXPECTED TO REACH FINAL GRADE

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	REVISED BASED ON MEDEP COMMENTS.	RLC

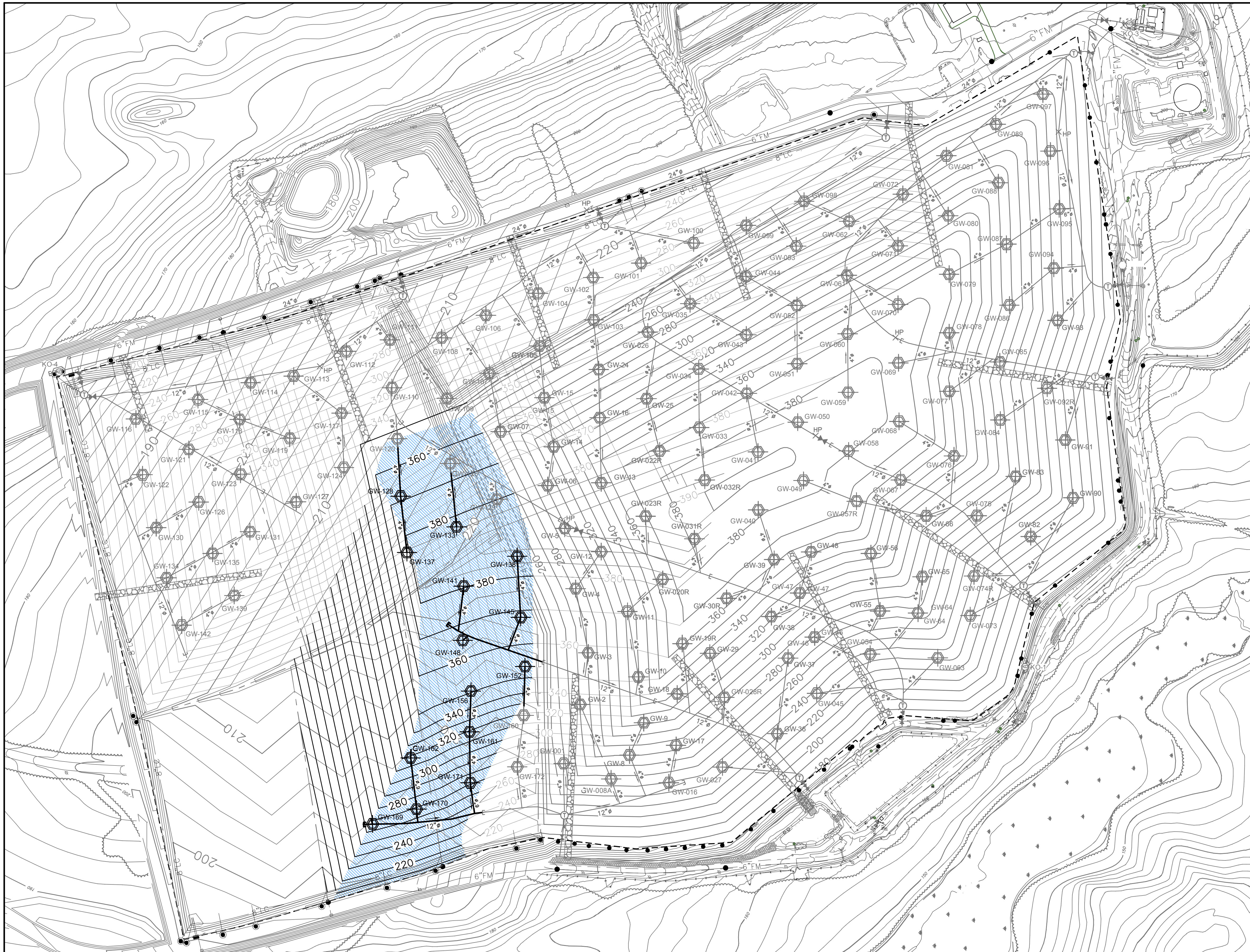
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

CELL 14 LFG INFRASTRUCTURE DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 7 OF 14



NOTES:

1. SHEETS 4 THRU 9 OF THIS DRAWING SET SHOW THE SEQUENTIAL CONSTRUCTION OF THE LANDFILL GAS SYSTEM FOR CELLS 11-16. ACTUAL LOCATIONS OF INDIVIDUAL FEATURES AND TIMING OF INSTALLATION MAY CHANGE DUE TO FUTURE PLANNING OR CONSTRAINTS DURING CONSTRUCTION.
2. GAS COLLECTION TRENCHES WILL BE INSTALLED AS FILLING PROGRESSES IN EACH CELL AND ARE NOT SHOWN FOR CLARITY. GAS COLLECTION TRENCHES ARE INTENDED TO BE A TEMPORARY MEASURE OF GAS COLLECTION UNTIL FINAL GRADES ARE REACHED AND VERTICAL WELLS ARE INSTALLED.
3. SEE SHEETS 1 AND 2 FOR ADDITIONAL NOTES AND LEGEND.

= AREA EXPECTED TO REACH FINAL GRADE

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

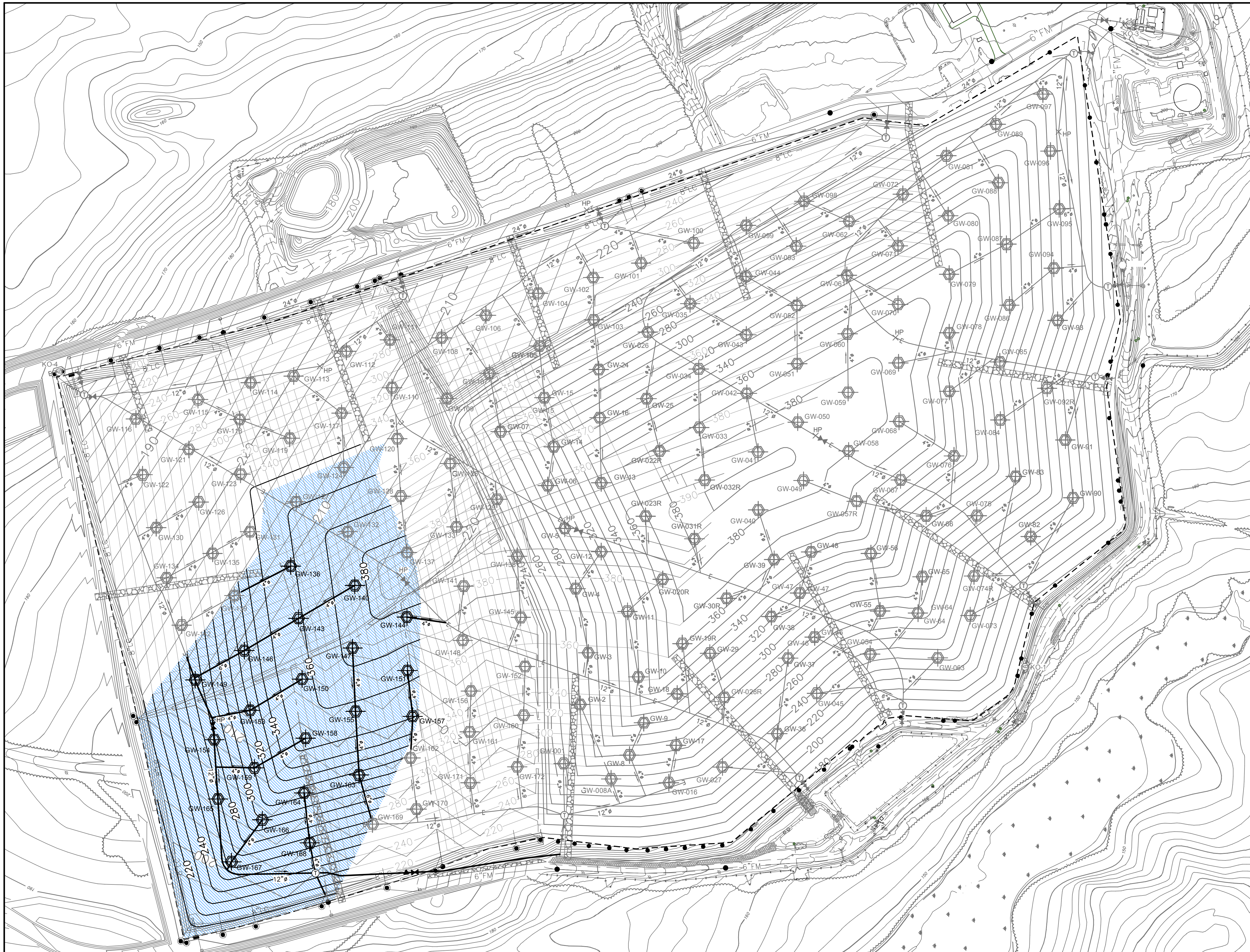
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

CELL 15 LFG INFRASTRUCTURE DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 8 OF 14



NOTES:

1. SHEETS 4 THRU 9 OF THIS DRAWING SET SHOW THE SEQUENTIAL CONSTRUCTION OF THE LANDFILL GAS SYSTEM FOR CELLS 11-16. ACTUAL LOCATIONS OF INDIVIDUAL FEATURES AND TIMING OF INSTALLATION MAY CHANGE DUE TO FUTURE PLANNING OR CONSTRAINTS DURING CONSTRUCTION.
2. GAS COLLECTION TRENCHES WILL BE INSTALLED AS FILLING PROGRESSES IN EACH CELL AND ARE NOT SHOWN FOR CLARITY. GAS COLLECTION TRENCHES ARE INTENDED TO BE A TEMPORARY MEASURE OF GAS COLLECTION UNTIL FINAL GRADES ARE REACHED AND VERTICAL WELLS ARE INSTALLED.
3. SEE SHEETS 1 AND 2 FOR ADDITIONAL NOTES AND LEGEND.

= AREA EXPECTED TO REACH FINAL GRADE

SANBORN | HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN

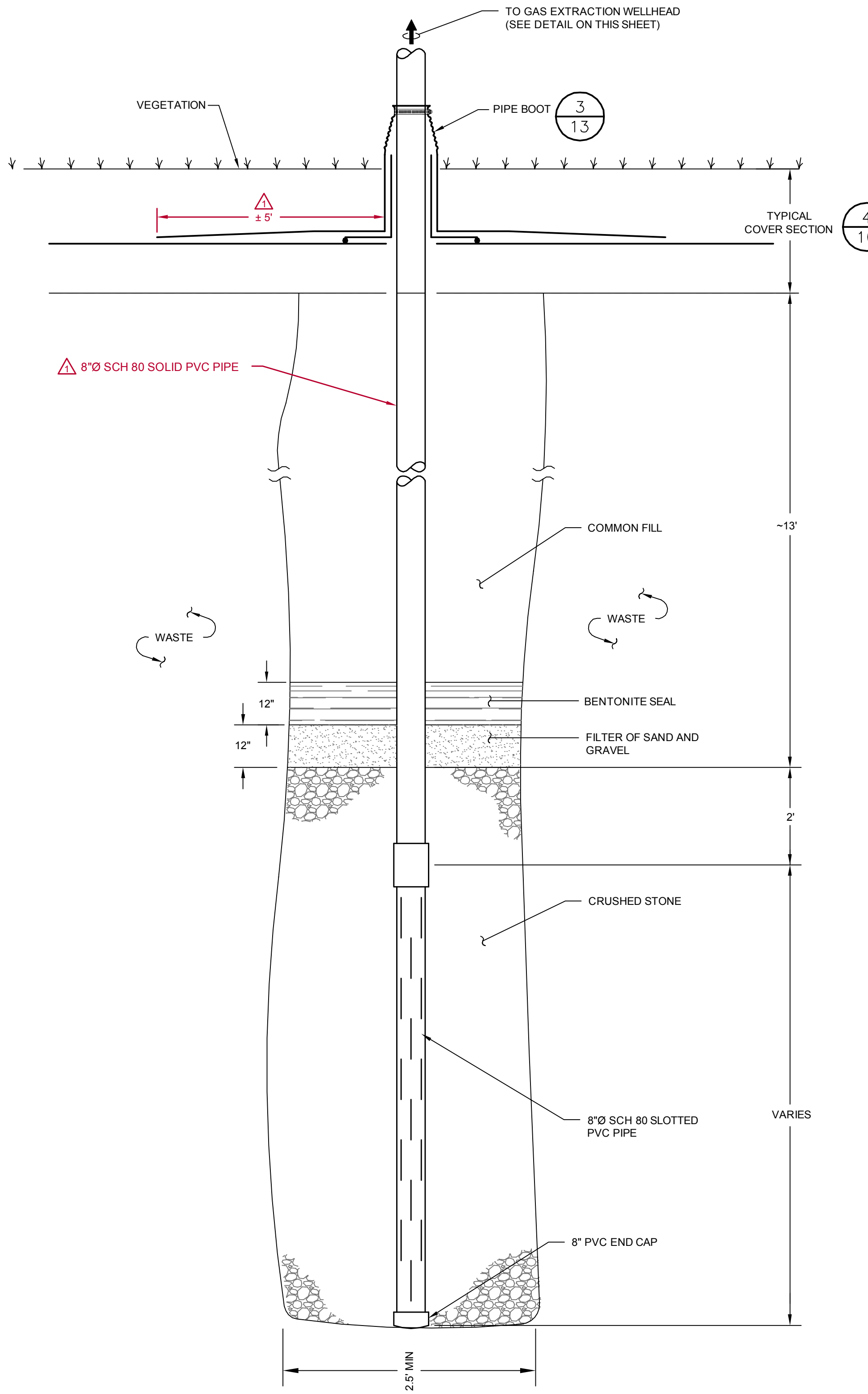
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

CELL 16 LFG INFRASTRUCTURE DEVELOPMENT PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 9 OF 14

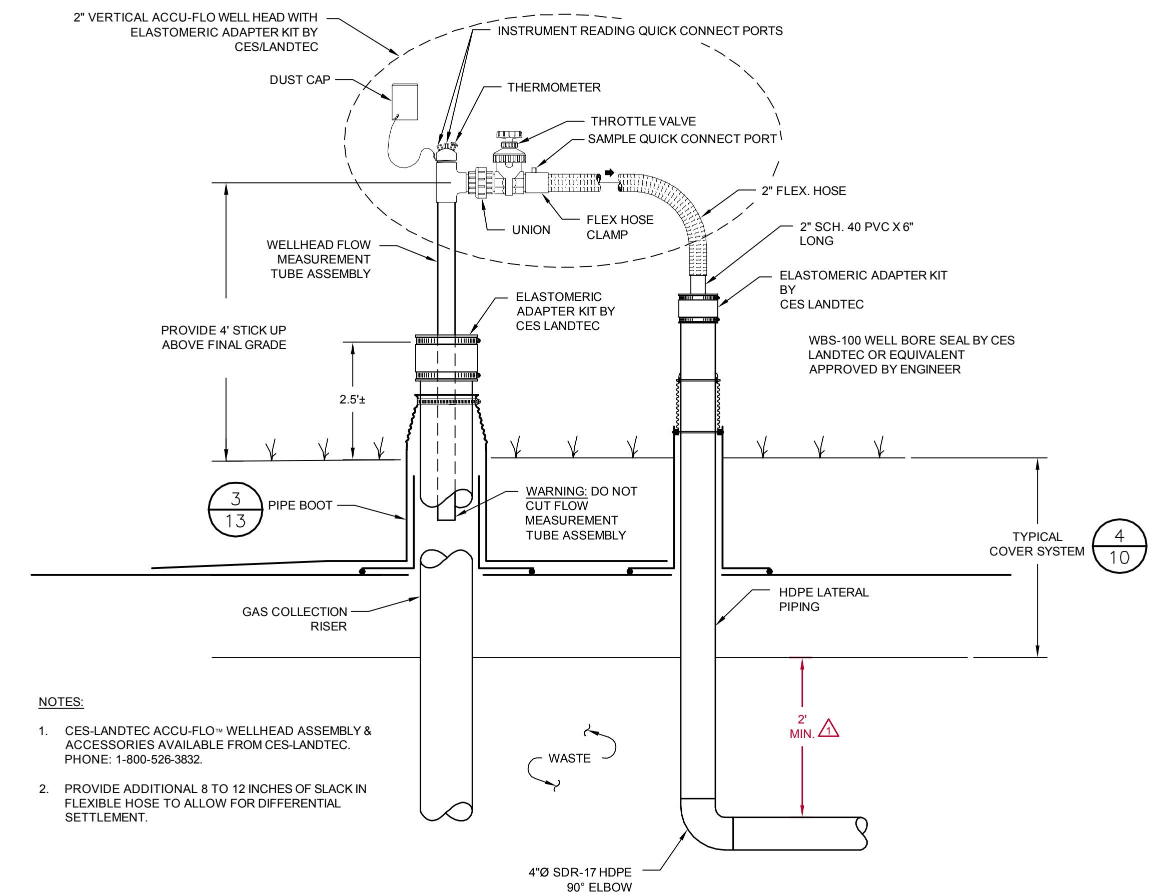


NOTES:

1. PIPE PERFORATED WITH SLOTS 1/8" TO 1/4" WIDE BY 8" LONG. FOUR SLOTS PER ROW SPACED 90° APART, WITH ADJACENT ROWS OFFSET BY 45°.
2. BOTTOM OF WELL IS DESIGNED TO BE 15 FEET ABOVE PRIMARY LINER SAND. REFER TO WELL SCHEDULE FOR ELEVATIONS.

1 TYPICAL GAS EXTRACTION WELL DETAIL

NOT TO SCALE

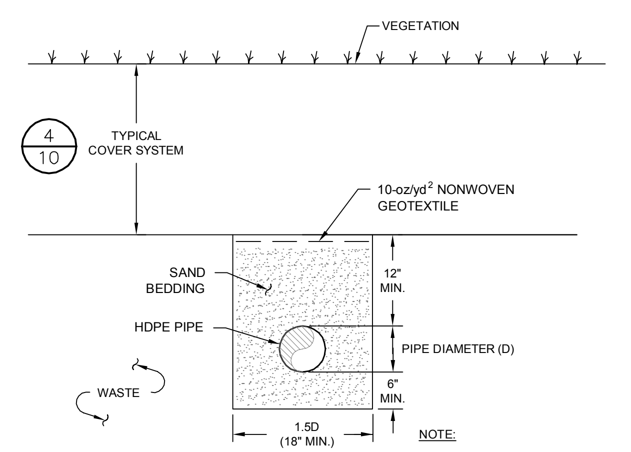


NOTES:

1. CES-LANDTEC ACCU-FLO™ WELLHEAD ASSEMBLY & ACCESSORIES AVAILABLE FROM CES-LANDTEC. PHONE: 1-800-526-3832.
2. PROVIDE ADDITIONAL 8 TO 12 INCHES OF SLACK IN FLEXIBLE HOSE TO ALLOW FOR DIFFERENTIAL SETTLEMENT.

2 TYPICAL GAS EXTRACTION WELLHEAD DETAIL

NOT TO SCALE

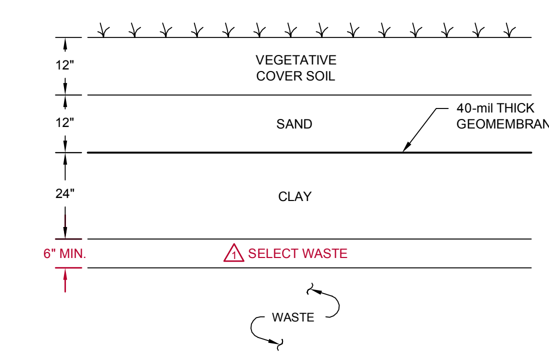


NOTE:

1. PROVIDE MINIMUM 6% PIPE SLOPE.

3 TYPICAL GAS PIPE TRENCH WITHIN LIMIT OF WASTE CONTAINMENT

NOT TO SCALE



4 TYPICAL COVER SYSTEM

NOT TO SCALE

SANBORN | HEAD

SCALE AS NOTED

STATE OF MAINE Eric S. Steinhauser 10809 PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	REVISED BASED ON MEDEP COMMENTS.	RLC

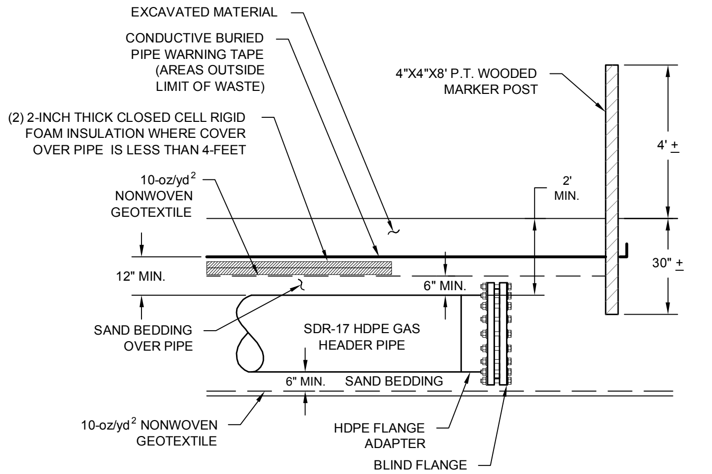
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LANDFILL GAS EXTRACTION SYSTEM DETAILS

PROJECT NUMBER: 2536.27

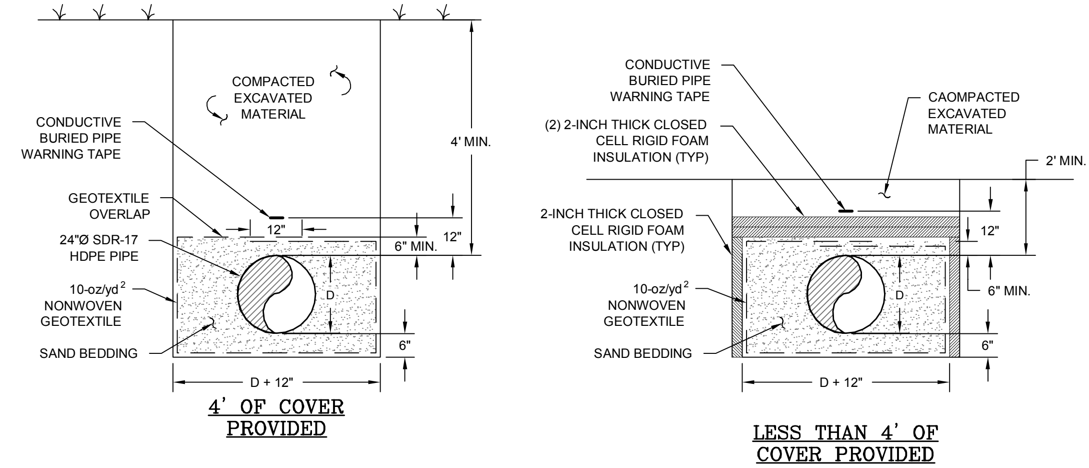
SHEET NUMBER: 10 OF 14



TEMPORARY PERIMETER LFG HEADER TERMINATION

1

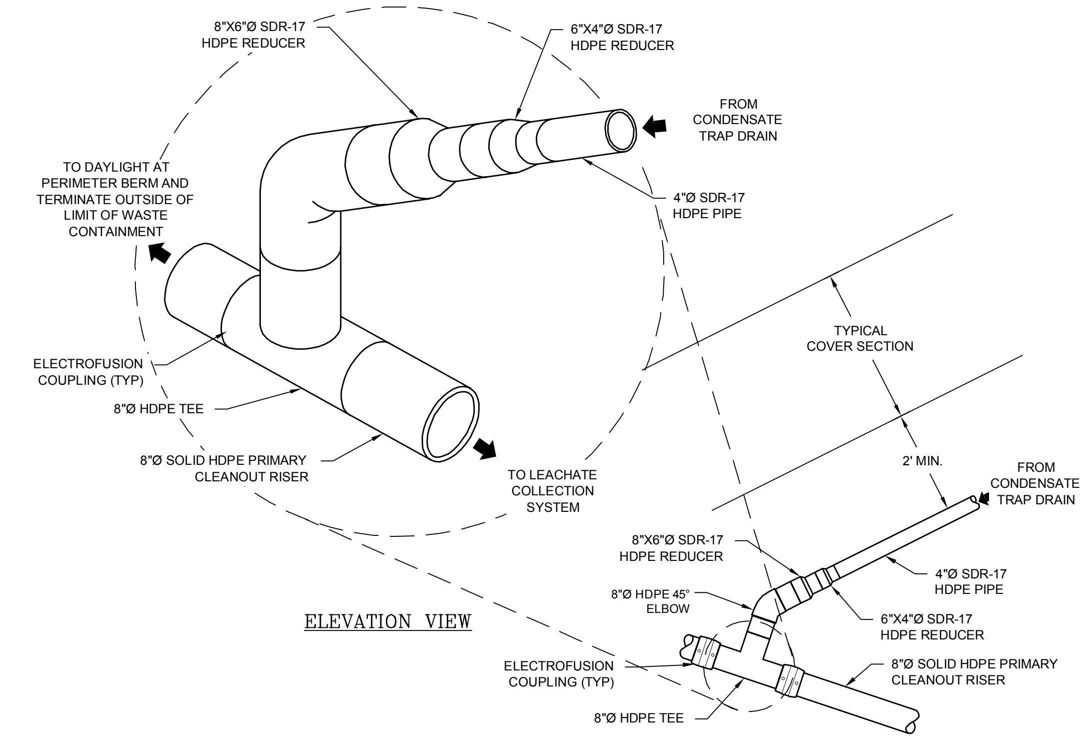
NOT TO SCALE



TYPICAL GAS HEADER PIPE TRENCH OUTSIDE LIMIT OF WASTE

2

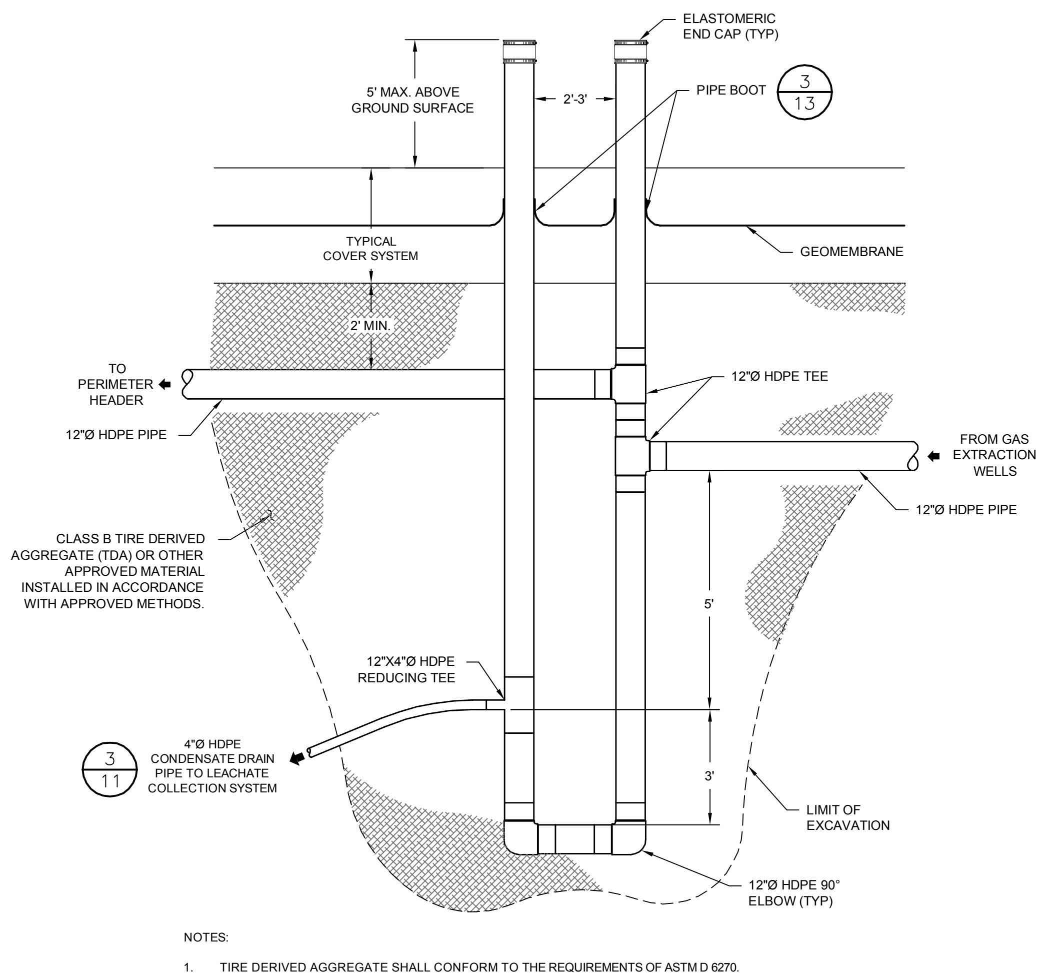
NOT TO SCALE



TYPICAL CLEANOUT TIE–IN DETAIL

3

NOT TO SCALE



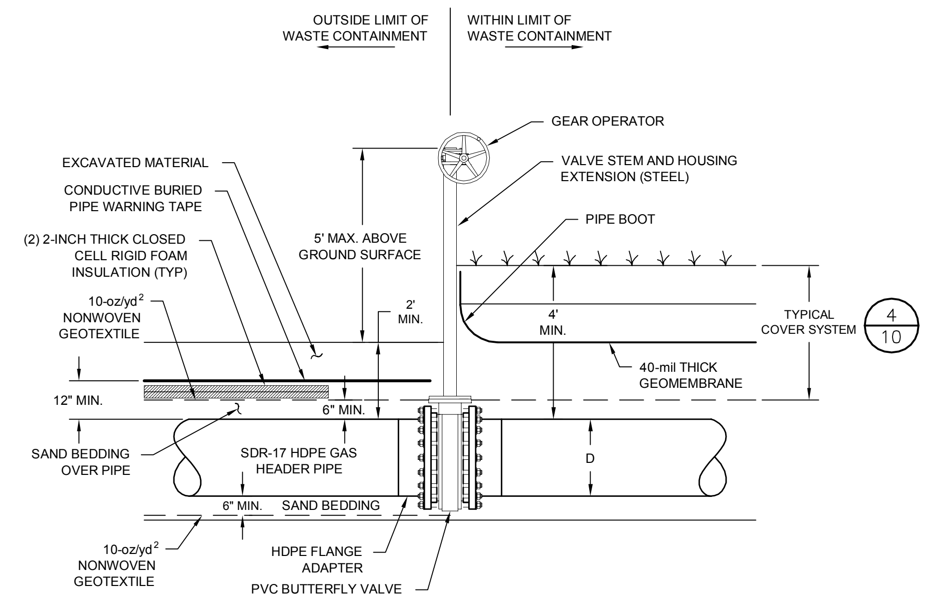
NOTES:

1. TIRE DERIVED AGGREGATE SHALL CONFORM TO THE REQUIREMENTS OF ASTM D 6270.

TYPICAL CONDENSATE TRAP

4

NOT TO SCALE



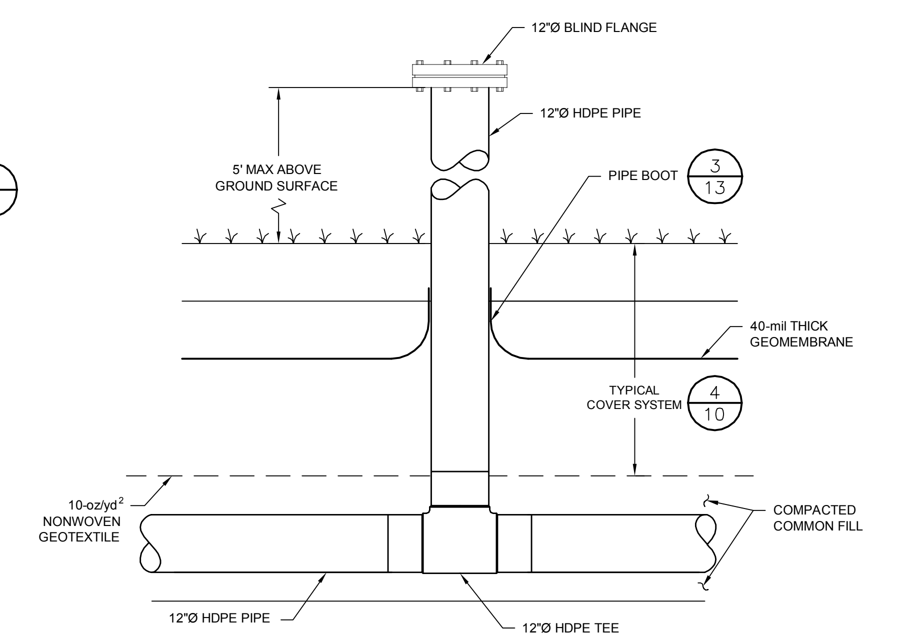
NOTE:

1. FLANGE BACKUP RINGS AND FASTENERS SHALL BE TYPE 316 STAINLESS STEEL.
2. PROTECT BELOW GRADE BOLTS AND FLANGES BY COVERING WITH A 6-MIL THICK POLYETHYLENE WRAP. DUCT TAPE WRAP TO HDPE PIPE.

CONTROL VALVE DETAIL (TYP)

5

NOT TO SCALE



VERTICAL RISER DETAIL (TYP)

6

NOT TO SCALE

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE Eric S. Steinhauser 10809 PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

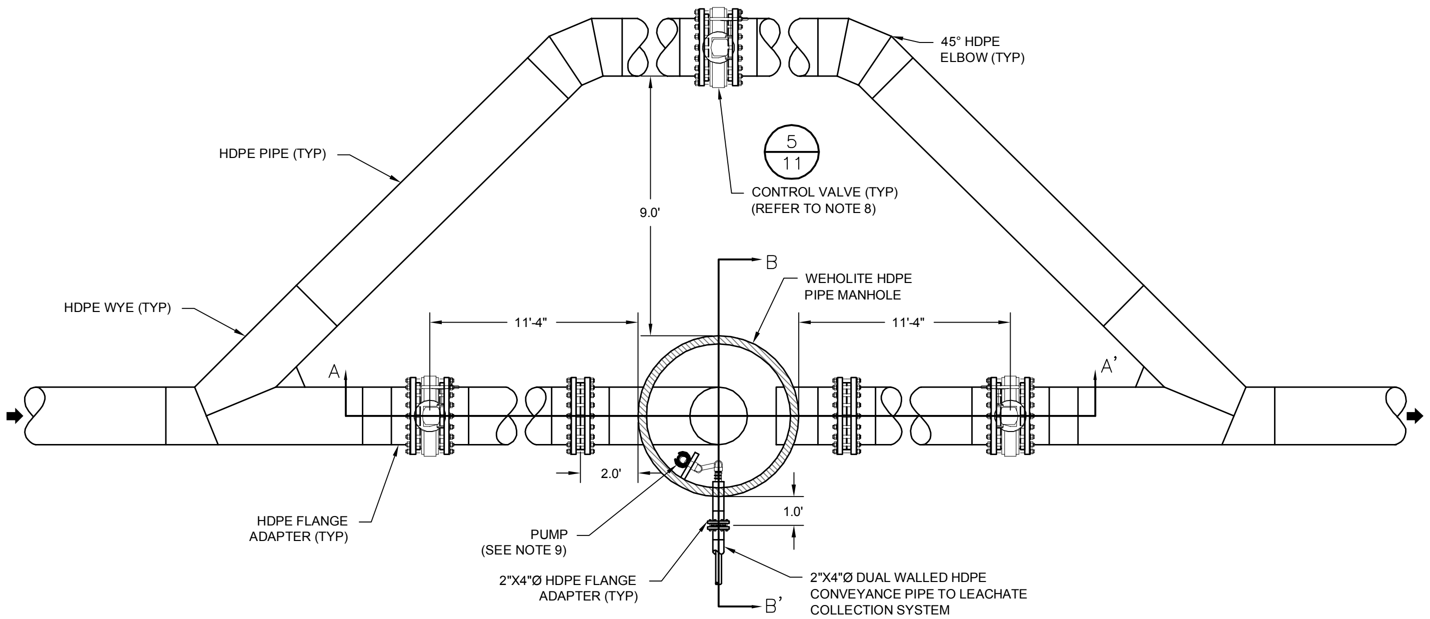
LANDFILL GAS EXTRACTION SYSTEM DETAILS

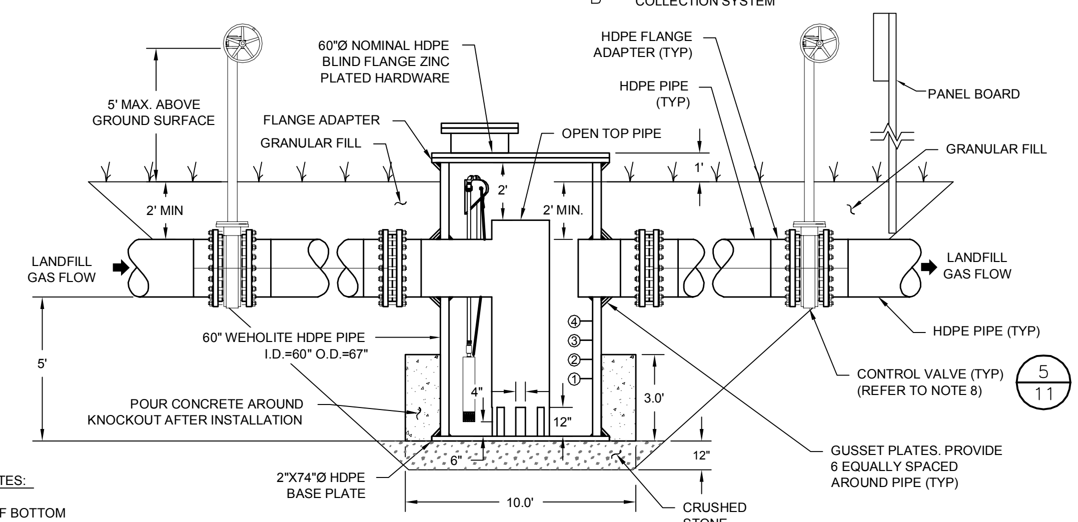
PROJECT NUMBER: 2536.27

SHEET NUMBER: 11 OF 14

NOTES:

1. REFER TO PREVIOUS SHEETS FOR ADDITIONAL NOTES.
2. ALL HDPE PIPE AND FITTINGS SHALL BE SDR-17 UNLESS OTHERWISE NOTED.
3. CONTRACTOR SHALL FOLLOW EXCAVATION PRACTICES AND REGULATIONS APPROVED BY OSHA REQUIREMENTS FOR PROTECTIVE SYSTEMS. - 1926.652.
4. CONDENSATE KNOCKOUT SHALL HAVE AN OSHA APPROVED CONFINED SPACE SIGN ATTACHED TO THE TOP OF THE COVER. SIGN SHALL BE 14" WIDE BY 10" HIGH, AND SHALL HAVE UV-RESISTANT PAINT ON AN ALUMINUM BASE. SIGN SHALL READ "DANGER CONFINED SPACE, HAZARDOUS ATMOSPHERE, ENTER BY PERMIT ONLY."
5. CONDENSATE PUMP SHALL BE GOULDS PUMP MODEL 1SC51C-1, AS SPECIFIED BY NEWSME LANDFILL OPERATIONS, LLC. OR EQUIVALENT.
6. CONTRACTOR SHALL INSTALL PUMP CONTROLS, ASSOCIATED CONTROL WIRING, AND ELECTRICAL POWER AS NEEDED TO OPERATE THE PUMP AND PROVIDE NECESSARY ALARMS.
7. CONDENSATE KNOCKOUT IS A CLASS 1 DIVISION 1 GROUP D CLASSIFIED SPACE. PROVIDE EXPLOSION-PROOF OR INTRINSICALLY-SAFE ELECTRICAL EQUIPMENT. PROVIDE CONDUIT SEALS ON ALL ELECTRICAL CONDUIT.
8. PROTECT BELOW GRADE BOLTS AND FLANGES BY COVERING WITH A 6-MIL THICK POLYETHYLENE WRAP. DUCT TAPE WRAP TO HDPE PIPE.

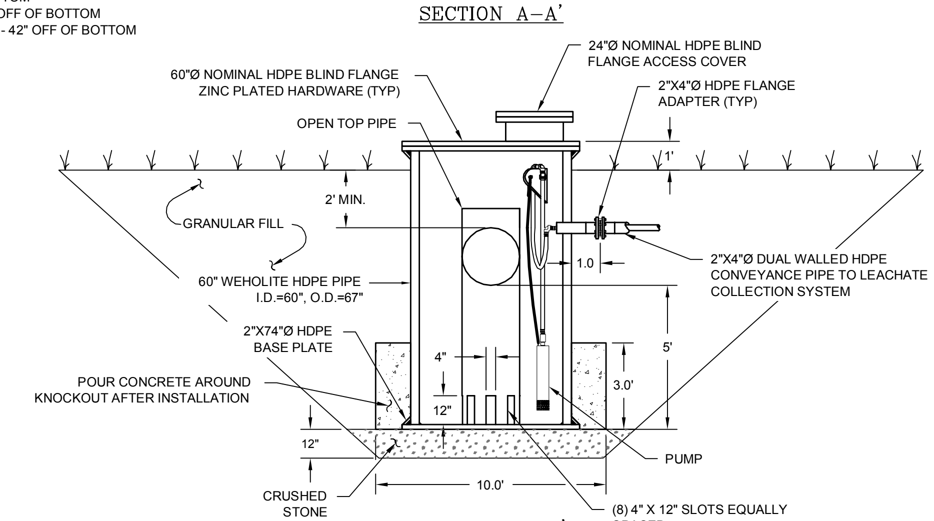




SECTION A–A'

LIQUID LEVEL CONTROL NOTES:

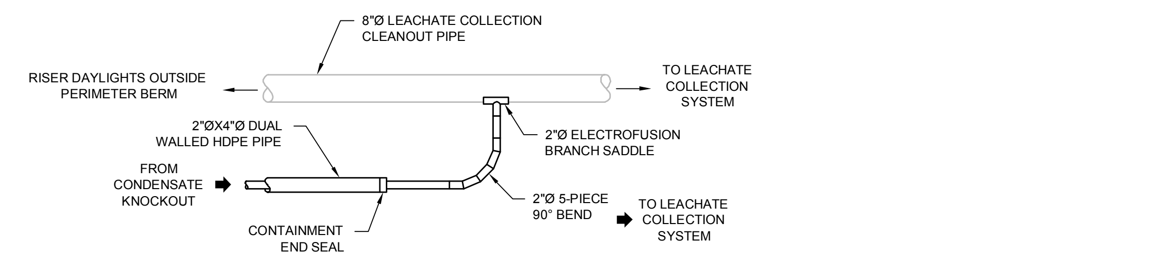
1. PUMP OFF - 18" OFF OF BOTTOM
2. PUMP ON - 30" OFF OF BOTTOM
3. HIGH WATER ALARM - 36" OFF OF BOTTOM
4. HIGH HIGH WATER ALARM - 42" OFF OF BOTTOM



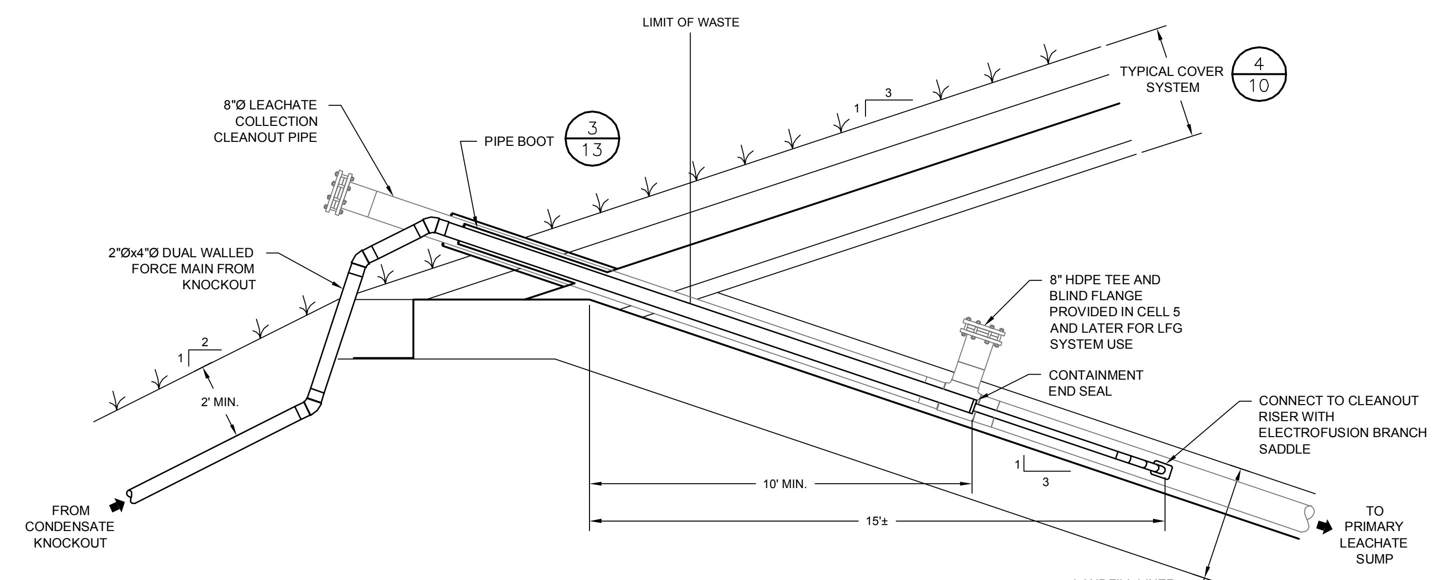
SECTION B–B'

1 CONDENSATE KNOCKOUT DETAIL

NOT TO SCALE



PLAN VIEW



ELEVATION VIEW

2 KNOCKOUT FORCE MAIN CONNECTION DETAIL

NOT TO SCALE

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE Eric S. Steinhauser 10809 PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	REMOVED NOTE 9.	RLC

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN

JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

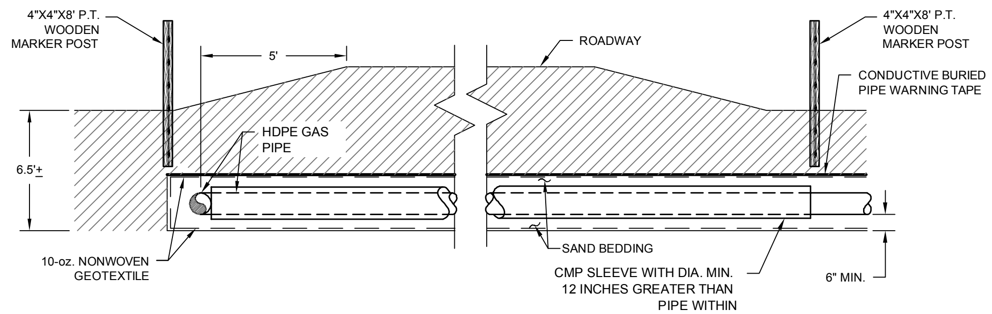
LANDFILL GAS EXTRACTION SYSTEM DETAILS

PROJECT NUMBER: 2536.27

SHEET NUMBER: 12 OF 14

NOTE:

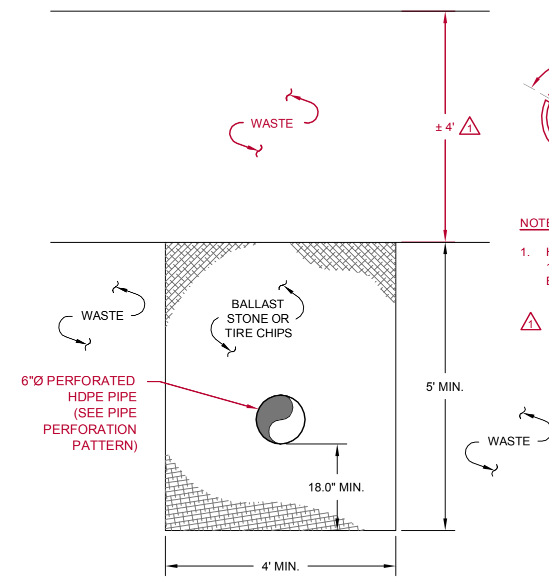
1. THESE DRAWINGS DO NOT SHOW ALL INTERIM LANDFILL ACCESS ROADS WHICH MAY BE CONSTRUCTED DURING FILLING OPERATIONS. THIS DETAIL APPLIES WHEN ROAD CROSSING CONDITIONS ARE PRESENT.



GAS PIPE ROAD CROSSING SLEEVE DETAIL

1

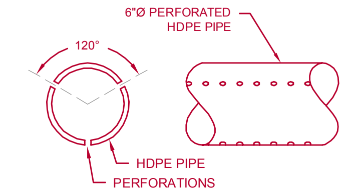
NOT TO SCALE



TYPICAL GAS COLLECTION TRENCH SECTION

2

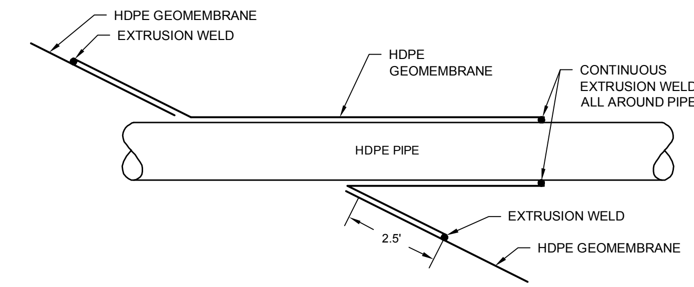
NOT TO SCALE



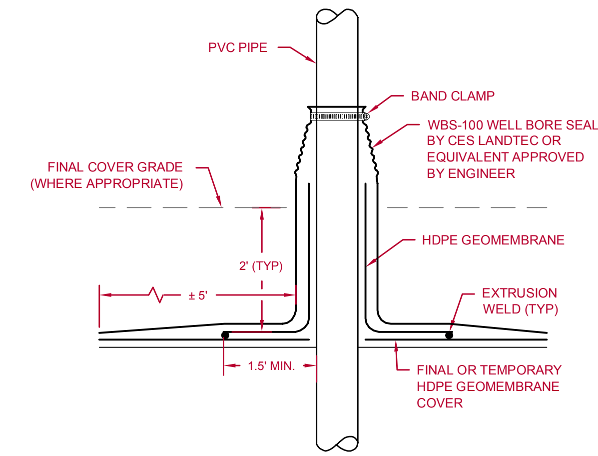
NOTE:

1. HOLES SHALL BE 1/2"Ø DRILLED HOLES SPACED 12" APART ALONG THE LENGTH OF THE PIPE OR EQUIVALENT APPROVED BY OWNER.

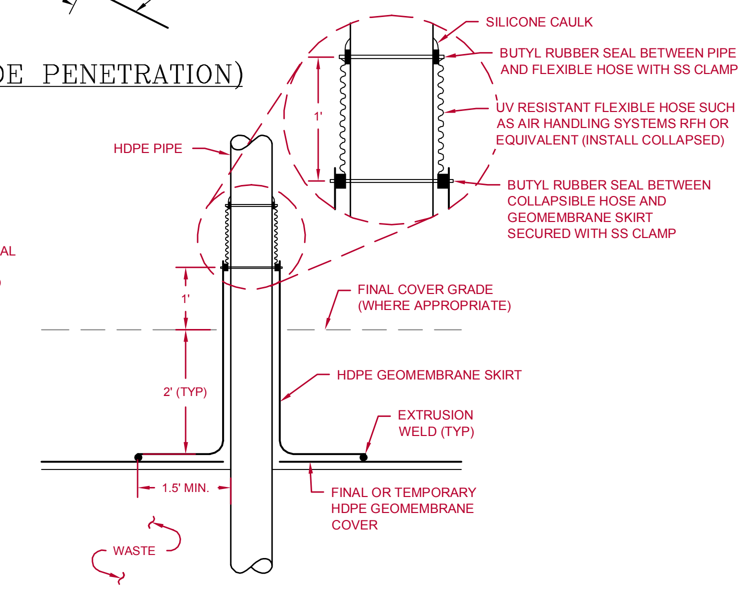
PIPE PERFORATION PATTERN



HDPE PIPE (SIDE PENETRATION)



PVC PIPE (RISER)

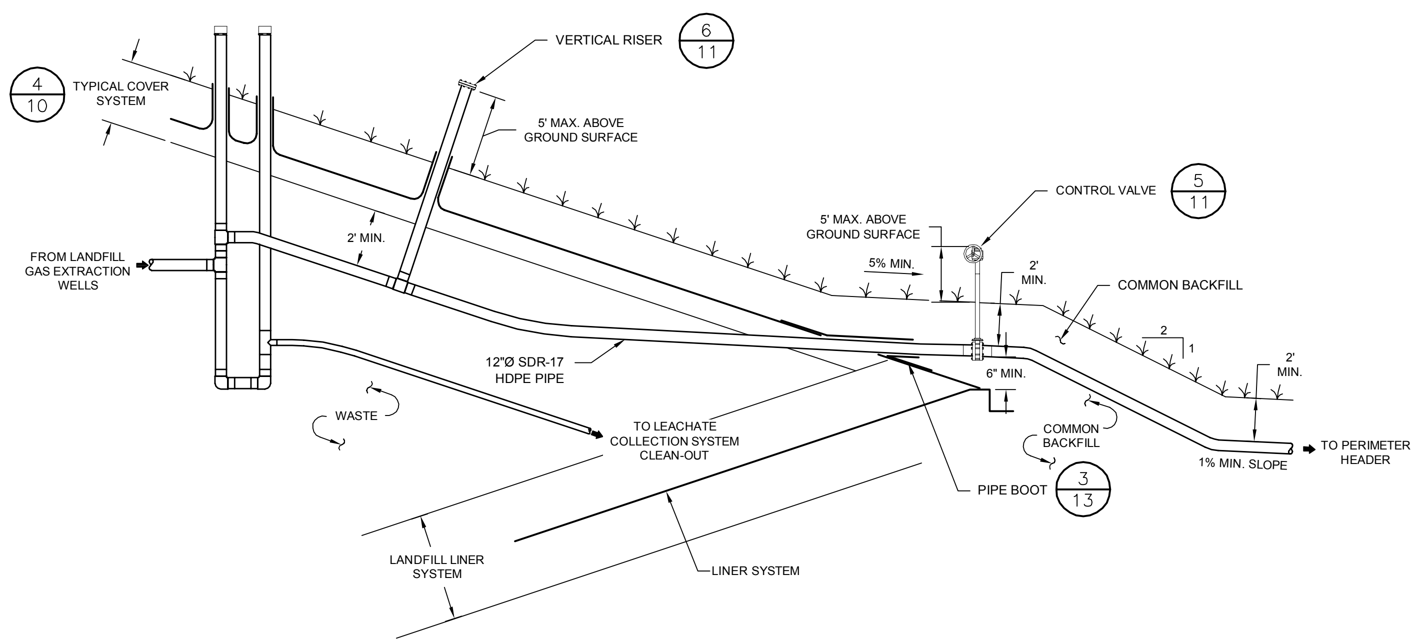


HDPE PIPE (RISER)

TYPICAL PIPE BOOT DETAIL

3

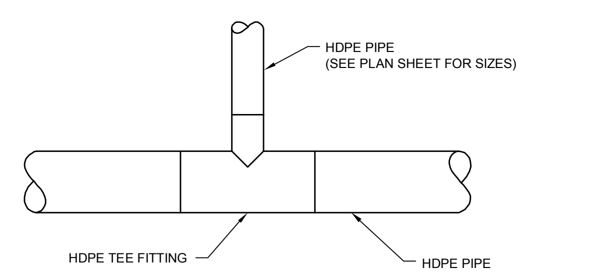
NOT TO SCALE



CAP PENETRATION DETAIL

4

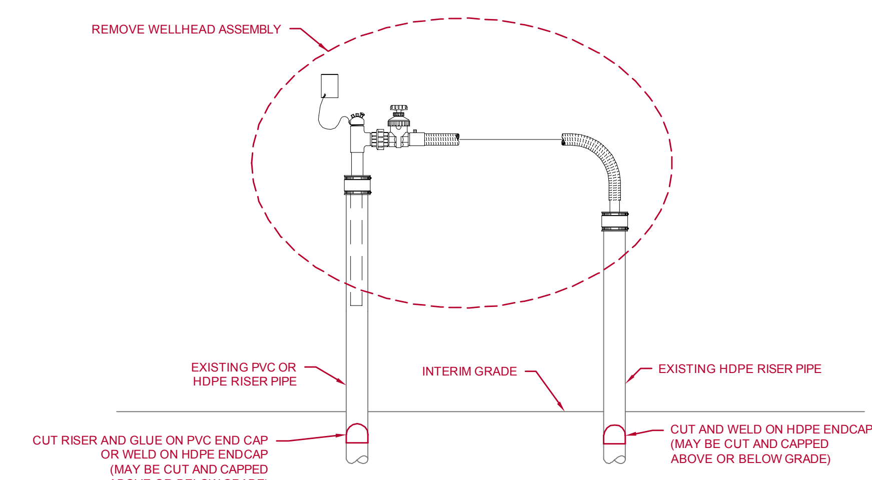
NOT TO SCALE



"TEE" CONNECTION DETAIL

5

NOT TO SCALE



LFG WELL DECOMMISIONING

6

NOT TO SCALE

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	02/25/16	REVISED BASED ON MEDEP COMMENTS.	RLC

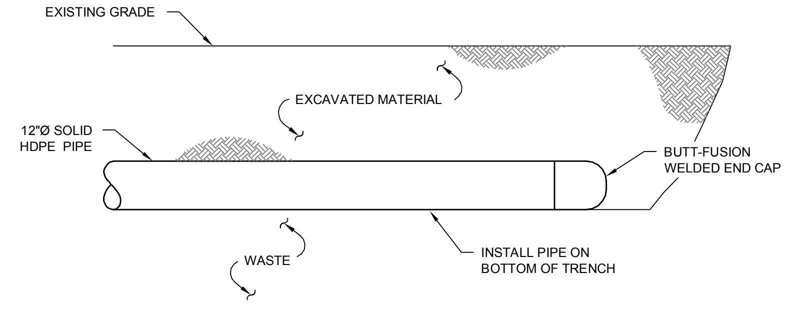
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LANDFILL GAS EXTRACTION SYSTEM DETAILS

PROJECT NUMBER: 2536.27

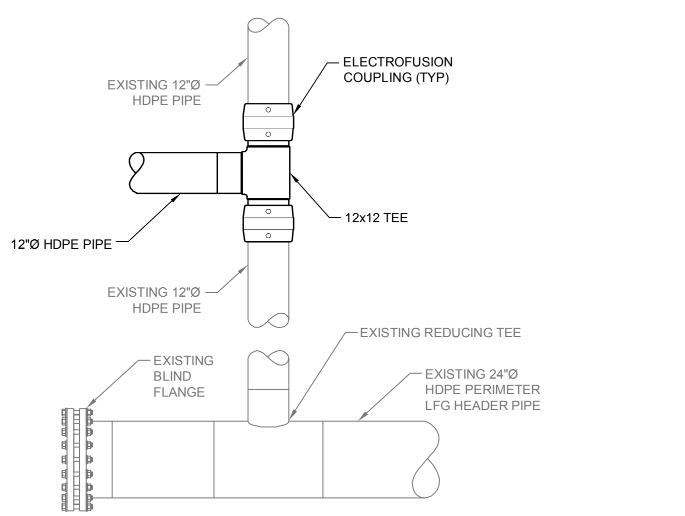
SHEET NUMBER: 13 OF 14



TEMPORARY LFG PIPE TERMINATION

1

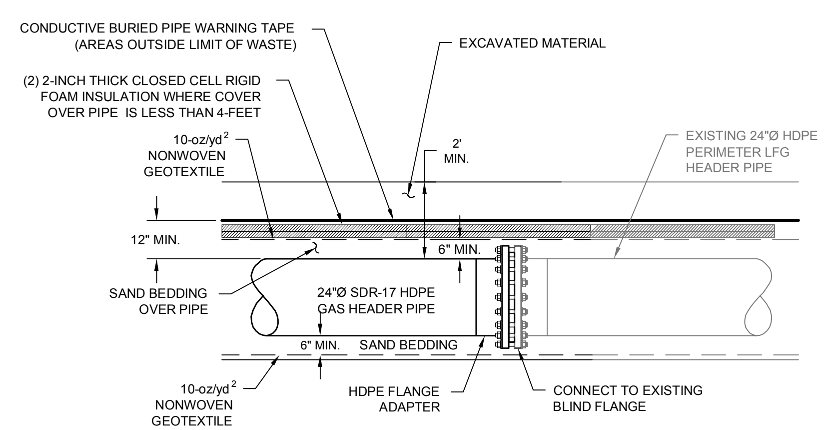
NOT TO SCALE



WEST LFG CONVEYANCE PIPE CONNECTION DETAIL

2

NOT TO SCALE



PERIMETER LFG CONVEYANCE PIPE CONNECTION DETAIL

3

NOT TO SCALE

LANDFILL GAS EXTRACTION WELL SCHEDULE

WELL DESIGNATION	NORTHING	EASTING	BOTTOM OF WASTE (FT)	TOP OF EXISTING WASTE (FT)	TOTAL WELL DEPTH (FT)	BOTTOM OF WELL SCREEN (FT)	TOP OF WELL SCREEN (FT)	SCREEN LENGTH (FT)	TOP OF CASING ELEV. (FT)
GW-001	479171.1	925548.7	203.1	271.3	53.2	218.1	256.3	38.2	274.3
GW-008A	479030.7	925503.0	196.1	256.9	45.7	211.1	241.9	30.7	259.9
GW-016	478859.9	925491.7	188.5	249.4	45.9	203.5	234.4	30.9	252.4
GW-020R	478878.2	926093.3	209.3	381.1	156.8	224.3	366.1	141.8	384.1
GW-022R	478887.0	926473.0	213.6	381.7	153.1	228.6	366.7	138.1	384.7
GW-023R	478928.7	926280.1	212.5	388.2	160.6	227.5	373.2	145.6	391.2
GW-026	478922.2	926825.3	217.1	349.1	116.9	232.1	334.1	101.9	352.1
GW-027	478701.0	925538.0	183.2	243.4	45.3	198.2	228.4	30.3	246.4
GW-028R	478695.3	925744.2	194.6	294.2	84.6	209.6	279.2	69.6	297.2
GW-031R	478784.3	926213.2	208.6	387.7	164.1	223.6	372.7	149.1	390.7
GW-032R	478753.5	926387.1	210.9	388.0	162.1	225.9	373.0	147.1	391.0
GW-033	478769.9	926542.9	214.8	380.4	150.6	229.8	365.4	135.6	383.4
GW-034	478771.5	926716.1	214.3	364.4	135.1	229.3	349.4	120.1	367.4
GW-035	478796.8	926909.2	217.6	339.1	106.6	232.6	324.1	91.6	342.1
GW-040	478595.2	926299.0	205.3	381.9	161.6	220.3	366.9	146.6	384.9
GW-041	478596.0	926470.3	209.4	387.5	163.2	224.4	372.5	148.2	390.5
GW-042	478629.0	926643.7	212.0	376.0	149.0	227.0	361.0	134.0	379.0
GW-043	478630.6	926816.9	213.5	359.6	131.1	228.5	344.6	116.1	362.6
GW-044	478631.1	926990.1	215.3	332.2	101.8	230.3	317.2	86.8	335.2
GW-045	478421.8	925756.4	177.0	237.0	45.0	192.0	222.0	30.0	240.0
GW-049	478461.3	926386.1	205.7	384.7	164.0	220.7	369.7	149.0	387.7
GW-050	478478.2	926558.5	214.0	383.1	154.1	229.0	368.1	139.1	386.1
GW-051	478479.8	926731.7	210.8	372.7	146.8	225.8	357.7	131.8	375.7
GW-052	478480.9	926903.9	214.7	356.4	126.7	229.7	341.4	111.7	359.4
GW-053	478482.6	927079.8	214.5	320.6	91.1	229.5	305.6	76.1	323.6
GW-054	478263.7	925872.0	176.7	237.0	45.2	191.7	222.0	30.2	240.0
GW-057R	478304.0	926324.8	196.4	365.8	154.4	211.4	350.8	139.4	368.8
GW-058	478327.4	926473.3	202.0	380.0	163.0	217.0	365.0	148.0	383.0
GW-059	478329.1	926646.5	209.0	380.5	156.5	224.0	365.5	141.5	383.5
GW-060	478330.7	926819.7	209.7	369.4	144.7	224.7	354.4	129.7	372.4
GW-061	478332.7	926992.9	211.8	353.0	126.1	226.8	338.0	111.1	356.0
GW-062	478326.0	927153.1	213.2	315.0	86.8	228.2	300.0	71.8	318.0
GW-063	478063.7	925861.2	170.3	231.9	46.6	185.3	216.9	31.6	234.9
GW-067	478173.5	926387.8	195.8	352.2	141.4	210.8	337.2	126.4	355.2
GW-068	478178.0	926560.9	201.2	375.9	159.7	216.2	360.9	144.7	378.9
GW-069	478180.1	926734.1	203.8	374.7	155.9	218.8	359.7	140.9	377.7
GW-070	478181.3	926907.3	207.5	367.6	145.1	222.5	352.6	130.1	370.6
GW-071	478181.5	927080.5	210.5	345.0	119.4	225.5	330.0	104.4	348.0
GW-072	478167.1	927233.0	212.3	293.9	66.6	227.3	278.9	51.6	296.9
GW-073	477967.4	925985.9	171.0	231.8	45.8	186.0	216.8	30.8	234.8
GW-074R	477956.4	926103.4	173.9	239.0	50.1	188.9	224.0	35.1	242.0
GW-075	477947.9	926282.0	180.2	278.1	82.9	195.2	263.1	67.9	281.1
GW-076	478015.5	926458.8	189.9	338.7	133.9	204.9	323.7	118.9	341.7
GW-077	478029.5	926650.0	196.6	340.9	129.3	211.6	325.9	114.3	343.9
GW-078	478029.8	926823.2	201.3	351.6	135.2	216.3	336.6	120.2	354.6
GW-079	478030.1	926996.4	205.9	360.0	139.1	220.9	345.0	124.1	363.0
GW-080	478033.0	927169.6	209.4	322.7	98.2	224.4	307.7	83.2	325.7
GW-081	478038.0	927346.9	216.1	271.7	40.6	231.1	256.7	25.6	274.7
GW-084	477878.6	926564.9	186.9	294.4	92.5	201.9	279.4	77.5	297.4
GW-085	477879.7	926736.9	193.1	295.4	87.2	208.1	280.4	72.2	298.4
GW-086	477851.7	926905.1	197.1	303.0	91.0	212.1	288.0	76.0	306.0
GW-087	477860.0	927085.1	202.7	304.0	86.4	217.7	289.0	71.4	307.0
GW-088	477884.2	927267.3	208.5	306.3	82.8	223.5	291.3	67.8	309.3
GW-089	477891.8	927440.3	211.8	262.8	36.0	226.8	247.8	21.0	265.8
GW-092R	477740.0	926659.1	184.4	245.0	45.7	199.4	230.0	30.7	248.0
GW-094	477720.1	927014.1	196.6	257.9	46.3	211.6	242.9	31.3	260.9
GW-095	477704.4	927189.8	201.7	251.1	34.4	216.7	236.1	19.4	254.1
GW-096	477730.2	927361.0	207.7	257.0	34.2	222.7	242.0	19.2	260.0
GW-097	477752.2	927529.6	211.3	250.9	24.7	226.3	235.9	9.7	253.9
GW-098	478459.4	927212.4	220.9	281.5	45.6	235.9	266.5	30.6	284.5
GW-099	478630.8	927140.8	223.9	284.9	46.0	238.9	269.9	31.0	287.9
GW-100	478786.2	927088.7	222.6	283.9	46.3	237.6	268.9	31.3	286.9
GW-101	478941.9	927028.9	221.8	285.4	48.6	236.8	270.4	33.6	288.4
GW-102	479083.5	926986.5	221.0	282.9	46.8	236.0	267.9	31.8	285.9
GW-103	479082.4	926861.4	225.4	322.1	81.7	240.4	307.1	66.7	325.1
GW-104	479246.4	926939.8	217.0	278.4	46.4	232.0	263.4	31.4	281.4
GW-105	479243.1	926784.1	222.7	327.4	89.7	237.7	312.4	74.7	330.4
GW-106	479401.5	926874.9	213.8	280.5	51.7	228.8	265.5	36.7	283.5
GW-107	479391.3	926703.4	221.5	335.2	98.7	236.5	320.2	83.7	338.2
GW-108	479532.6	926808.3	211.0	285.9	59.9	226.0	270.9	44.9	288.9
GW-109	479517.1	926628.7	221.8	343.8	107.0	236.8	328.8	92.0	346.8
GW-110	479679.4	926660.7	209.0	314.1	90.0	224.0	299.1	75.0	317.1
GW-111	479685.6	926802.0	206.1	269.6	48.4	221.1	254.6	33.4	272.6

LANDFILL GAS EXTRACTION WELL SCHEDULE

WELL DESIGNATION	NORTHING	EASTING	BOTTOM OF WASTE (FT)	TOP OF EXISTING WASTE (FT)	TOTAL WELL DEPTH (FT)	BOTTOM OF WELL SCREEN (FT)	TOP OF WELL SCREEN (FT)	SCREEN LENGTH (FT)	TOP OF CASING ELEV. (FT)
GW-112	479814.8	926768.1	203.2	264.1	45.9	218.2	249.1	30.9	267.1
GW-113	479970.9	926695.1	200.0	267.4	52.4	215.0	252.4	37.4	270.4
GW-114	480098.0	926675.4	196.5	257.8	46.3	211.5	242.8	31.3	260.8
GW-115	480253.9	926625.6	192.2	254.1	46.9	207.2	239.1	31.9	257.1
GW-116	480438.0	926567.9	186.2	249.0	47.8	201.2	234.0	32.8	252.0
GW-117	479829.0	926585.2	206.0	319.0	98.0	221.0	304.0	83.0	322.0
GW-118	480130.3	926566.5	197.6	287.5	75.0	212.6	272.5	60.0	290.5
GW-119	479982.2	926510.9	203.0	323.0	105.1	218.0	308.0	90.1	326.0
GW-120	479663.7	926509.6	212.5	352.4	124.9	227.5	337.4	109.9	355.4
GW-121	480281.2	926478.0	194.3	295.9	86.6	209.3	280.9	71.6	298.9
GW-122	480417.9	926402.8	191.2	252.4	46.1	206.2	237.4	31.1	255.4
GW-123	480127.8	926404.4	200.6	338.1	122.5	215.6	323.1	107.5	341.1
GW-124	479822.2	926424.4	209.3	354.5	130.2	224.3	339.5	115.2	357.5
GW-125	479507.4	926436.3	217.6	365.0	132.5	232.6	350.0	117.5	368.0
GW-126	480251.9	926322.4	198.4	303.1	89.8	213.4	288.1	74.8	306.1
GW-127	479963.3	926322.7	206.8	358.7	136.9	221.8	343.7	121.9	361.7
GW-128	479654.0	926339.8	214.8	368.6	138.7	229.8	353.6	123.7	371.6
GW-129	479368.9	926330.6	219.7	379.9	145.1	234.7	364.9	130.1	382.9
GW-130	480377.6	926246.4	195.9	258.0	47.0	210.9	243.0	32.0	261.0
GW-131	480099.9	926234.5	204.5	348.3	128.8	219.5	333.3	113.8	351.3
GW-132	479810.3	926234.3	213.1	372.5	144.4	228.1	357.5	129.4	375.5
GW-133	479488.4	926248.7	219.1	381.6	147.5	234.1	366.6	132.5	384.6
GW-134	480347.0	926098.0	199.7	260.8	46.1	214.7	245.8	31.1	263.8
GW-135	480210.8	926170.1	202.5	309.2	91.8	217.5	294.2	76.8	312.2
GW-136	479979.6	926132.6	210.3	358.7	133.3	225.3	343.7	118.3	361.7
GW-137	479635.0	926172.9	217.0	382.0	150.0	232.0	367.0	135.0	385.0
GW-138	479308.9	926161.6	219.6	383.0	148.5	234.6	368.0	133.5	386.0
GW-139	480146.5	926045.6	206.6	324.4	102.8	221.6	309.4	87.8	327.4
GW-140	479789.5	926075.3	215.2	376.7	146.4	230.2	361.7	131.4	379.7
GW-141	479467.3	926073.2	221.0	380.3	144.3	236.0	365.3	129.3	383.3
GW-142	480302.2	925958.1	203.6	268.7	50.0	218.6	253.7	35.0	271.7
GW-143	479956.4	925978.3	213.4	358.7	130.3	228.4	343.7	115.3	361.7
GW-144	479635.4	925981.6	219.6	377.6	143.0	234.6	362.6	128.0	380.6
GW-145	479298.3	925980.7	218.8	368.0	134.2	233.8	353.0	119.2	371.0
GW-146	480116.8	925882.5	211.1	326.2	100.1	226.1	311.2	85.1	329.2
GW-147	479796.3	925890.5	217.1	372.0	140.0	232.1	357.0	125.0	375.0
GW-148	479469.9	925913.1	217.2	365.4	133.2	232.2	350.4	118.2	368.4
GW-149	480261.9	925796.2	207.7	273.9	51.2	222.7	258.9	36.2	276.9
GW-150	479945.9	925798.4	213.4	356.9	128.5	228.4	341.9	113.5	359.9
GW-151	479633.7	925823.4	212.9	363.1	135.2	227.9	348.1	120.2	366.1
GW-152	479285.6	925835.4	215.1	353.4	123.3	230.1	338.4	108.3	356.4
GW-153	480099.6	925705.7	212.2	320.9	93.6	227.2	305.9	78.6	323.9
GW-154	480205.6	925618.7	208.3	279.4	56.1	223.3	264.4	41.1	282.4
GW-155	479787.3	925702.6	211.1	357.1	131.0	226.1	342.1	116.0	360.1
GW-156	479445.8	925762.8	212.9	350.5	122.6	227.9	335.5	107.6	353.5
GW-157	479618.4	925688.2	209.9	350.4	125.5	224.9	335.4	110.5	353.4
GW-158	479935.4	925623.3	209.1	350.9	126.9	224.1	335.9	111.9	353.9
GW-159	480086.8	925535.4	208.4	310.8	87.3	223.4	295.8	72.3	313.8
GW-160	479289.6	925691.2	211.1	318.4	92.3	226.1	303.4	77.3	321.4
GW-161	479449.9	925642.2	210.6	319.0	93.4	225.6	304.0	78.4	322.0
GW-162	479624.3	925565.0	207.6	317.9	95.3	222.6	302.9	80.3	320.9
GW-163	479777.3	925514.4	207.2	316.8	94.5	222.2	301.8	79.5	319.8
GW-164	479939.8	925461.4	205.7	313.8	93.2	220.7	298.8	78.2	316.8
GW-165	480194.7	925443.5	204.6	268.2	48.6	219.6	253.2	33.6	271.2
GW-166	480063.5	925381.7	205.2	298.4	78.2	220.2	283.4	63.2	301.4
GW-167	480154.6	925257.7	200.8	262.9	47.1	215.8	247.9	32.1	265.9
GW-168	479923.1	925313.5	202.2	264.8	47.6	217.2	249.8	32.6	267.8
GW-169	479737.1	925369.4	203.7	266.6	47.9	218.7	251.6	32.9	269.6
GW-170	479606.9	925413.5	204.4	268.4	49.0	219.4	253.4	34.0	271.4
GW-171	479447.5	925491.2	207.5	271.6	49.1	222.5	256.6	34.1	274.6
GW-172	479308.8	925538.8	207.2	270.5	48.3	222.2	255.5	33.3	273.5

NOTES:

1. LFG WELLS GW-001 THROUGH GW-097 WERE TAKEN FROM PREVIOUS LFG DESIGN DRAWINGS PREPARED BY SANBORN HEAD IN OCTOBER 2003 AND REVISED IN SEPTEMBER 2010. WELLS ALREADY INSTALLED AT THE SITE ACCOUNT FOR GAPS IN NUMBERING.
2. LFG EXTRACTION WELLS SHALL BE INSTALLED WITHIN ONE FOOT OF LISTED LOCATIONS
3. A TEMPORARY BENCHMARK WITH ELEVATION SHALL BE ESTABLISHED AT EACH WELL PRIOR TO DRILLING.
4. 15 FEET OF SOLID RISER IS TO BE PROVIDED BELOW INTERMEDIATE COVER GRADES. THE INTENT IS TO PROVIDE 3 FEET OF STICK UP ABOVE FILL GRADES.
5. ELEVATIONS SHALL BE CONFIRMED AGAINST AS-BUILT TOP OF PRIMARY SAND GRADES AND FILL GRADES PRIOR TO CONSTRUCTION.

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: JUNE 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LANDFILL GAS EXTRACTION SYSTEM DETAILS

PROJECT NUMBER: 2536.27

SHEET NUMBER: 14 OF 14

SH-4

VOLUME III - APPENDIX I
HEADER SIZING CALCULATION
(REVISED FEBRUARY 2016)

PURPOSE: Estimate the internal gas velocity and pressure loss that can be expected for a 24-inch diameter landfill gas (LFG) header pipe proposed as part of the expansion to the LFG system at the Juniper Ridge Landfill (JRL) in Old Town, Maine. Compare these values with industry accepted values to justify pipe sizing.

METHOD: LFG generation modeling (performed separately using LandGEM) suggests that a future total LFG flow rate of 4,500 scfm may be expected in the proposed 24-inch perimeter header pipe located in the proposed east perimeter berm. Check to see if the velocity of LFG, V, is no greater than 20 ft/s for counter-current gas and condensate flow or 40 ft/s for concurrent gas and condensate flow [REF. 1] for the total LFG flow rate estimated by LandGEM. Next, use the Low-Pressure Mueller Equation to estimate pipe pressure losses. Check to see if Pressure Loss, ΔP, is less than 1 inch per 100 feet of pipe or 0.01 in/ft.

GIVEN:

- Total LFG flow rate from LandGEM = 4,500 scfm
- Internal diameter of 24-inch diameter SDR-17 HDPE pipe = 21.007 in.
- Cross-sectional area of 24-inch diameter SDR-17 HDPE pipe = 2.407 ft^2
- Approximate length of proposed header pipe = 3,330 feet

CALCULATION:

Velocity of the gas inside pipe, V:

$$V = \frac{Q}{A}$$

where V = LFG Velocity, feet per second
Q = LFG flow rate, cubic feet per second
A= Cross sectional area of the pipe, square feet
where $A = \pi r^2$ and $r =$ radius of the pipe, feet

$$V = \left((4{,}500\ \text{scfm})\left(\frac{1\ \text{min}}{60\ \text{sec}}\right)\right) \div 2.407\ \text{ft}^2 = 31.2\ \text{ft/s}$$

Check: $31.2\ \text{ft/s} < 40\text{ft/s}$ OKAY

Friction Losses in Pipe: Low-Pressure Mueller Equation [REF 1]:

$$\Delta P = L\left[\frac{60 \cdot Q \cdot G^{0.425}}{2971 \cdot d^{2.725}}\right]^{1.739}$$

Where: $\Delta P =$ pressure loss, inches of water column

L = length of pipe, ft.
Q = gas flow rate, ft^3/min.
G = specific gravity of the gas, use 0.81 for landfill gas
d = internal pipe diameter, in.

$$\Delta P = 3{,}330 \text{ ft} \times \left[\frac{60 \cdot (4{,}500 \text{ scfm}) \cdot (0.81)^{0.425}}{2971 \cdot (21.007 \text{ in})^{2.725}}\right]^{1.739}$$

$$\Delta P = 3.93 \text{ in W.C.}$$

$$\text{Check:} \quad \frac{\Delta P}{L} < \frac{0.01 \text{ in}}{\text{ft}}$$

$$\frac{3.93 \text{ in}}{3{,}330 \text{ ft}} = \frac{0.00118 \text{ in}}{1 \text{ ft}} < \frac{0.01 \text{ in}}{1 \text{ ft}} \qquad \text{OKAY}$$

RESULTS:

This calculation demonstrates that the 24-inch diameter pipe meets the above criteria for velocity (concurrent flow) and pressure loss. A section of this pipe also experiences counter-current flow. Spreadsheet calculations, using the methods detailed in this sample calculation, were performed for the counter-current flow portion of 24-inch diameter pipe as well as each of the primary LFG header pipes (numbered 1 thru 8) designed as part of the LFG system expansion. The calculations are included as Attachment A and the results of which are shown in Table 1.

Table 1

Header #	Projected LFG Flow (scfm)*	Pipe Diameter (in)	Pipe Length (ft)	LFG Velocity (ft/s)	Pressure Loss (in W.C.)	Pressure Loss less than 1in/100ft?	Comments
24-inch (Concurrent)	4,500	24	3,330	31.2	3.93	Yes	None
24-inch (Counter-Current)	2,250	24	1,700	15.6	0.6	Yes	None
1	1,175	12	2,130	28.8	4.87	Yes	None
2	969	12	970	23.8	1.59	Yes	None
3	1,410	12	2,400	34.6	7.53	Yes	None
4	426	12	1,070	10.5	0.42	Yes	Pipe has capacity to handle more flow. **
5	1,031	12	1,375	25.3	2.5	Yes	Flow is split in two directions.***
6	893	12	1,175	21.9	1.67	Yes	Flow is split in two directions.***
7	955	12	1,870	23.4	2.98	Yes	None
8	333	12	760	8.2	0.19	Yes	Pipe has capacity to handle more flow.

*Estimated LFG flow rate projections for each header were calculated using LandGEM and are based on waste volumes calculated using AutoCAD (Attachment B).
**12-inch diameter LFG conveyance pipe is the largest pipe size used inside the limit of waste and was selected for convenience of construction, operation, and maintenance.
***Header pipes 5 and 6 meet flow criteria under normal operating conditions when flow is split in two directions (i.e. split flow).

REFERENCES:

1. *Geotechnical Aspects of Landfill Design and Construction*, Xuede Qian, Robert M. Koerner, Donald H. Gray, Prentice Hall, Upper Saddle River, NJ, 2002, pp. 370-373.

Attachment A

File Number:	2536.27
Project:	JRL LFG Master Plan
Location:	Juniper Ridge Landfill, Old Town, Maine
Subject:	Pipe Sizing Calculations
Calculated By:	R. Clay — Date: 4/3/2015
Checked By:	E. Steinhauser — Date: 4/20/2015
Header:	**24-inch (from Sample Calc)**

Header Information

Nominal Diameter	24	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	3,330	ft
Inside Diameter	21.007	in
Flow Area	2.407	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	4,500	cfm		
Velocity	31.2	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	3.93	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	1.72	gal/min.
	2,472	gal/day
	902,390	gal/year

Attachment A

File Number:	2536.27		
Project:	JRL LFG Master Plan		
Location:	Juniper Ridge Landfill, Old Town, Maine		
Subject:	Pipe Sizing Calculations		
Calculated By:	R. Clay	Date:	2/1/2016
Checked By:	E. Steinhauser	Date:	2/1/2016
Header:	**24-inch (Counter-Current)**		

Header Information

Nominal Diameter	24	in	
SDR-	17		
Condensate Flow	Counter-Current		
Approximate Pipe Length	1,700	ft	<---Approx. length of pipe under counter-current flow conditions
Inside Diameter	21.007	in	
Flow Area	2.407	ft^2	

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	2,250	cfm	<---Assumed 50% of total gas generation (4,500 scfm)	
Velocity	15.6	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	0.60	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.86	gal/min.
	1,236	gal/day
	451,195	gal/year

Attachment A

File Number:	2536.27
Project:	JRL LFG Master Plan
Location:	Juniper Ridge Landfill, Old Town, Maine
Subject:	Pipe Sizing Calculations
Calculated By:	R. Clay — Date: 4/3/2015
Checked By:	E. Steinhauser — Date: 4/20/2015
Header:	**Header 1**

Header Information

Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	2,130	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	1,175	cfm		
Velocity	28.8	ft/s	ACCEPTABLE	$V = Q/A$
Pressure Loss	4.87	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.45	gal/min.
	646	gal/day
	235,624	gal/year

Attachment A

File Number: 2536.27
Project: JRL LFG Master Plan
Location: Juniper Ridge Landfill, Old Town, Maine
Subject: Pipe Sizing Calculations
Calculated By: R. Clay   Date: 4/3/2015
Checked By: E. Steinhauser   Date: 4/20/2015
Header: **Header 2**

Header Information

Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	970	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	969	cfm		
Velocity	23.8	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	1.59	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.37	gal/min.
	532	gal/day
	194,315	gal/year

Attachment A

File Number: 2536.27
Project: JRL LFG Master Plan
Location: Juniper Ridge Landfill, Old Town, Maine
Subject: Pipe Sizing Calculations
Calculated By: R. Clay   Date: 4/3/2015
Checked By: E. Steinhauser   Date: 4/20/2015
Header: **Header 3**

Header Information

Nominal Diameter	12	in		
SDR-	17			
Condensate Flow	Concurrent			
Approximate Pipe Length	2,400	ft		
Inside Diameter	11.160	in		
Flow Area	0.679	ft^2		

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	1,410	cfm		
Velocity	34.6	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	7.53	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.54	gal/min.
	775	gal/day
	282,749	gal/year

Attachment A

File Number:	2536.27
Project:	JRL LFG Master Plan
Location:	Juniper Ridge Landfill, Old Town, Maine
Subject:	Pipe Sizing Calculations
Calculated By:	R. Clay — Date: 4/3/2015
Checked By:	E. Steinhauser — Date: 4/20/2015
Header:	**Header 4**

Header Information

Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	1,070	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	426	cfm		
Velocity	10.5	ft/s	ACCEPTABLE	$V = Q/A$
Pressure Loss	0.42	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.16	gal/min.
	234	gal/day
	85,426	gal/year

Attachment A

File Number:	2536.27
Project:	JRL LFG Master Plan
Location:	Juniper Ridge Landfill, Old Town, Maine
Subject:	Pipe Sizing Calculations
Calculated By:	R. Clay — Date: 2/24/2016
Checked By:	E. Steinhauser — Date: 2/24/2016
Header:	**Header 5**

Header Information

Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	1,375	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	1,031	cfm	<---Under normal operating conditions (flow is split in two directions.)	
Velocity	25.3	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	2.50	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.39	gal/min.
	566	gal/day
	206,748	gal/year

Attachment A

File Number:	2536.27		
Project:	JRL LFG Master Plan		
Location:	Juniper Ridge Landfill, Old Town, Maine		
Subject:	Pipe Sizing Calculations		
Calculated By:	R. Clay	Date:	2/24/2016
Checked By:	E. Steinhauser	Date:	2/24/2016
Header:	**Header 6**		

Header Information

Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	1,175	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	893	cfm	<---Under normal operating conditions (flow is split in two directions.)	
Velocity	21.9	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	1.67	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.34	gal/min.
	490	gal/day
	178,974	gal/year

Attachment A

File Number:	2536.27
Project:	JRL LFG Master Plan
Location:	Juniper Ridge Landfill, Old Town, Maine
Subject:	Pipe Sizing Calculations
Calculated By:	R. Clay — Date: 4/3/2015
Checked By:	E. Steinhauser — Date: 4/20/2015
Header:	**Header 7**

Header Information

Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	1,870	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	955	cfm		
Velocity	23.4	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	2.98	in WC	ACCEPTABLE	$\Delta P = L*[(60*Q*G^{0.425})/(2971*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.36	gal/min.
	525	gal/day
	191,507	gal/year

Attachment A

File Number: 2536.27
Project: JRL LFG Master Plan
Location: Juniper Ridge Landfill, Old Town, Maine
Subject: Pipe Sizing Calculations
Calculated By: R. Clay   Date: 4/3/2015
Checked By: E. Steinhauser   Date: 4/20/2015
Header: **Header 8**

Header Information

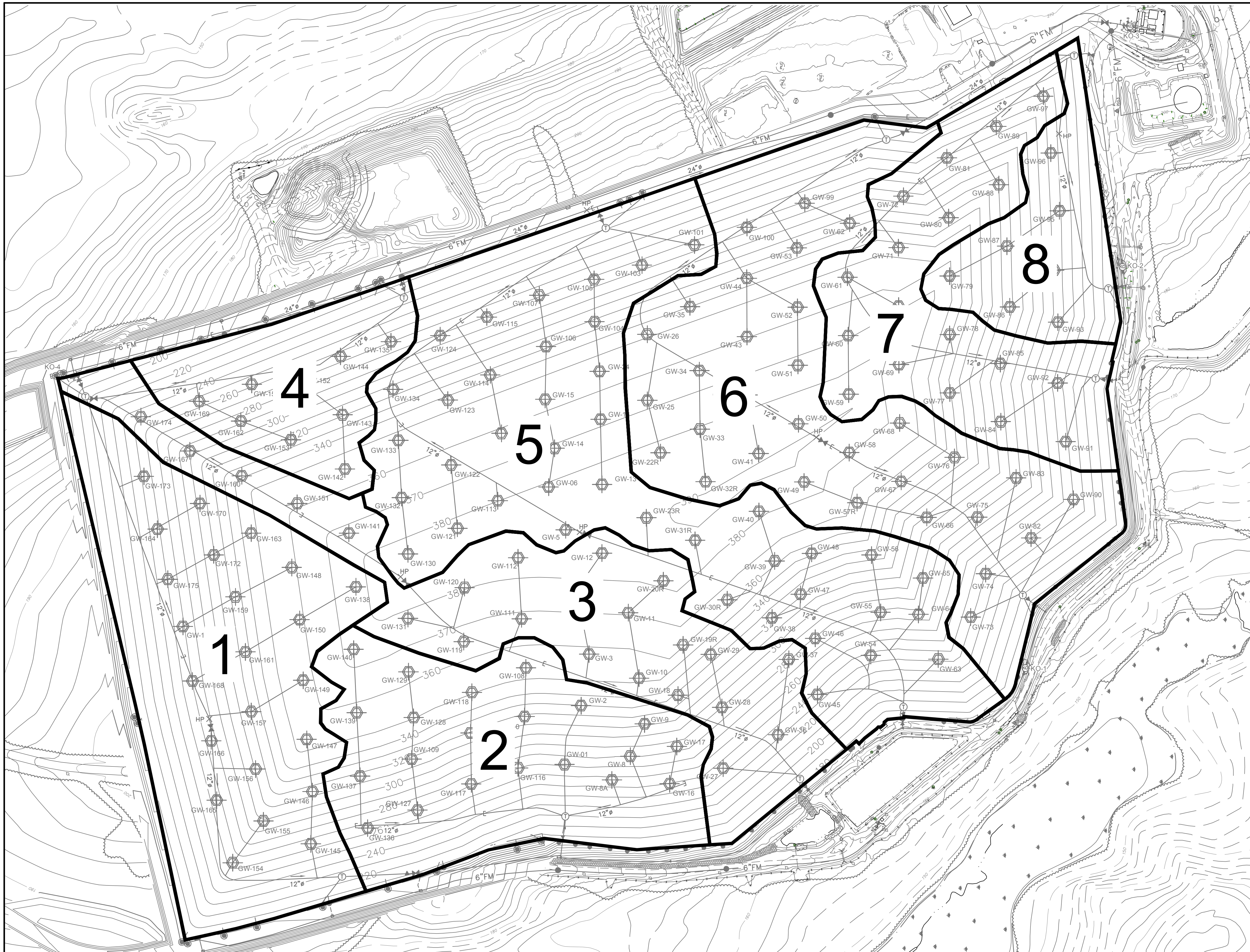
Nominal Diameter	12	in
SDR-	17	
Condensate Flow	Concurrent	
Approximate Pipe Length	760	ft
Inside Diameter	11.160	in
Flow Area	0.679	ft^2

Flow Parameters

Spec. Gravity of Gas	0.81	0.81 is typical for LFG		
Total Flow	333	cfm		
Velocity	8.2	ft/s	ACCEPTABLE	V = Q/A
Pressure Loss	0.19	in WC	ACCEPTABLE	$\Delta P = L^*[(60^*Q^*G^{0.425})/(2971^*d^{2.725})]^{(1/0.575)}$

Condensate

Temp. Initial	110	deg. F
Temp. Reduced	50	deg. F
Generation Rate	0.13	gal/min.
	183	gal/day
	66,777	gal/year



NOTES:

1. BASE MAP WAS PREPARED BY AERIAL SURVEY & PHOTO INC., OF NORRIDGEWOCK, MAINE. PHOTO DATE JULY 31, 2014. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE.
2. PROPOSED EXPANSION GRADES WERE PROVIDED TO SANBORN HEAD BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE.
3. LOCATIONS OF LANDFILL COMPONENTS SHOWN ON THIS PLAN, SUCH AS LEACHATE CLEANOUTS, ARE BASED ON DESIGN LOCATIONS PROVIDED TO SHA BY SEVEE & MAHER ENGINEERS, INC. OF CUMBERLAND, MAINE.

PIPE SIZING LEGEND:

- LFG HEADER PIPE
- GW-35 VERTICAL LFG WELL
- CONDENSATE TRAP
- CONTROL VALVE
- KO-1 CONDENSATE KNOCKOUT
- CLEANOUT LOCATION
- VERTICAL RISER

Volume Summary

Name	2d Area	Approximate Volume of Waste
Volume 1	797608.91 Sq. Ft.	2351587.97 Cu. Yd.<Fill>
Volume 2	612117.49 Sq. Ft.	1979317.58 Cu. Yd.<Fill>
Volume 3	658782.80 Sq. Ft.	2987541.75 Cu. Yd.<Fill>
Volume 4	286832.42 Sq. Ft.	770057.11 Cu. Yd.<Fill>
Volume 5	1149697.11 Sq. Ft.	4755219.43 Cu. Yd.<Fill>
Volume 6	936511.60 Sq. Ft.	3934417.80 Cu. Yd.<Fill>
Volume 7	512148.42 Sq. Ft.	1952651.12 Cu. Yd.<Fill>
Volume 8	243286.41 Sq. Ft.	603030.01 Cu. Yd.<Fill>

ATTACHMENT B

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 301'

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: FEBRUARY 2015

LFG SYSTEM EXPANSION MASTER PLAN

JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

HEADER SIZING WORKSHEET

PROJECT NUMBER: 2536.27

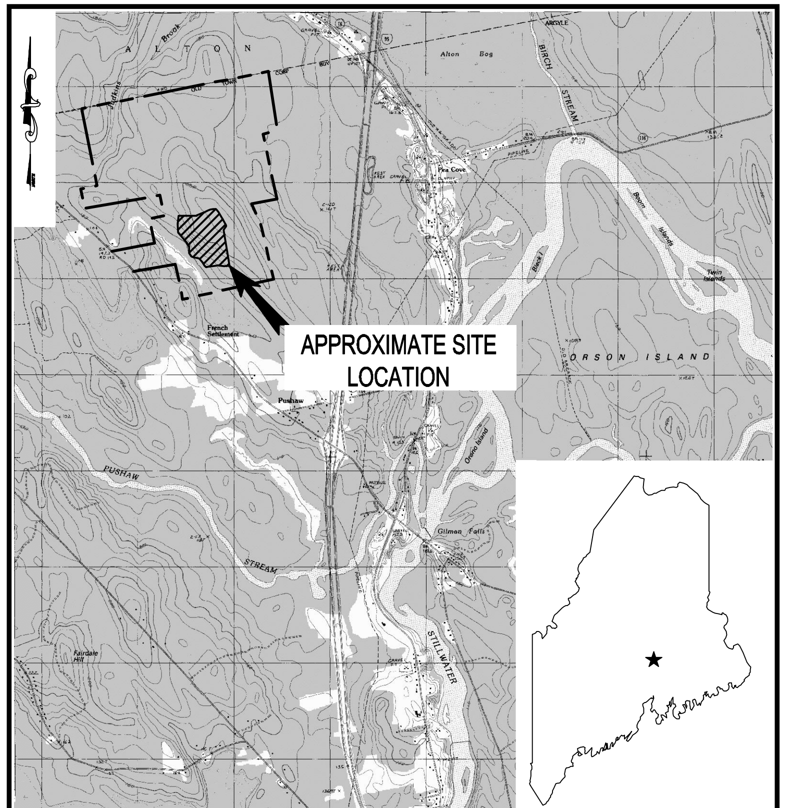
SHEET NUMBER: 1 OF 1

SH-5

VOLUME III - APPENDIX I
CELL 11 LANDFILL GAS SYSTEM EXPANSION DRAWINGS
(REVISED FEBRUARY 2016)

CELL 11 LANDFILL GAS SYSTEM EXPANSION DRAWINGS

JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE
JUNE 2015
(REVISED FEBRUARY 2016)



FROM: USGS TOPOGRAPHIC MAP- OLD TOWN, MAINE (1988)

LOCUS PLAN
SCALE: 1"=1000'

SHEET INDEX

REVISION TABLE

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED BASED ON MEDEP COMMENTS.	RLC

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

⚠ THIS DRAWING SET SHALL BE REVISED PRIOR TO CONSTRUCTION IN ORDER TO SHOW AS-BUILT LFG INFRASTRUCTURE INSTALLED FOR CELLS 9 AND 10, EVALUATE CONNECTIONS TO AS-BUILT LFG INFRASTRUCTURE FOR CELLS 9 AND 10, AND TO UPDATE THE WELL SCHEDULE TO REFLECT AS-BUILT LEACHATE COLLECTION SYSTEM GRADES IN CELL 11.

PREPARED FOR:

new england
NEW ENGLAND WASTE SERVICES
NEWSME LANDFILL OPERATIONS, LLC

JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

PREPARED BY:

SANBORN HEAD

20 FOUNDRY STREET, CONCORD, NEW HAMPSHIRE 03301
(603) 229-1900 FAX (603) 229-1919

PROJ.#: 2536.27 "CELL 11 LANDFILL GAS SYSTEM EXPANSION DRAWINGS, JUNIPER RIDGE LANDFILL, OLD TOWN, MAINE" REVISED FEBRUARY 2016.

NOTES:

1. THE EXISTING LANDFILL GAS EXTRACTION SYSTEM INFRASTRUCTURE FEATURES SHOWN ARE BASED ON A COMBINATION OF DESIGN AND AS-BUILT DOCUMENTATION AVAILABLE TO SANBORN, HEAD & ASSOCIATES, INC. (SANBORN HEAD). ACTUAL LOCATIONS OF INDIVIDUAL FEATURES MAY BE DIFFERENT THAN SHOWN.

2. BASE MAP PREPARED BY AERIAL SURVEY & PHOTO INC., OF NORRIDGEWOCK, MAINE. PHOTO DATE DECEMBER 31, 2014. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE.

LEGEND:

EXISTING

Symbol	Description
190	10-FOOT CONTOUR
	2-FOOT CONTOUR
	LIMIT OF WASTE CONTAINMENT
	CELL LIMIT
	EDGE OF ROAD
	LANDFILL GAS CONVEYANCE PIPE
	LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)
	TREELINE
	FENCE LINE
GW-9	LANDFILL GAS EXTRACTION WELL
GCT-21	COLLECTION TRENCH WELLHEAD
	COLLECTION TRENCH TERMINATION
	PIPE END CAP
	LEACHATE COLLECTION PIPE CLEANOUT
	LEACHATE COLLECTION INLET
	LANDFILL GAS EXTRACTION WELLHEAD
	RIPRAP

SANBORN | HEAD

GRAPHICAL SCALE

100' 50' 0 100' 200'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS

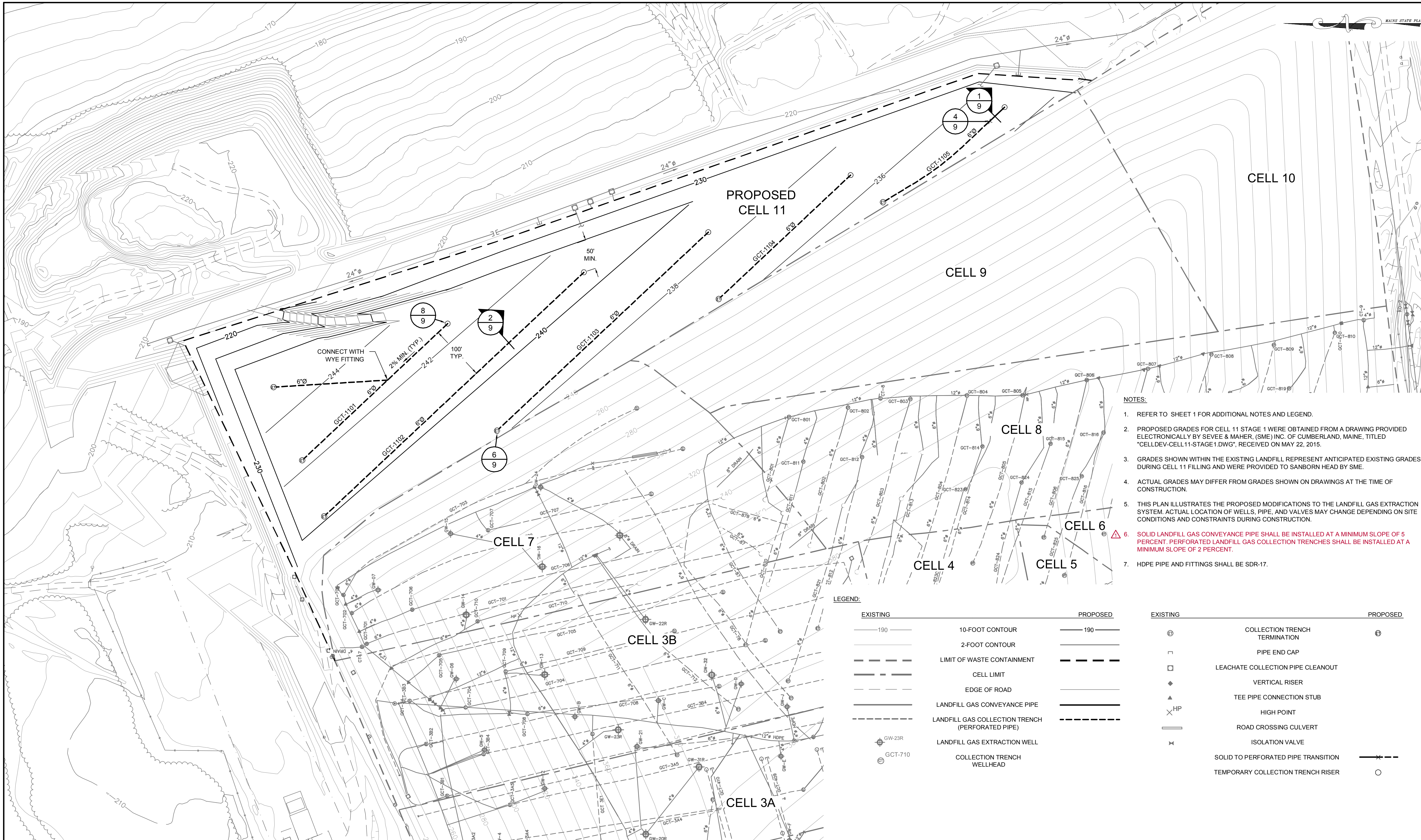
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

EXISTING CONDITIONS PLAN

PROJECT NUMBER: 2536.27

SHEET NUMBER: 1 OF 11



NOTES:

1. REFER TO SHEET 1 FOR ADDITIONAL NOTES AND LEGEND.
2. PROPOSED GRADES FOR CELL 11 STAGE 1 WERE OBTAINED FROM A DRAWING PROVIDED ELECTRONICALLY BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE, TITLED "CELLDEV-CELL11-STAGE1.DWG", RECEIVED ON MAY 22, 2015.
3. GRADES SHOWN WITHIN THE EXISTING LANDFILL REPRESENT ANTICIPATED EXISTING GRADES DURING CELL 11 FILLING AND WERE PROVIDED TO SANBORN HEAD BY SME.
4. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
5. THIS PLAN ILLUSTRATES THE PROPOSED MODIFICATIONS TO THE LANDFILL GAS EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
6. SOLID LANDFILL GAS CONVEYANCE PIPE SHALL BE INSTALLED AT A MINIMUM SLOPE OF 5 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
7. HDPE PIPE AND FITTINGS SHALL BE SDR-17.

LEGEND:

EXISTING		PROPOSED
190	10-FOOT CONTOUR	190
	2-FOOT CONTOUR	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
	LANDFILL GAS CONVEYANCE PIPE	
	LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)	
GW-23R	LANDFILL GAS EXTRACTION WELL	
GCT-710	COLLECTION TRENCH WELLHEAD	

EXISTING		PROPOSED
	COLLECTION TRENCH TERMINATION	
	PIPE END CAP	
	LEACHATE COLLECTION PIPE CLEANOUT	
	VERTICAL RISER	
	TEE PIPE CONNECTION STUB	
HP	HIGH POINT	
	ROAD CROSSING CULVERT	
	ISOLATION VALVE	
	SOLID TO PERFORATED PIPE TRANSITION	
	TEMPORARY COLLECTION TRENCH RISER	

SANBORN | HEAD

GRAPHICAL SCALE

80' 40' 0' 80' 160'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED NOTES.	RLC

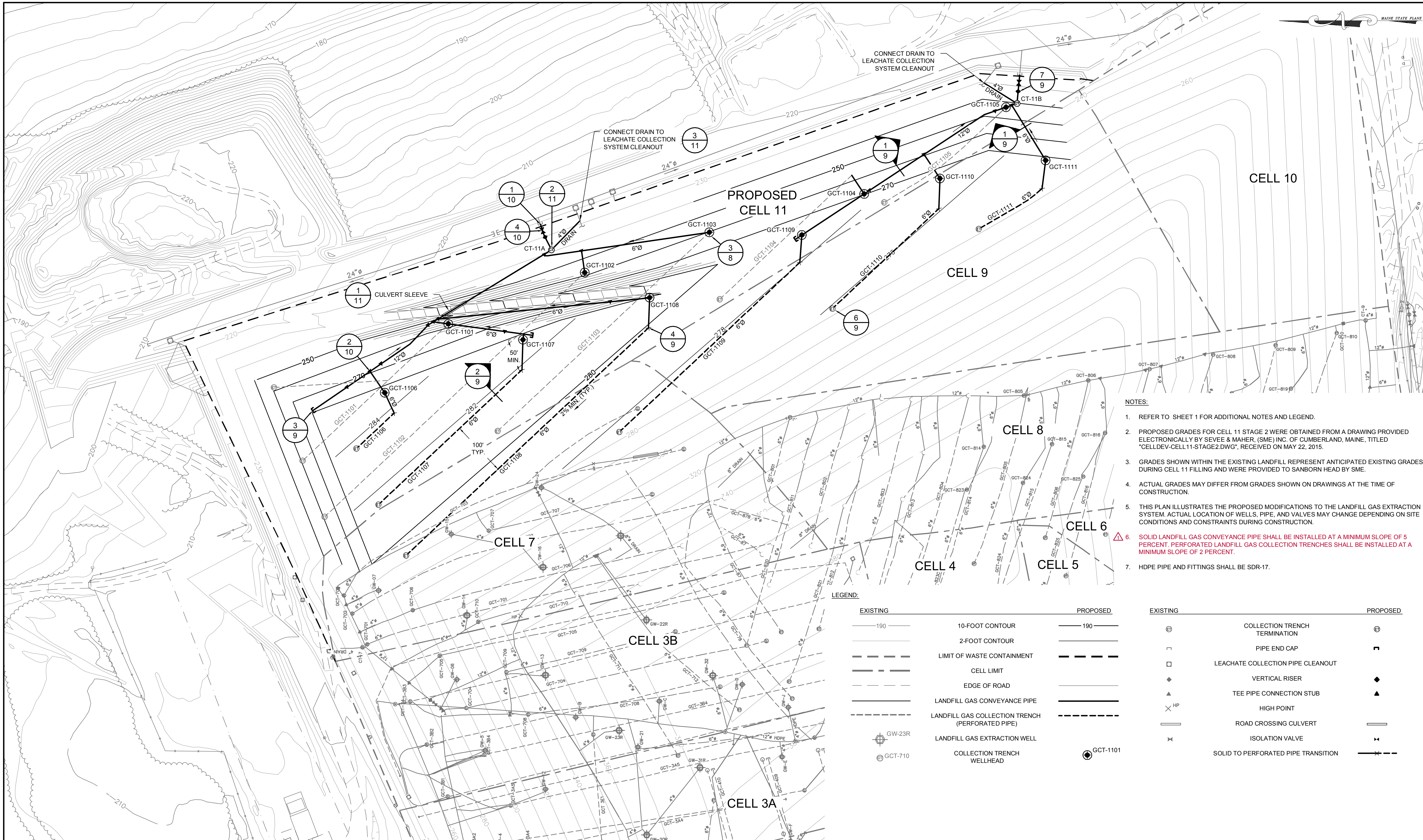
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LFG INFRASTRUCTURE DEVELOPMENT PLAN - STAGE 1

PROJECT NUMBER: 2536.27

SHEET NUMBER: 2 OF 11



NOTES:

1. REFER TO SHEET 1 FOR ADDITIONAL NOTES AND LEGEND.
2. PROPOSED GRADES FOR CELL 11 STAGE 2 WERE OBTAINED FROM A DRAWING PROVIDED ELECTRONICALLY BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE, TITLED "CELLDEV-CELL11-STAGE2.DWG", RECEIVED ON MAY 22, 2015.
3. GRADES SHOWN WITHIN THE EXISTING LANDFILL REPRESENT ANTICIPATED EXISTING GRADES DURING CELL 11 FILLING AND WERE PROVIDED TO SANBORN HEAD BY SME.
4. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
5. THIS PLAN ILLUSTRATES THE PROPOSED MODIFICATIONS TO THE LANDFILL GAS EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
6. SOLID LANDFILL GAS CONVEYANCE PIPE SHALL BE INSTALLED AT A MINIMUM SLOPE OF 5 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
7. HDPE PIPE AND FITTINGS SHALL BE SDR-17.

LEGEND:

EXISTING		PROPOSED
190	10-FOOT CONTOUR	190
	2-FOOT CONTOUR	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
	LANDFILL GAS CONVEYANCE PIPE	
	LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)	
GW-23R	LANDFILL GAS EXTRACTION WELL	
GCT-710	COLLECTION TRENCH WELLHEAD	GCT-1101

EXISTING		PROPOSED
	COLLECTION TRENCH TERMINATION	
	PIPE END CAP	
	LEACHATE COLLECTION PIPE CLEANOUT	
	VERTICAL RISER	
	TEE PIPE CONNECTION STUB	
HP	HIGH POINT	
	ROAD CROSSING CULVERT	
	ISOLATION VALVE	
	SOLID TO PERFORATED PIPE TRANSITION	

SANBORN ||| HEAD

GRAPHICAL SCALE
80' 40' 0' 80' 160'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED NOTES.	RLC

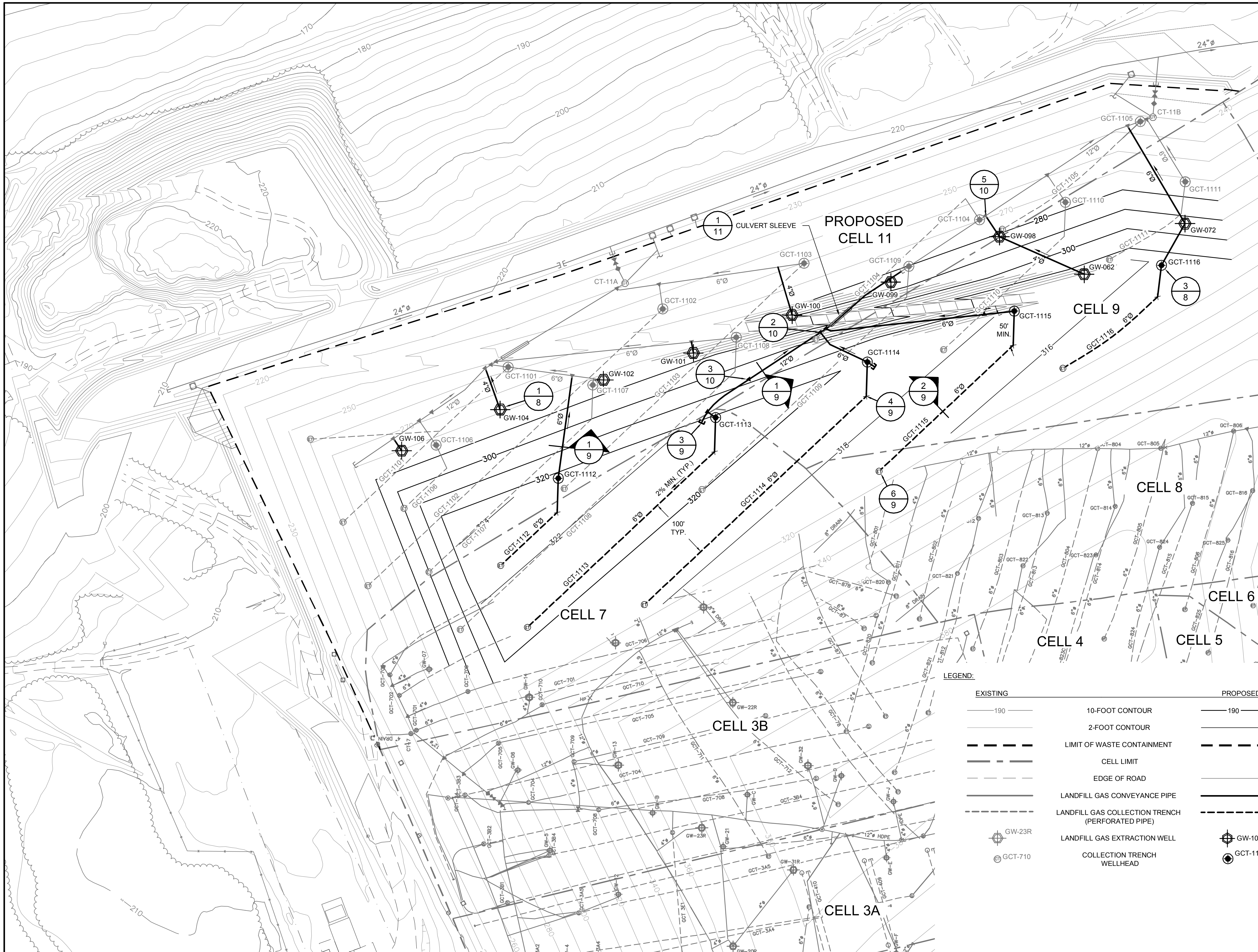
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LFG INFRASTRUCTURE DEVELOPMENT PLAN - STAGE 2

PROJECT NUMBER: 2536.27

SHEET NUMBER: 3 OF 11



NOTES:

1. REFER TO SHEET 1 FOR ADDITIONAL NOTES AND LEGEND.
2. PROPOSED GRADES FOR CELL 11 STAGE 3 WERE OBTAINED FROM A DRAWING PROVIDED ELECTRONICALLY BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE, TITLED "CELLDEV-CELL11-STAGE3.DWG", RECEIVED ON MAY 22, 2015.
3. GRADES SHOWN WITHIN THE EXISTING LANDFILL REPRESENT ANTICIPATED EXISTING GRADES DURING CELL 11 FILLING AND WERE PROVIDED TO SANBORN HEAD BY SME.
4. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
5. THIS PLAN ILLUSTRATES THE PROPOSED MODIFICATIONS TO THE LANDFILL GAS EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
6. SOLID LANDFILL GAS CONVEYANCE PIPE SHALL BE INSTALLED AT A MINIMUM SLOPE OF 5 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
7. HDPE PIPE AND FITTINGS SHALL BE SDR-17.

LEGEND:

EXISTING		PROPOSED
190	10-FOOT CONTOUR	190
	2-FOOT CONTOUR	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
	LANDFILL GAS CONVEYANCE PIPE	
	LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)	
GW-23R	LANDFILL GAS EXTRACTION WELL	GW-104
GCT-710	COLLECTION TRENCH WELLHEAD	GCT-1115

EXISTING		PROPOSED
	COLLECTION TRENCH TERMINATION	
	PIPE END CAP	
	LEACHATE COLLECTION PIPE CLEANOUT	
	VERTICAL RISER	
	TEE PIPE CONNECTION STUB	
HP	HIGH POINT	
	ROAD CROSSING CULVERT	
	ISOLATION VALVE	
	SOLID TO PERFORATED PIPE TRANSITION	

SANBORN HEAD

GRAPHICAL SCALE
80' 40' 0' 80' 160'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED NOTES.	RLC

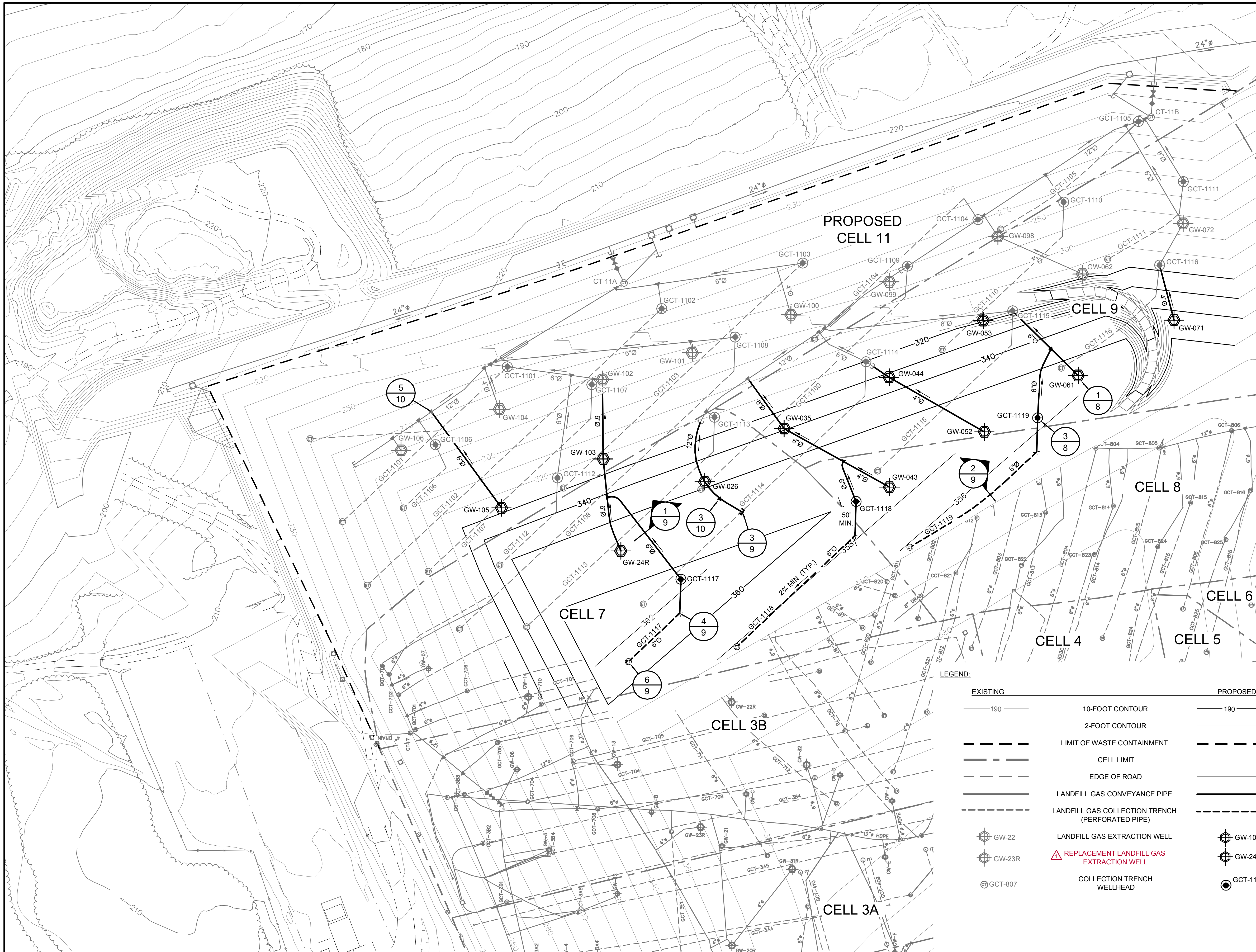
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LFG INFRASTRUCTURE DEVELOPMENT PLAN - STAGE 3

PROJECT NUMBER: 2536.27

SHEET NUMBER: 4 OF 11



NOTES:

1. REFER TO SHEET 1 FOR ADDITIONAL NOTES AND LEGEND.
2. PROPOSED GRADES FOR CELL 11 STAGE 4 WERE OBTAINED FROM A DRAWING PROVIDED ELECTRONICALLY BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE, TITLED "CELLDEV-CELL11-STAGE4.DWG", RECEIVED ON MAY 22, 2015.
3. GRADES SHOWN WITHIN THE EXISTING LANDFILL REPRESENT ANTICIPATED EXISTING GRADES DURING CELL 11 FILLING AND WERE PROVIDED TO SANBORN HEAD BY SME.
4. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
5. THIS PLAN ILLUSTRATES THE PROPOSED MODIFICATIONS TO THE LANDFILL GAS EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
6. SOLID LANDFILL GAS CONVEYANCE PIPE SHALL BE INSTALLED AT A MINIMUM SLOPE OF 5 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
7. HDPE PIPE AND FITTINGS SHALL BE SDR-17.

LEGEND:

EXISTING		PROPOSED
190	10-FOOT CONTOUR	190
	2-FOOT CONTOUR	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
	LANDFILL GAS CONVEYANCE PIPE	
	LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)	
GW-22	LANDFILL GAS EXTRACTION WELL	GW-105
GW-23R	REPLACEMENT LANDFILL GAS EXTRACTION WELL	GW-24R
GCT-807	COLLECTION TRENCH WELLHEAD	GCT-11xx

EXISTING		PROPOSED
	COLLECTION TRENCH TERMINATION	
	PIPE END CAP	
	LEACHATE COLLECTION PIPE CLEANOUT	
	VERTICAL RISER	
	TEE PIPE CONNECTION STUB	
HP	HIGH POINT	
	ROAD CROSSING CULVERT	
	ISOLATION VALVE	

SANBORN HEAD

GRAPHICAL SCALE

80' 40' 0' 80' 160'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED NOTES AND LEGEND.	RLC

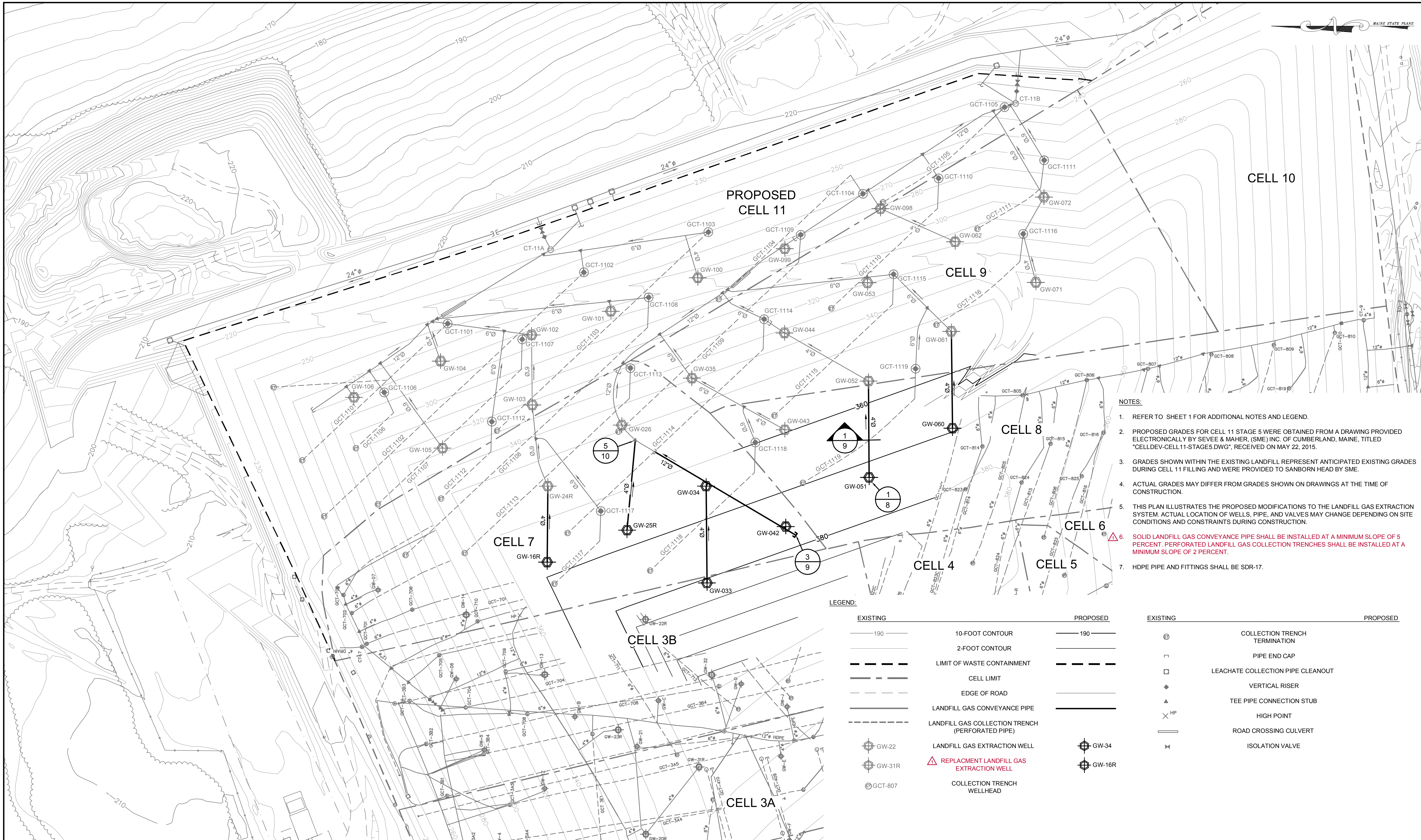
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

LFG INFRASTRUCTURE DEVELOPMENT PLAN - STAGE 4

PROJECT NUMBER: 2536.27

SHEET NUMBER: 5 OF 11



NOTES:

1. REFER TO SHEET 1 FOR ADDITIONAL NOTES AND LEGEND.
2. PROPOSED GRADES FOR CELL 11 STAGE 5 WERE OBTAINED FROM A DRAWING PROVIDED ELECTRONICALLY BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE, TITLED "CELLDEV-CELL11-STAGE5.DWG", RECEIVED ON MAY 22, 2015.
3. GRADES SHOWN WITHIN THE EXISTING LANDFILL REPRESENT ANTICIPATED EXISTING GRADES DURING CELL 11 FILLING AND WERE PROVIDED TO SANBORN HEAD BY SME.
4. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
5. THIS PLAN ILLUSTRATES THE PROPOSED MODIFICATIONS TO THE LANDFILL GAS EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
6. SOLID LANDFILL GAS CONVEYANCE PIPE SHALL BE INSTALLED AT A MINIMUM SLOPE OF 5 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
7. HDPE PIPE AND FITTINGS SHALL BE SDR-17.

LEGEND:

EXISTING		PROPOSED
190	10-FOOT CONTOUR	190
	2-FOOT CONTOUR	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
	LANDFILL GAS CONVEYANCE PIPE	
	LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)	
GW-22	LANDFILL GAS EXTRACTION WELL	GW-34
GW-31R	REPLACMENT LANDFILL GAS EXTRACTION WELL	GW-16R
GCT-807	COLLECTION TRENCH WELLHEAD	

EXISTING		PROPOSED
	COLLECTION TRENCH TERMINATION	
	PIPE END CAP	
	LEACHATE COLLECTION PIPE CLEANOUT	
	VERTICAL RISER	
	TEE PIPE CONNECTION STUB	
HP	HIGH POINT	
	ROAD CROSSING CULVERT	
	ISOLATION VALVE	

SANBORN HEAD

GRAPHICAL SCALE

80' 40' 0' 80' 160'

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED NOTES AND LEGEND.	RLC

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS

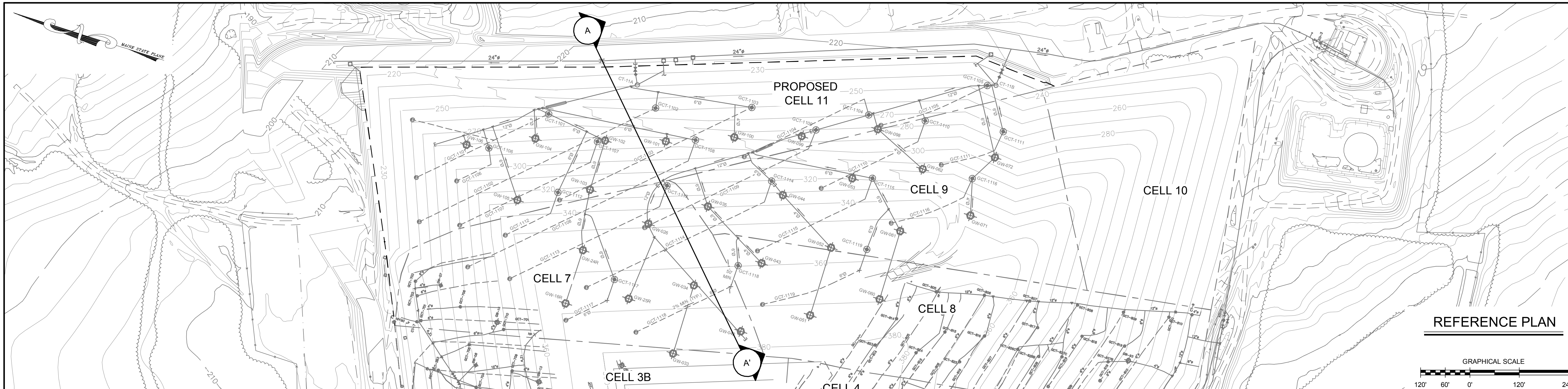
JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

LFG INFRASTRUCTURE DEVELOPMENT PLAN - STAGE 5

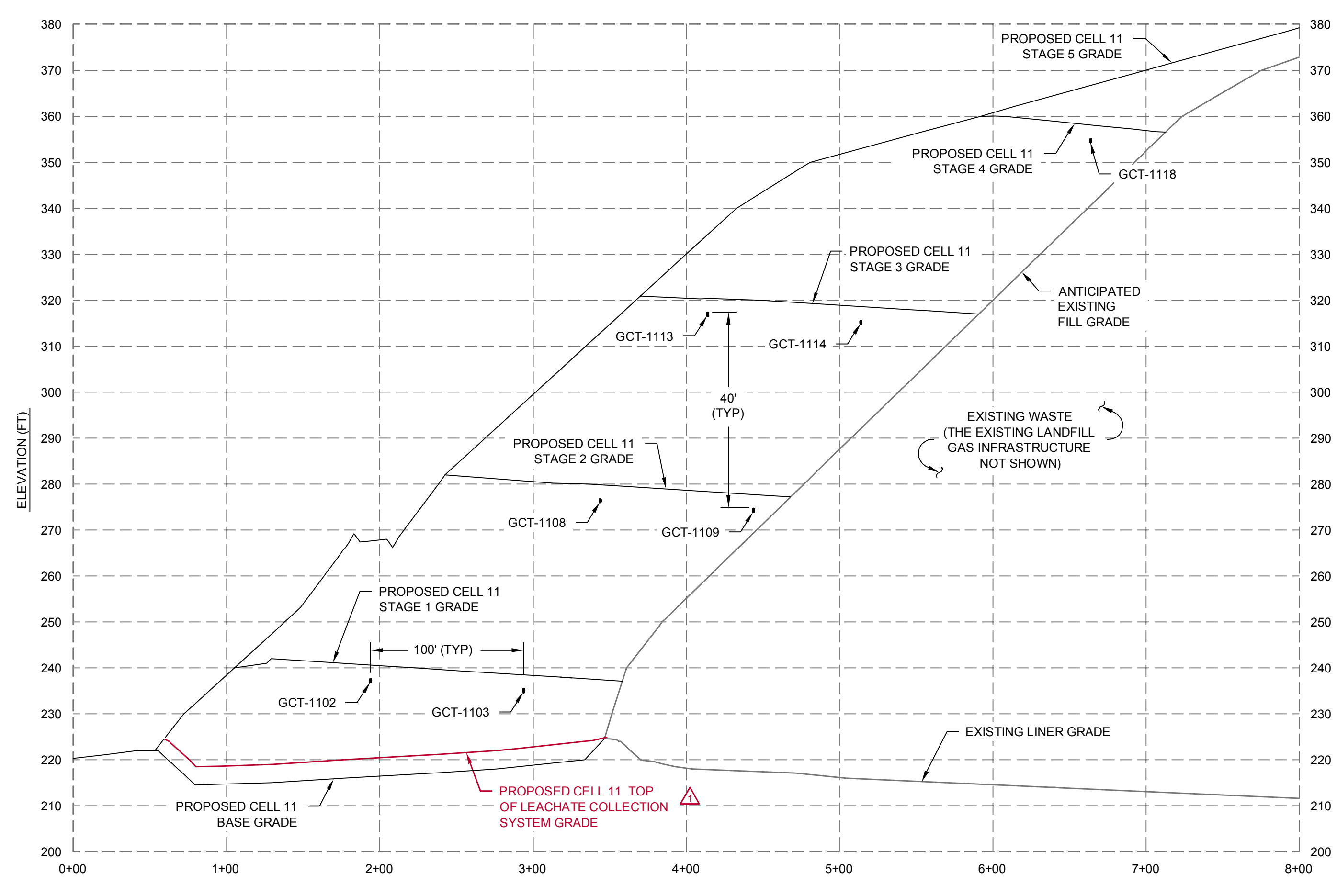
PROJECT NUMBER: 2536.27

SHEET NUMBER: 6 OF 11



REFERENCE PLAN

GRAPHICAL SCALE
120' 60' 0' 120' 240'



SECTION A-A'

NOTES:

1. THE CROSS SECTION SHOWN ON THIS SHEET ILLUSTRATES THE PROPOSED LAYOUT AND SPACING OF GAS COLLECTION TRENCHES IN CELL 11. ACTUAL GRADES MAY DIFFER FROM THE GRADES SHOWN AT THE TIME OF CONSTRUCTION. THE LOCATIONS OF COLLECTION TRENCHES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION. VERTICAL WELLS ARE NOT SHOWN ON THE CROSS SECTION FOR CLARITY.
2. REFER TO SHEET 1 FOR ADDITIONAL NOTES AND LEGEND.

VERTICAL SCALE
20' 10' 0 20' 40'
60' 30' 0 60' 120'
HORIZONTAL SCALE

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED SECTION A-A'	RLC

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

CROSS SECTION

PROJECT NUMBER: 2536.27

SHEET NUMBER: 7 OF 11

NOTES:

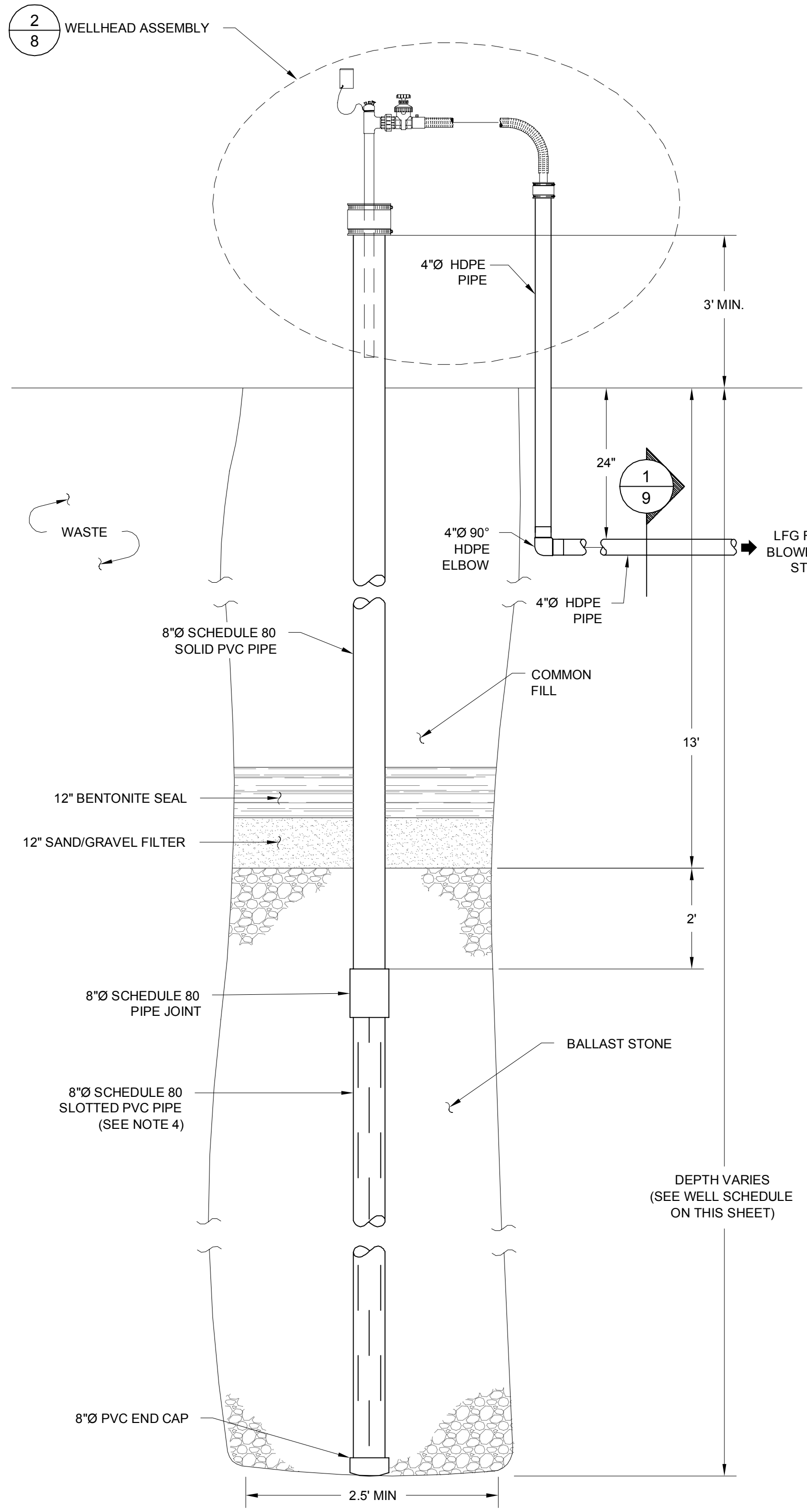
1. ALL HDPE PIPE SHALL BE SDR-17, UNLESS OTHERWISE NOTED.
2. ALL SOLID HDPE PIPE SHALL BE BUTT-FUSION WELDED UNLESS OTHERWISE INDICATED OR AN ALTERNATIVE IS APPROVED BY THE ENGINEER.
3. COVER SOLID HDPE PIPE ON LANDFILL SLOPES WITH MINIMUM 2 FEET OF SOIL AND STABILIZE AGAINST EROSION.
4. PIPE PERFORATED WITH SLOTS ⅛" TO ¼" WIDE BY 8" LONG. FOUR SLOTS PER ROW SPACED 90° APART, WITH ADJACENT ROWS OFFSET BY 45°.

WELL SCHEDULE NOTES:

1. LFG EXTRACTION WELLS SHALL BE INSTALLED WITHIN ONE FOOT OF LISTED LOCATIONS.
2. A TEMPORARY BENCHMARK WITH ELEVATION SHALL BE ESTABLISHED AT EACH WELL PRIOR TO DRILLING.
3. 15 FEET OF SOLID RISER IS TO BE PROVIDED BELOW INTERMEDIATE COVER GRADES. THE INTENT IS TO PROVIDE 3 FEET OF STICK UP ABOVE FILL GRADES.
4. ELEVATIONS SHALL BE CONFIRMED AGAINST AS-BUILT TOP OF PRIMARY SAND GRADES AND FILL GRADES PRIOR TO CONSTRUCTION.

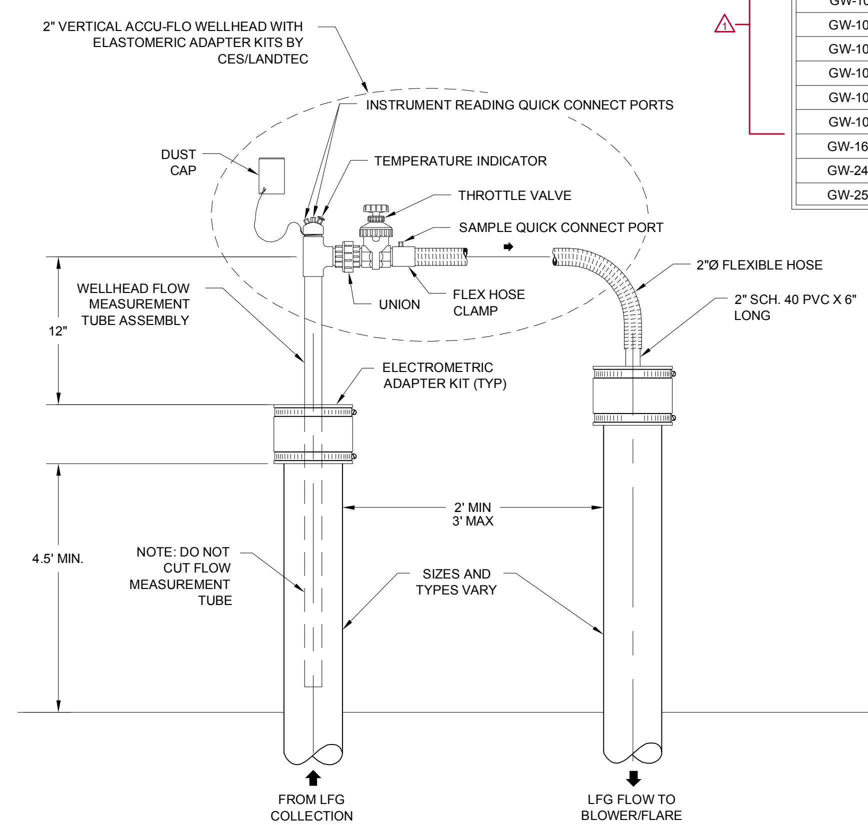
WELL SCHEDULE

WELL DESIGNATION	NORTHING	EASTING	BOTTOM OF WASTE (FT)	TOP OF EXISTING WASTE (FT)	TOTAL WELL DEPTH (FT)	BOTTOM OF WELL SCREEN (FT)	TOP OF WELL SCREEN (FT)	SCREEN LENGTH (FT)	TOP OF CASING ELEV. (FT)
GW-026	478922.2	926825.3	217.1	349.1	116.9	232.1	334.1	101.9	352.1
GW-033	478769.9	926542.9	214.8	380.4	150.6	229.8	365.4	135.6	383.4
GW-034	478771.5	926716.1	214.3	364.4	135.1	229.3	349.4	120.1	367.4
GW-035	478796.8	926909.2	217.6	339.1	106.6	232.6	324.1	91.6	342.1
GW-042	478629.0	926643.7	212.0	376.0	149.0	227.0	361.0	134.0	379.0
GW-043	478630.6	926816.9	213.5	359.6	131.1	228.5	344.6	116.1	362.6
GW-044	478631.1	926990.1	215.3	332.2	101.8	230.3	317.2	86.8	335.2
GW-051	478479.8	926731.7	210.8	372.7	146.8	225.8	357.7	131.8	375.7
GW-052	478480.9	926903.9	214.7	356.4	126.7	229.7	341.4	111.7	359.4
GW-053	478482.6	927079.8	214.5	320.6	91.1	229.5	305.6	76.1	323.6
GW-060	478330.7	926819.7	209.7	369.4	144.7	224.7	354.4	129.7	372.4
GW-061	478332.7	926992.9	211.8	353.0	126.1	226.8	338.0	111.1	356.0
GW-062	478326.0	927153.1	213.2	315.0	86.8	228.2	300.0	71.8	318.0
GW-071	478181.5	927080.5	210.5	345.0	119.4	225.5	330.0	104.4	348.0
GW-072	478167.1	927233.0	212.3	293.9	66.6	227.3	278.9	51.6	296.9
GW-098	478459.4	927212.4	220.9	281.5	45.6	235.9	266.5	30.6	284.5
GW-099	478630.8	927140.8	223.9	284.9	46.0	238.9	269.9	31.0	287.9
GW-100	478786.2	927088.7	222.6	288.8	51.1	237.6	273.8	36.1	291.8
GW-101	478941.9	927028.9	221.8	284.3	47.6	236.8	269.3	32.6	287.3
GW-102	479083.5	926986.5	221.0	282.9	46.8	236.0	267.9	31.8	285.9
GW-103	479082.4	926861.4	225.1	322.1	82.0	240.1	307.1	67.0	325.1
GW-104	479246.4	926939.8	217.0	278.4	46.4	232.0	263.4	31.4	281.4
GW-105	479243.1	926784.1	222.7	327.4	89.7	237.7	312.4	74.7	330.4
GW-106	479401.5	926874.9	213.8	280.5	51.7	228.8	265.5	36.7	283.5
GW-16R	479055.8	926580.2	216.3	367.5	136.2	231.3	352.5	121.2	370.5
GW-24R	479054.6	926716.2	217.6	354.8	122.2	232.6	339.8	107.2	357.8
GW-25R	478912.8	926637.2	215.2	367.1	136.9	230.2	352.1	121.9	370.1



1 LFG EXTRACTION WELL DETAIL

NOT TO SCALE



WELLHEAD NOTES:

A. CES-LANDTEC ACCU-FLOW WELLHEAD SHALL BE INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S INSTALLATION INSTRUCTION MANUAL. INSTRUCTIONS SHALL BE THOROUGHLY READ BEFORE ATTEMPTING ASSEMBLY AND INSTALLATION OF WELLHEAD.

B. WELLHEAD AND FLOW MEASUREMENT TUBE ASSEMBLY SHALL BE COMPATIBLE WITH CES-LANDTEC GEM-2000 (LANDFILL GAS INSTRUMENT).

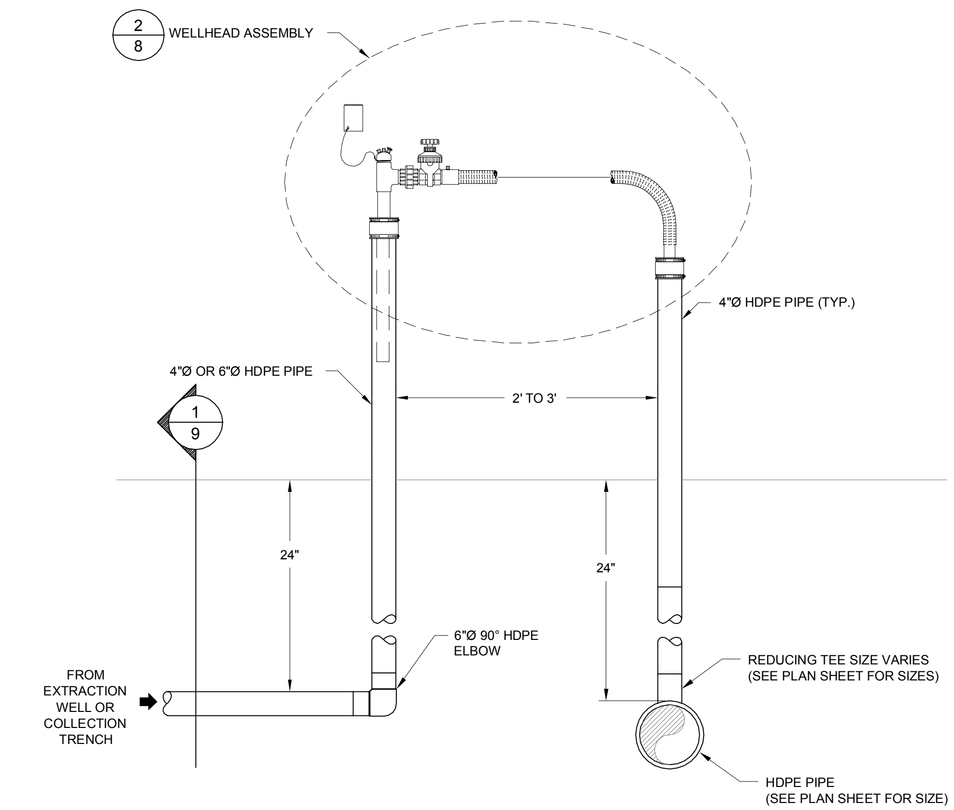
C. FOR FLEXIBLE CONNECTIONS TO PIPE, USE ONLY "IPS WELD-ON 795" PLASTIC PIPE CEMENT OR EQUAL APPROVED BY THE ENGINEER.

D. WARNING: DO NOT CUT THE FLOW MEASUREMENT TUBE ASSEMBLY. FAILURE TO HEED THIS WARNING WILL RESULT IN A DAMAGED OR INOPERATIVE WELLHEAD AND VOID THE WARRANTY. SUCH DAMAGE WOULD REQUIRE REPLACEMENT OF THE WELLHEAD AT THE CONTRACTOR'S EXPENSE.

E. ALLOW SUFFICIENT SLACK IN FLEX HOSE FOR PIPE EXPANSION AND CONTRACTION; AN EXTRA 8 TO 12 INCHES IS RECOMMENDED.

2 WELLHEAD ASSEMBLY DETAIL

NOT TO SCALE



3 COLLECTION TRENCH WELLHEAD

NOT TO SCALE

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY
1	FEB. 2016	REVISED WELL SCHEDULE.	RLC

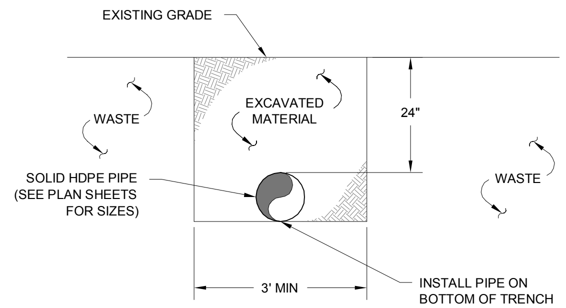
DRAWN BY: R.CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

DETAILS

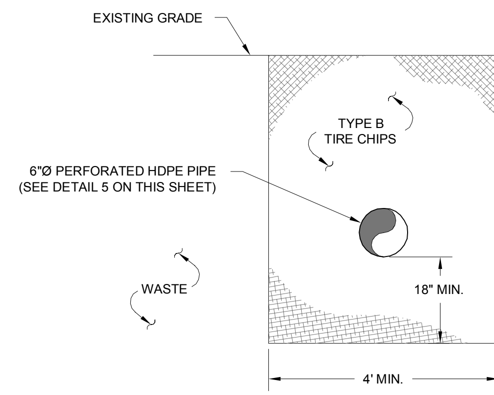
PROJECT NUMBER: 2536.27

SHEET NUMBER: 8 OF 11



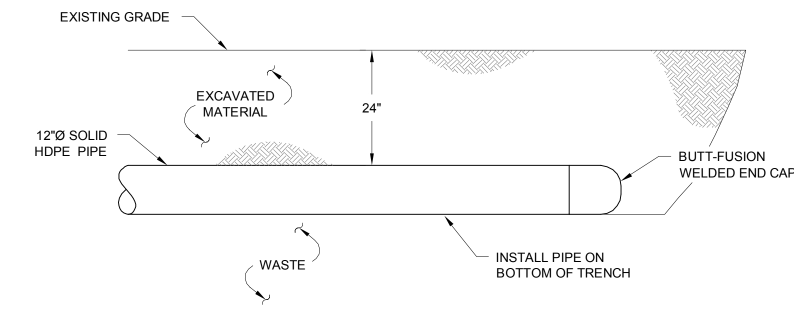
1 SOLID PIPE TRENCH

NOT TO SCALE



2 PERFORATED PIPE TRENCH

NOT TO SCALE

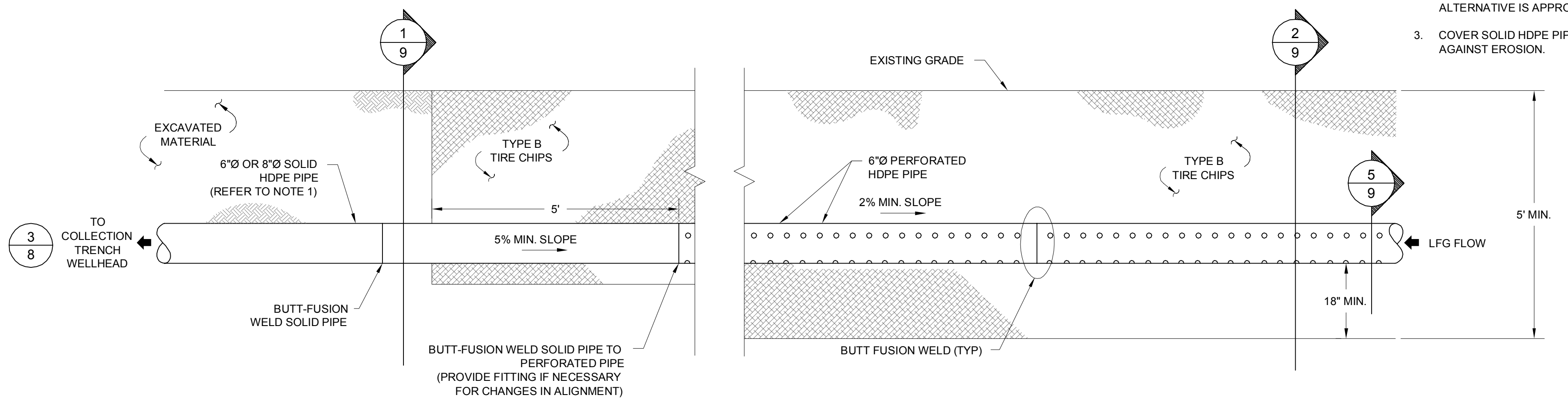


3 TEMPORARY PIPE TERMINATION

NOT TO SCALE

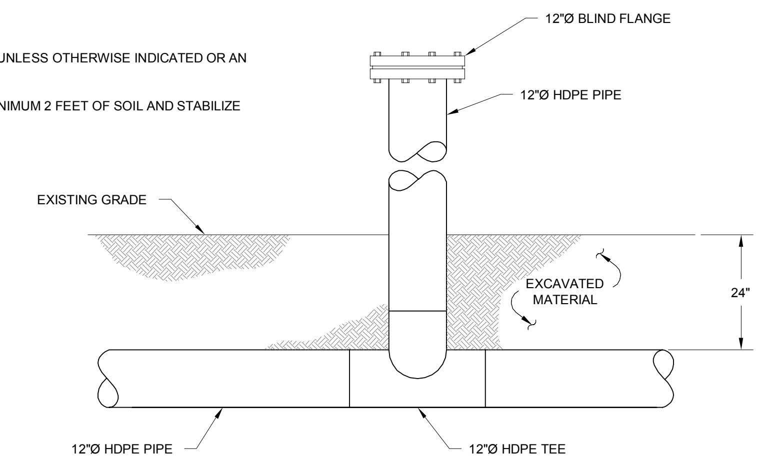
NOTES:

1. ALL HDPE PIPE SHALL BE SDR-17.
2. ALL SOLID HDPE PIPE SHALL BE BUTT-FUSION WELDED UNLESS OTHERWISE INDICATED OR AN ALTERNATIVE IS APPROVED BY THE ENGINEER.
3. COVER SOLID HDPE PIPE ON LANDFILL SLOPES WITH MINIMUM 2 FEET OF SOIL AND STABILIZE AGAINST EROSION.



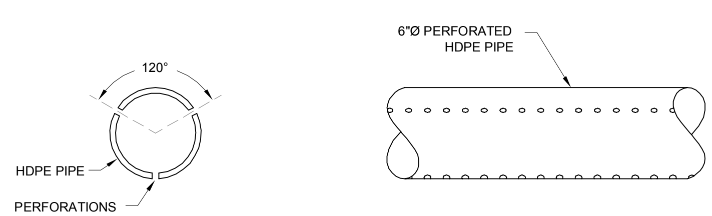
4 LFG COLLECTION TRENCH TRANSITION

NOT TO SCALE



7 VERTICAL RISER

NOT TO SCALE

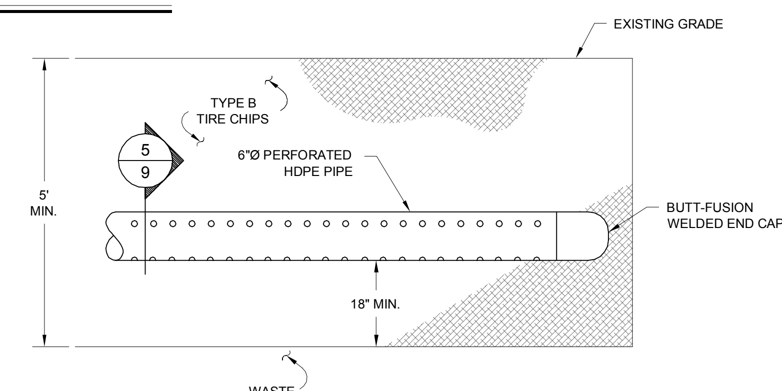


NOTE:

1. HOLES SHALL BE 1/2"Ø DRILLED HOLES SPACED 12" APART ALONG THE LENGTH OF THE PIPE OR APPROVED EQUIVALENT BY OWNER.

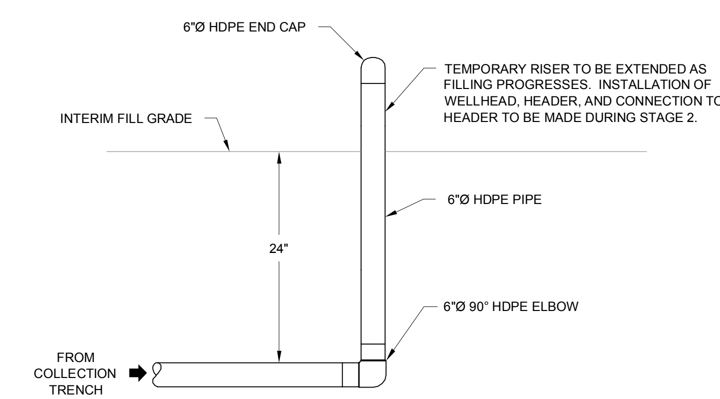
5 TYPICAL PERFORATED PIPE

NOT TO SCALE



6 COLLECTION TRENCH TERMINATION

NOT TO SCALE



8 TEMPORARY COLLECTION TRENCH RISER

NOT TO SCALE

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

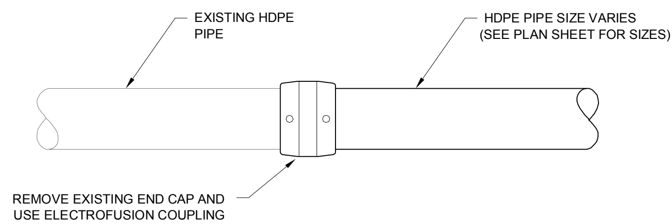
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

DETAILS

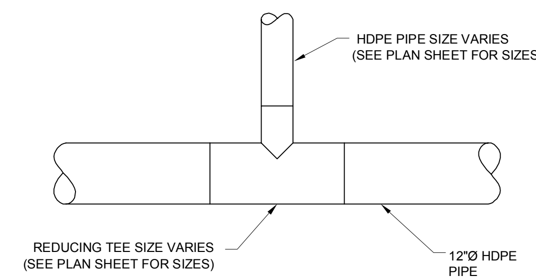
PROJECT NUMBER: 2536.27

SHEET NUMBER: 9 OF 11



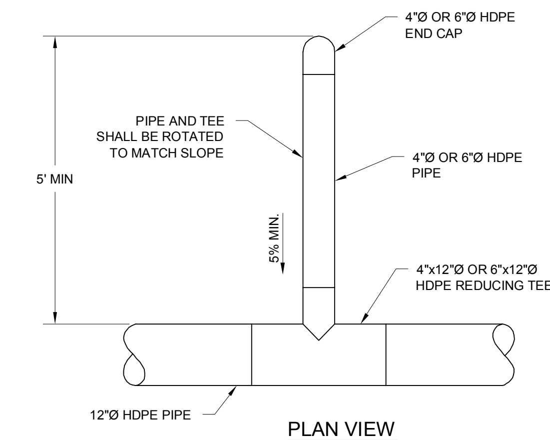
1 EXISTING CONVEYANCE PIPE CONNECTION

NOT TO SCALE



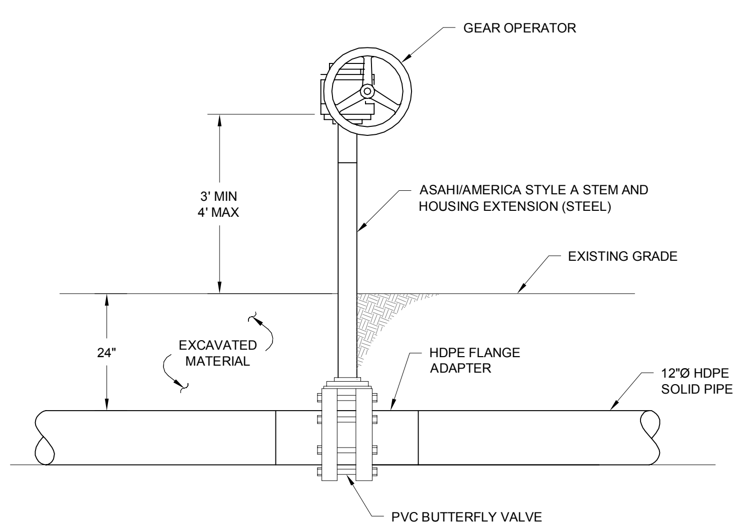
2 "TEE" CONNECTION

NOT TO SCALE



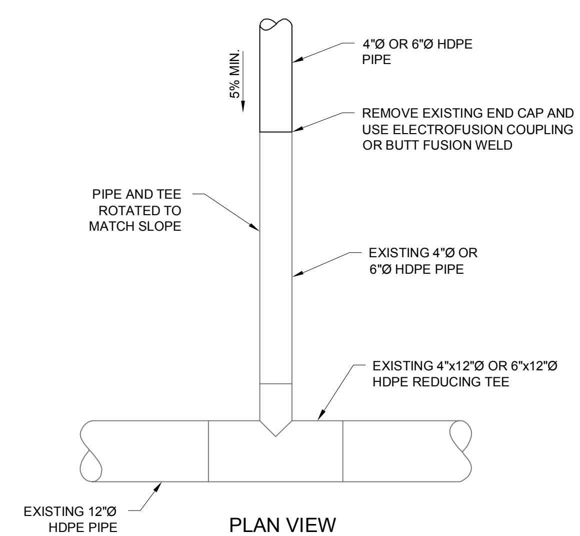
3 TEE PIPE CONNECTION STUB

NOT TO SCALE



4 LFG FLOW CONTROL VALVE

NOT TO SCALE



5 CONNECTION TO TEE PIPE STUB

NOT TO SCALE

NOTES:

1. ALL HDPE PIPE SHALL BE SDR-17.
2. ALL SOLID HDPE PIPE SHALL BE BUTT-FUSION WELDED UNLESS OTHERWISE INDICATED OR AN ALTERNATIVE IS APPROVED BY THE ENGINEER.
3. COVER SOLID HDPE PIPE ON LANDFILL SLOPES WITH MINIMUM 2 FEET OF SOIL AND STABILIZE AGAINST EROSION.

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric S. Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

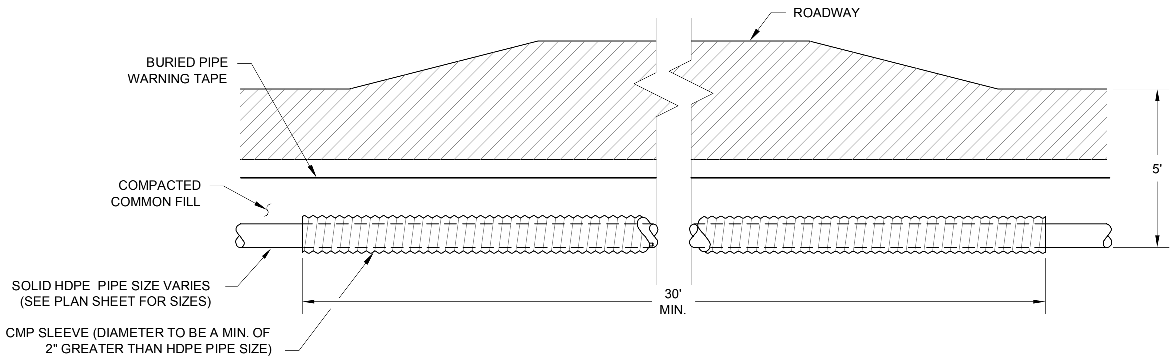
DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

DETAILS

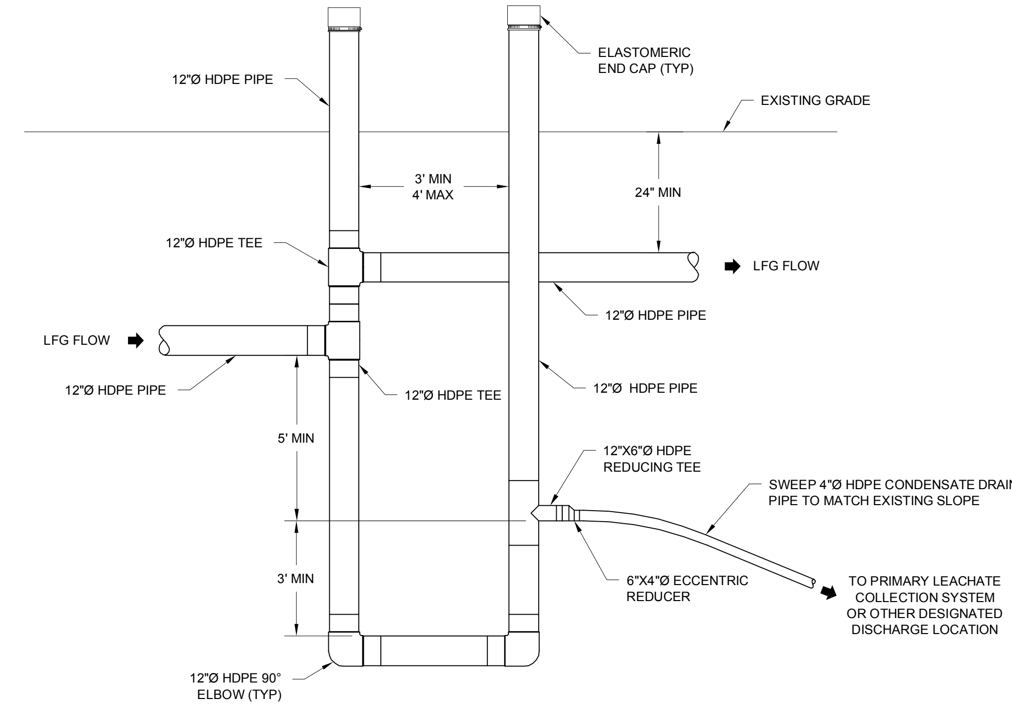
PROJECT NUMBER: 2536.27

SHEET NUMBER: 10 OF 11



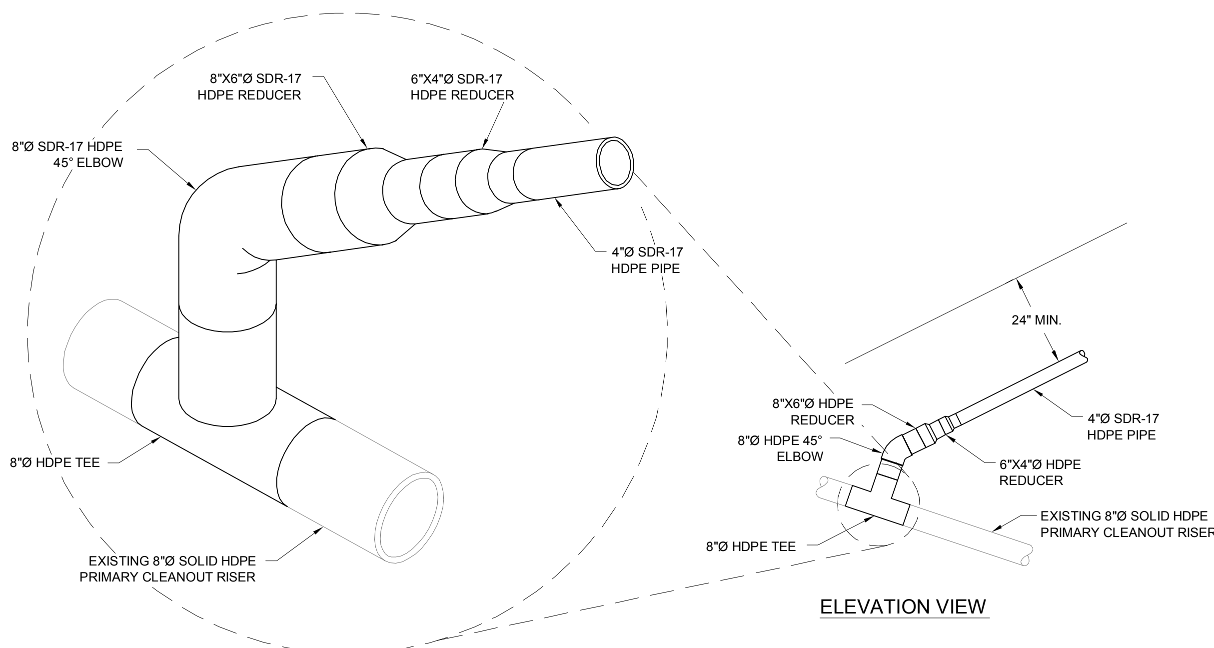
1 ROAD CROSSING PIPE SLEEVE

NOT TO SCALE



2 CONDENSATE TRAP

NOT TO SCALE



3 CONDENSATE DRAIN PIPE CONNECTION TO LEACHATE COLLECTION INLET/CLEANOUT SECTION

NOT TO SCALE

NOTES:

1. ALL HDPE PIPE SHALL BE SDR-17.
2. ALL SOLID HDPE PIPE SHALL BE BUTT-FUSION WELDED UNLESS OTHERWISE INDICATED OR AN ALTERNATIVE IS APPROVED BY THE ENGINEER.
3. COVER SOLID HDPE PIPE ON LANDFILL SLOPES WITH MINIMUM 2 FEET OF SOIL AND STABILIZE AGAINST EROSION.

SANBORN HEAD

SCALE AS NOTED

STATE OF MAINE
Eric
S.
Steinhauser
10809
PROFESSIONAL ENGINEER

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: R. CLAY
PIC: E. STEINHAUSER
DATE: JUNE 2015

CELL 11 LFG SYSTEM EXPANSION DRAWINGS
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

DETAILS

PROJECT NUMBER: 2536.27

SHEET NUMBER: 11 OF 11

SH-6

VOLUME III - APPENDIX I
TECHNICAL SPECIFICATION
02560 - LANDFILL GAS EXTRACTION WELLS
(REVISED FEBRUARY 2016)

SECTION 02560 (REVISED)

LANDFILL GAS EXTRACTION WELLS

PART 1 - GENERAL

1.1 SCOPE OF APPLICATION

A. Supply all equipment, materials, and labor needed to install landfill gas extraction wells as specified herein and as indicated on the Drawings.

1.2 SUBMITTALS

A. Submit to ENGINEER Certificates of Compliance on materials furnished, and manufacturer's brochures containing complete information and instructions pertaining to the storage, handling, installation, and inspection of pipe and appurtenances furnished.

B. Submit to ENGINEER well logs within 7 days of the completion of well installations.

C. The well logs shall depict a construction diagram for each well drilled, including the total depth of the well, the temperature of spoils, depth, thickness, and description of soil or waste strata, and the occurrence of any water bearing zones.

1.3 SITE CONDITIONS

A. Obstructions and saturated conditions such as sludges, and foundry sands are sometimes encountered when drilling in landfills, many of which can be drilled through. CONTRACTOR is expected to make reasonable effort to drill through obstructions and saturated conditions and will be paid for offset redrilling and boring abandonment only with prior written approval from OWNER.

B. Well drilling shall be performed on a level surface. CONTRACTOR shall provide a level surface at each drilling location as required. The size of the level area shall be acceptable to the drilling subcontractor. Any soil placed to level the drilling location shall be removed following well installation. If soil placed for leveling contacts refuse, it shall be disposed of in the active area of the landfill.

PART 2 - PRODUCTS

2.1 BALLAST STONE

A. Ballast Stone shall be hard, durable, and resistant to weathering and to water action, free from overburden, spoil, and organic materials. Ballast Stone shall be washed, and uniformly blended according to the particle size distribution requirements shown below.

Sieve Size	Percent Passing by Weight
2-inch	100
1½-inch	90 - 100
1-inch	30 - 40
¾-inch	10 - 15
½-inch	0 - 5
⅜-inch	0 - 5

2.2 BENTONITE SEAL

A. Bentonite Seal shall be constructed using dry bentonite chips or pellets.

2.3 SAND/GRAVEL FILTER

A. Sand/Gravel Filter should conform to the following particle size distribution.

Sieve Size	Percent Passing by Weight
1½-inch	100
No. 4	70 - 100
No. 40	30 - 80
No. 200	0 - 15

2.4 COMMON FILL

A. Common Fill should be soil containing no stone larger than 4 inches, and shall have a maximum of 75 percent passing the No. 40 sieve and a minimum of 35 percent passing the No. 200 sieve.

2.5 PVC PIPE

A. Refer to Specification Section 15212.

2.6 WELLHEAD

A. Wellheads should be nominal 2-inch size CES/LANDTEC Accu-Flo wellhead, Model 200, with elastomer adapter kits, or equivalent approved by OWNER. The wellhead components are indicated on the Drawings.

B. Wellhead and flow measurement tube shall be compatible with the CES/LANDTEC GEM-2000™ Landfill Gas Monitor.

PART 3 - EXECUTION

3.1 DRILLING

A. ENGINEER shall observe all drilling operations.

B. Wells shall be drilled to the minimum diameter and the specific depths shown on the Drawings. CONTRACTOR shall drill the wells using bucket-type augers and dry drilling equipment; wet rotary drilling equipment may not be used.

C. Well depths shown on the Drawings are estimated based on projected top of waste elevations and may be adjusted in the field by ENGINEER. At no time shall the drilling extend deeper than the bottom of well screen elevation.

D. If water is encountered in a borehole, then CONTRACTOR may be directed to drill beyond the point that it was encountered. If wet conditions remain, then drilling may be terminated and the length of perforated pipe adjusted by ENGINEER, or the well may be relocated. If wet conditions cease (e.g., due to trapped water layer), then drilling will continue to the design depth.

E. As soon as drilling is completed, a safety screen shall be placed over the top of the borehole. This screen shall stay in place until backfilling is within 4 feet of the surface. Safety screen size should be large enough to accommodate all backfill materials and any tools used during backfill yet not large enough for any human to accidentally fall through.

F. Wells shall be drilled straight and plumb and the well pipe shall be installed in the center of the borehole. CONTRACTOR will take all compression off of the pipe by mechanical means and center the pipe in the middle of the borehole before starting to backfill.

G. PVC well pipe shall be solvent cemented and mechanically fastened with stainless steel fasteners.

3.2 BACKFILLING

A. Backfilling the borehole shall commence immediately after drilling is completed and the PVC pipe has been installed. Backfill materials shall be installed as indicated on the Drawings and as approved by ENGINEER.

B. Ballast Stone shall be poured or scooped through the safety screen at a rate that will not endanger the integrity of the well casing and limits the potential for bridging.

C. The Sand/Gravel Filter shall be poured through the safety screen until a layer at least 1-foot thick above the Ballast Stone is formed.

D. The Bentonite Seal will be formed by evenly distributing bentonite around the annulus of the well until a minimum plug thickness of 1 foot has been achieved.

E. Common Fill shall be rodded in the boring to provide even distribution and compaction.

3.3 DISPOSAL

A. Refuse from well drilling operations shall be hauled to the active face of the landfill operation the same day it is excavated.

[END OF SECTION 02560]

P:\2500s\2536.27\Source Files\Response to MEDEP Comments\Attachments\SH-6 - Tech Specs\02560.doc

SH-7

VOLUME III - APPENDIX I
CONSTRUCTION QUALITY ASSURANCE PLAN
(REVISED FEBRUARY 2016)

SANBORN HEAD

Building Trust. Engineering Success.

Construction Quality Assurance Plan

Landfill Gas Extraction System Expansion

Juniper Ridge Landfill

Old Town, Maine

Prepared for

NEWSME Landfill Operations, LLC

Old Town, Maine

Prepared by

Sanborn, Head & Associates, Inc.

20 Foundry Street

Concord, New Hampshire

File 2536.27

Revised February 2016

STATE OF MAINE
Eric
S.
Steinhauser
10809
PROFESSIONAL ENGINEER

TABLE OF CONTENTS

1.0 INTRODUCTION

This Construction Quality Assurance (CQA) Plan addresses the quality assurance activities specific to the installation of landfill gas (LFG) extraction systems components at the Juniper Ridge Landfill. In the context of this Plan, quality assurance refers to means and actions employed to assure that the components of the LFG extraction system are installed in accordance with the drawings and specifications. Quality assurance is provided by a party independent from the Contractor. Quality control refers to those actions taken by the contractor and the manufacturers to ensure that materials and workmanship meet the requirements of the drawings and specifications.

The scope of this CQA Plan applies to manufacturing, shipment, handling, and installation of LFG extraction system components. The CQA Plan does not address design guidelines, installation specifications, or selection of the components. The specifications define the quality of materials and workmanship to be used and employed. The CQA Plan defines the means to assure that the level of material and workmanship used in the construction meets or exceeds the requirements of the drawings and specifications.

2.0 PARTIES

2.1 Project Manager

The Project Manager is an official representative of NEWSME Landfill Operations LLC (NEWSME) and is responsible for the construction project. The Project Manager coordinates the project meetings as defined in Section 3.0, and serves as a liaison between all parties involved in the project. The Project Manager is also responsible for proper resolution of quality assurance issues that arise during construction.

2.2 Design Engineer

The Design Engineer is the individual and/or firms responsible for the preparation of the design, including drawings and specifications. The Engineer is responsible for approving all changes to the drawings and specifications, and for making design clarifications during construction. The Engineer may attend the pre-construction meeting and progress meetings as requested by the Project Manager. At the completion of the construction, the Design Engineer will prepare record drawings based on as-built information provided by the CQA Engineer.

2.3 CQA Engineer

The CQA Engineer is either a qualified representative of NEWSME, or a representative of an engineering firm, independent of NEWSME, that is experienced in observing and documenting construction. The number of CQA Engineer personnel needed on site at a given time will be decided by the Contractor's schedule.

The CQA Engineer is responsible for observing and documenting the construction activities as defined in this CQA Plan. Specific duties of the CQA Engineer personnel include:

- Reviewing the drawings and specifications, and all modifications thereto;
- Reviewing other project-specific documentation, including proposed layouts, and manufacturer and Contractor literature;
- Documenting construction operations using field reports, logs, and/or photographs;
- Attending project meetings;
- Noting on-site activities that could result in damage and/or delays;
- Reporting unapproved construction deviations to the Project Manager;
- Verifying that the contractor is obtaining as-built survey information as required by this plan, the drawings, and specifications; and
- Preparing a construction documentation report.

3.0 COMMUNICATION

3.1 Pre-Construction Meeting

A pre-construction meeting should be held at the site prior to the start of construction. The Maine Department of Environmental Protection should be notified of the meeting time and location prior to the date of the meeting. Typically, the meeting is to be attended by the Project Manager and representatives of the Design Engineer, CQA Engineer, and Contractor. Specific agenda topics for meeting include:

- Review of the project team members, and their roles and responsibilities;
- Review of the site-specific safety and security requirements;
- Review of the project design components and goals; and
- Review of construction schedule.

The meeting shall be documented by the Project Manager or his designee.

3.2 Progress Meetings

Progress meetings should be held with the Project Manager and representatives of the Contractor, CQA Engineer, and other parties invited by the Project Manager. The agenda for the progress meetings should include a discussion of:

- Current progress;
- Planned activities for the next week;
- Issues requiring resolution; and
- New business.

The Project Manager, or his designee, should document the meetings, specifically noting problems and decisions. If any matter remains unresolved at the end of this meeting, then the Project Manager is responsible for assuring that the matter is resolved and the resolution is communicated to the appropriate parties.

4.0 DOCUMENTATION

The CQA Engineer is responsible for providing the Project Manager with documentation that clearly and succinctly describes the construction activities and the locations of the constructed components. A complete file of the construction documentation should be maintained on site. Documentation consists of daily reports, test reports, as-built survey, and the Construction Documentation Report.

4.1 Daily Reports

A report and/or log should be prepared for each day construction is performed. The report and/or log should document the construction and monitoring activities performed that day, identifying problems encountered and remedial action taken. Documentation should include the equipment used, the work force provided including subcontractors. The report and/or log should be completed at the end of the work day, prior to the CQA Engineer leaving the site.

4.2 Testing Reports

On-site pneumatic pressure testing of pipe shall be reported on an appropriate Test Report Log. Test reports shall be submitted along with the daily report.

4.3 As-Built Survey

The CQA Engineer is responsible for verifying that the Contractor as-built survey is correct and accurate. In addition, the CQA Engineer is responsible for documenting changes to the construction details. As-built survey drawings are to include horizontal and vertical locations of trench end points, landfill gas extraction wells, and well heads. The Contractor is responsible for recording changes to pertinent details and supplying this information to the CQA Engineer. The CQA Engineer will forward the as-built survey and changes to the construction details to the Design Engineer, who will prepare the record drawings to be included in the CQA Engineer's Construction Documentation Report.

4.4 Construction Documentation Report

The CQA Engineer is responsible for preparing a report that documents the construction activities and includes the record drawings prepared by the Design Engineer. The report should include the following:

- Parties and personnel involved with the project;
- Seal and signature of a professional engineer licensed in the State of Maine;
- Record drawings, sealed and signed by a professional engineer licensed in the State of Maine;

- Written clarifications and interpretations of the specifications;
- Change Orders to the design;
- Minutes from pertinent meetings;
- Copies of the pertinent CQA records (e.g., contractor submittals; pipe test logs; and daily reports); and
- Photographs.

P:\2500s\2536.27\Source Files\Response to MEDEP Comments\Attachments\SH-7 - CQA Plan\20160203 JRL LFG Master Plan - CQA Plan.docx

SH-8

VOLUME III - APPENDIX I
OPERATIONS AND MAINTENANCE MANUAL
LANDFILL GAS MANAGEMENT SYSTEM
(REVISED FEBRUARY 2016)

Operations and Maintenance Manual

Landfill Gas Management System
Juniper Ridge Landfill
Old Town, Maine

Prepared for
NEWSME Landfill Operations, LLC
Old Town, Maine

Prepared by
Sanborn, Head & Associates, Inc.
20 Foundry Street
Concord, New Hampshire

File 2536.27
Revised February 2016

STATE OF MAINE
Eric
S.
Steinhauser
10809
LICENSED
PROFESSIONAL ENGINEER

TABLE OF CONTENTS

TABLES

FIGURES

1.0 INTRODUCTION

This manual describes operations and maintenance procedures related to the landfill gas (LFG) management system at the Juniper Ridge Landfill (JRL) in Old Town, Maine. The landfill is owned by the State of Maine; however, the facility, and landfill gas, is managed by NEWSME Landfill Operations, LLC (NEWSME). The objectives of the landfill gas management system are to: (i) control odors emanating from the landfill; and (ii) to comply with the federal and state requirements regarding landfill gas emissions.

Currently, JRL has an active landfill gas management system installed in all constructed landfill cells that have reached the necessary grade to initiate installation.

Individuals designated to operate and maintain the landfill gas management system are properly trained with respect to the potential hazards of landfill gas and the proper operating procedures for the site-specific equipment. This manual is a general guide and is not intended to be a substitute for proper hands-on training in operations, regulatory requirements, and site-specific safety activities that may be required by various local, state, and/or federal agencies.

Landfill operations staff should become familiar with the properties of landfill gas and related hazards discussed below, and should receive proper training, which may include lockout/tag out procedures as well as electrical and pneumatic safety procedures.

2.0 CHARACTERISTICS OF LANDFILL GAS AND POTENTIAL HAZARDS

2.1 Landfill Gas Characteristics

Landfill gas is generated when anaerobic bacteria consume organic matter in waste. Landfill gas is composed of methane, carbon dioxide, oxygen, nitrogen, water vapor and trace amounts of hydrogen, ethane, hydrogen sulfide, and volatile organic compounds (VOCs). Oxygen and nitrogen are typically present because of air entrained in the landfill (air is approximately 21 percent oxygen and 79 percent nitrogen). Air entrainment in landfills occurs during placement of waste, from atmospheric weather effects, because of landfill gas management system operations, or from diffusion into the landfill. Typical ranges of constituent concentrations in landfill gas are presented in Table 1.

Important landfill gas characteristics are provided below.

- Landfill gas primarily consists of methane (up to approximately 50 percent) and carbon dioxide (up to approximately 40 percent).

- Landfill gas has relatively high moisture content; cooling generally results in the formation of condensate.

- Landfill gas is flammable and potentially explosive under the right combination of methane and oxygen, plus an ignition source. See below for more information on the flammable concentration range of methane.

- Landfill gas may migrate through surrounding soils, within open conduits, and trench backfill.

- Landfill gas may accumulate in confined spaces.

- Landfill gas has a specific gravity that is usually close to that of air.

- Within the landfill, the typical temperature range for landfill gas is 16 degrees (°) to 66° Celsius (C) (60° to 150° Fahrenheit (F)).

- Component gases (methane, carbon dioxide, water vapor and others) tend to stay together, but may separate through soil and liquid contact.

- Secondary constituents (trace gases) may cause nuisance odors, environmental pollution, and may create a health risk.

The flammable range of methane is approximately 5 to 15 percent (by volume) in air. The lower limit of 5 percent is referred to as the Lower Explosive Limit (LEL); the upper limit of 15 percent is referred to as the Upper Explosive Limit (UEL). The specific gravity of methane and carbon dioxide are 0.55 and 1.52, respectively. However, the specific gravity of landfill gas is close to that of air, and it should not be assumed that methane gas would rise. The landfill gas mixture may be lighter or heavier than air and its behavior will be dictated by its overall composition.

Methane and carbon dioxide are odorless gasses. However, landfill gas has its own characteristic odor due to the presence of trace compounds in the gas. Some of the most significant examples of the classes of odor causing trace constituents include esters, phenols, organic acids, solvents, and sulfur compounds (including mercaptans). However, landfill gas may not always exhibit an identifiable odor because the odor carrying trace components may be stripped off because of movement through cover or adjacent soil.

Landfill gas levels can be monitored using various meters. At a minimum, LEL, percent methane, percent oxygen, and parts per million (ppm) hydrogen sulfide should be measured at any location where there is potential for landfill gas to be present and where personnel could be exposed to landfill gas, including landfill sideriser buildings.

Personnel should take immediate action to evacuate the area, if monitoring results indicate:

- The LEL is 50 percent or higher (2.5 percent methane by volume);
- The concentration of oxygen is lower than 19.5 percent; or
- The concentration of hydrogen sulfide is more than 5 parts per million (ppm).

2.2 Potential Hazards to Personnel

Landfill operations staff should be familiar with the following types of hazards related to the presence of landfill gas and landfill gas condensate, and the appropriate and safe procedures to identify and avoid them.

Methane is a colorless, odorless, flammable, and potentially explosive gas that may be emitted into the atmosphere as landfill gas together with other volatile trace gases. Landfill gas, which may contain other gases, may migrate through soil and bedrock into surrounding areas or contact groundwater where it may adversely affect the environment. Landfill gas may travel long distances underground and accumulate underneath and in structures and confined or enclosed spaces creating a potential explosion hazard. Carbon dioxide, the other major component of landfill gas, is colorless, odorless, and non-combustible.

2.2.1 Respiratory Hazards

Methane and carbon dioxide are asphyxiates. A potential hazard posed by landfill gas is oxygen deficiency, which may cause asphyxiation. As landfill gas builds up it displaces air, hence reducing the amount of oxygen that can be inhaled by a person. An oxygen deficient atmosphere exists when the oxygen comprises 19.5 percent or less of the air. It is imperative that confined space regulations and procedures be followed before personnel enter confined spaces or locations where an oxygen deficient atmosphere could exist. Under certain circumstances, special "permitted entry" requirements apply.

Potentially lethal concentrations of hydrogen sulfide (H_2S) may be present in landfill gas at JRL. Personnel must always be alert for the presence of H_2S. H_2S is a colorless, toxic, flammable gas, which at low concentrations has an offensive odor similar to that of rotten eggs. Sense of smell can be lost within 2 to 15 minutes of exposure to H_2S; therefore sense of smell is NOT adequate for indicating the continuing presence of H_2S or for warning of hazardous concentrations. The effects of inhalation depend on the concentration of H_2S and the duration of exposure. Exposure at high concentrations can quickly lead to death. Atmospheres containing concentrations of H_2S greater than 100 ppm are considered immediately dangerous to life and health (IDLH).[1]

Air monitoring should be performed prior to entering areas expected to contain LFG or hazardous atmospheres. If significant concentrations of LFG constituents are present, the space should be ventilated. If the gas cannot be removed, appropriate respiratory protection, necessary personal protective equipment (PPE), and rescue and communication equipment should be used before entering the area.

Other volatile organic components of landfill gas may also pose respiratory hazards.

2.2.2 Explosive Atmosphere

Landfill gas tends to migrate and may accumulate in confined spaces. The occurrence and accumulation of methane is sometimes transient. The presence of slight amounts of methane, less than the LEL, is an indication that more may accumulate under other conditions if corrective action is not taken. If methane is detected at concentrations greater than 15 percent (100 percent of the UEL) by volume, then there is always the potential for an explosive methane-air "front" that could be formed nearby by dilution.

Personnel should take the following precautions.

[1] https://www.osha.gov/Publications/hydrogen_sulfide.html

- Avoid any possible source of ignition when working on the landfill gas management system. Sources of ignition may include cell phones (on or off), battery-powered watches, flashlights, non-intrinsically safe equipment, etc. **Smoking is prohibited when working on or near the landfill gas system components**.

- Avoid wearing synthetic clothing, such as polyester, as these materials are extremely flammable. **Wearing synthetic clothing can be fatal in a methane gas flash fire.**

2.2.3 Potential for Landfill Fire

If large quantities of air are introduced into the landfill in a localized area, through either natural occurrence or overly aggressive operation of the landfill gas extraction system, then poorly supported combustion of the buried waste may occur and carbon monoxide may be detected. Subsurface fires produce temperatures of several hundred degrees Fahrenheit within the landfill and typically results from short-circuiting air intrusion into:

- The landfill/cover soil interface;
- Cracks, breaks or buried imperfections in the cover/cap;
- Breaks in buried collection piping and extraction wells; or
- Backfill surrounding collection system components (e.g., from the filter or gravel pack of an extraction well or the gravel backfill around a sump).

Preventing the introduction of air into the landfill by proper operation of the landfill gas extraction system and maintenance of the landfill cover is the best course of action.

2.2.4 Landfill Gas Condensate

Landfill gas condensate may contain trace chemicals and may be biologically active. Appropriate protective gloves and splash protection equipment should be used when working with landfill gas condensate. Operating personnel should avoid direct skin contact.

Condensate is odorous, and may release VOCs. Careful control during condensate handling and disposal is necessary to limit release of odors.

The vapors emanating from condensate storage tanks may be flammable.

3.0 SYSTEM COMPONENTS AND MONITORING PROGRAM

3.1 Introduction

The primary objectives of the landfill gas management system are to: (i) control odors emanating from the landfill; and (ii) comply with the federal, state, and local requirements regarding landfill gas emissions. Further, the landfill gas management system should be operated to maintain anaerobic conditions within the landfill, thereby limiting the intrusion of air into the waste. To meet these objectives, the system components should be monitored on a routine basis.

The landfill gas management systems at JRL are installed in phases as each landfill cell is developed and filled. Due to the waste stream mix at JRL (comprising of pulp/paper sludge, wastewater sludge, CDD residuals, FEPR, and other special wastes) Hydrogen Sulfide (H_2S) tends to be generated rather quickly (within three-months of initial waste placement within a cell). Due to the odor issues associated with rapid LFG generation, LFG is managed using horizontal collection trenches constructed in the waste as the cell is developed. Landfill gas extracted from the landfill is conveyed to a sulfur removal system and then to a blower/flare station for landfill gas treatment. As the outer slopes of the cells are filled to final grades, vertical extraction wells are installed in accordance with the final landfill gas plan approved by the Maine Department of Environmental Protection (MEDEP). The active landfill gas extraction system will continue to operate at the landfill, extracting landfill gas from the horizontal trenches and vertical extraction wells installed in areas where final grades are achieved.

Included with this manual are figures that illustrate the landfill gas infrastructure associated with the constructed cells at the facility.

A description of the various components of the landfill gas management system is provided below. Where applicable, the monitoring requirements for the various components are also discussed below and summarized in Table 2.

3.2 Gas Collection Trenches

Gas collection trenches are installed at discrete locations in the waste mass to provide interim landfill gas collection coverage during filling of the landfill, prior to reaching final grade. The first series of trenches in an area of new waste fill will be excavated into the waste mass after approximately 20 feet of waste has been placed in the cell. Additional trenches are constructed at about 40-foot vertical increments thereafter. The trenches are spaced horizontally about 100 feet apart with typically 3-4 trenches per 40-foot lift (dependent upon overall cell width). The constructed trench will consist of a stone or tire chip-filled excavation with a perforated 6-inch diameter SDR-17 HDPE conveyance pipe. The trench will be installed at a minimum 2 percent slope and will drain away from the gas collection wellhead and back into the landfill via the perforations in the pipe. If necessary, collection trench risers will be extended by welding on additional lengths of HDPE pipe as filling progresses. Wellheads will be installed to extract LFG from the collection trenches.

3.3 Condensate Traps and Knockouts

Landfill gas condensate is managed primarily with condensate traps constructed inside the limit of waste containment, and with condensate knockout structures constructed outside the limit of waste containment.

Condensate traps are typically constructed at low points along landfill gas conveyance pipes. The traps are designed to allow condensate to drain into the waste mass or to discharge to the primary leachate collection system. Each trap consists of a U-shaped tube filled with liquid to provide a seal against the vacuum in the system. To maintain a seal, the liquid column in the trap must be at least as high as the maximum vacuum obtainable in that portion of the system.

Condensate knockout structures are constructed to remove condensate from landfill gas conveyance pipe located outside the limit of waste containment. Condensate collected in a knockout structure is pumped to either the nearest primary leachate collection system sideriser cleanout pipe or to a leachate storage tank on site. A condensate knockout structure at JRL is a Class 1 Division 1 Group D classified space, and is considered to be a confined space with entry by permit only.

3.4 Wellhead Assemblies

The wellhead assemblies are generally installed at each horizontal collection trench and on the vertical extraction wells. The wellheads provide a means to control landfill gas flow and a means to collect monitoring data. The wellhead assemblies include a gate valve, flexible hose, fittings, and taps that are designed to allow for:

- Differential settlement between the landfill gas transmission pipe and the wellhead assembly;
- Sampling of the gas in the wellhead;
- Measurement of the gas flow rate;
- Measurement of the gas temperature;
- Control of the gas flow rate; and
- Access to the well from the top for equipment or measurements.

Wellheads are monitored on a monthly basis (as part of the LFG monitoring program) to observe their general condition, with particular attention to the condition of the flexible hose between the wellhead and the transmission pipe. Well bore seals should also be inspected and adjusted, if necessary, to create an adequate seal. Additional data to be gathered includes:

- Valve position (percent open);
- Gas flow rate;
- Static pressure;
- Percent methane;
- Percent carbon dioxide;
- Percent oxygen; and
- Gas temperature.

3.5 Sulfur Removal System

JRL operates a Thiopaq® sulfur removal system as part of the gas control system to remove total reduced sulfur (TRS) compounds, primarily hydrogen sulfide (H_2S), from the LFG prior to combustion. JRL maintains the flexibility to operate other temporary or additional TRS control equipment, such as Sulfatreat, for cases of scrubber downtime or temporary surges in LFG flow or TRS concentration.

The sulfur removal system is monitored to maintain the concentration of H_2S exiting the system to 1,000 ppmv or less. Using colorimetric tubes, the H_2S concentration must be measured at the inlet and outlet of the sulfur removal system on two days every week, with at least three days between sampling. On the two weekly sampling days (e.g., Monday and Friday), JRL must collect two rounds of samples at least four hours apart.

JRL also collects monthly samples from the inlet (three samples) and outlet (three samples) to estimate sulfur dioxide (SO_2) emissions using an approved method such as laboratory analysis. O&M procedures for the Thiopaq® sulfur removal system are detailed in a separate O&M manual from this document.

3.6 Blower/Flare Station

Currently, there is one 3,500 cubic feet per minute (cfm) flare station with dual blowers that extracts landfill gas from the entire landfill.

The blower/flare station is checked at least weekly to evaluate: (i) the condition of pipe and connections; (ii) the consistency of the flare operation; and (iii) the condition of the automatic sparking ignition system.

3.7 Ancillary Observations

In the course of monitoring system components, the condition of the landfill cover systems should also be observed for indications of settlement, tears and rips in the exposed synthetic cover, stressed vegetation, improper drainage, and condition of cover soils. Further, the presence of odors should also be noted.

3.8 Monitoring and Reporting

NEWSME monitors the LFG system components on a monthly basis as required by the New Source Performance Standards (NSPS) for MSW Landfills as described below in Section 4.0. Semiannual reports on the gas collection and control system (GCCS) are submitted to MEDEP and an annual certification of compliance is submitted to the U. S. Environmental Protection Agency (USEPA).

4.0 OPERATION AND ADJUSTMENT

4.1 Well System Tuning

The adjustment or "tuning" of the landfill gas management system involves monitoring various parameters and making adjustments to optimize the extraction of landfill gas from each extraction point in the system. The objective of the tuning is to provide negative pressure at each extraction point without causing intrusion of air into the landfill. Tuning is an iterative process, as adjustments to any portion of the system have the potential to affect the entire system. A discussion of the monitoring and tuning procedures is provided below. A summary of the steps to be taken and data to be gathered is presented in Table 3.

4.2 Equipment

The instrument used to monitor landfill gas flow rates, pressures, and composition at JRL is a GEM-2000 or GEM-5000 Portable Gas Analyzer (meter) manufactured by CES Landtec of Colton, California. Other site approved equivalents may be used. Like most analytical instrumentation, it is important to field calibrate the meter prior to using it to collect landfill gas data. The instrument requires calibration with methane, carbon dioxide, and oxygen span gases. The meter should be field-checked with calibration gas, and if necessary, calibrated in accordance with the manufacturer's recommendations at least once each day that it is used. A log of the calibration should be kept with the instrument. Vibration, shock, and large temperature changes can affect the calibration of the instrument.

CES Landtec certifies that this instrument is intrinsically safe. However, it is generally good practice to avoid operating this instrument in an explosive atmosphere. Factory calibration should be performed in accordance with manufacturer's requirements.

4.3 Indicator Parameters

Typical ranges in concentrations of the various constituents found in landfill gas are presented in Table 1. Normal values for the landfill are established using data collected during operation. System tuning should be performed based on the methane, carbon dioxide, and oxygen concentrations in the landfill gas at individual wellheads. Typically, as the methane and carbon dioxide concentrations decrease, the concentrations of oxygen and nitrogen will increase. Such a trend indicates air intrusion into the system and adjustments should be made to reduce the landfill gas extraction (vacuum) rate to maintain oxygen to five percent or less. Alternatively, the landfill gas extraction rate could be increased if the methane and carbon dioxide concentrations are consistently in the middle to upper end of the typical ranges and/or indicate the development of an upward trend.

The composition of the landfill gas is measured using a meter. The gas sample is obtained through the static pressure port on the meter, and the percent methane, carbon dioxide, oxygen, and balance gas (predominately nitrogen) is reported.

4.4 Flow Rates

The landfill gas extraction flow rates will be different at each monitoring point and will vary with barometric pressure changes as well as landfill cover condition (i.e., whether geomembrane has been constructed over the extraction location or the condition of the intermediate cover). Likewise, the cumulative landfill gas flow rate at the flare will vary with time. The flow rates can be adjusted, if needed, based on the concentrations of the various constituents as described above. As the operating record of the system becomes established over time, "normal" flow ranges for the individual extraction points as well as the system may be established.

The landfill gas flow rate is calculated from differential pressure readings obtained at the wellheads. The portable gas meter can be programmed to directly correlate differential pressure to flow rate.

4.5 Pressures

Landfill gas pressure will vary throughout the system at any given time, and will vary with varying extraction rates. The pressures at the extraction points should be negative (vacuum) to provide active extraction from that point. If the pressure is positive, then adjustments should be made to increase the flow rate, provided that the methane concentration is at least 30 percent with no more than five percent oxygen.

The static pressure at the landfill gas extraction points and along the conveyance pipe is obtained by connecting the meter to the static pressure ports at the monitoring locations.

4.6 Temperature

Landfill gas temperature at the extraction points is obtained directly from thermometers installed on the wellhead assemblies or from a meter using a thermocouple inserted into the wellhead thermometer port. A LFG temperature greater than 150^{o}F is considered excessively high for anaerobic degradation at JRL. When the measured temperature is greater than 150^{o}F, the wellhead valve opening should be reduced to decrease flow and temperature.

The default LFG temperature limit in New Source Performance Standards (NSPS) is 55 °C (131 °F), but the NSPS also provides for a higher operating temperature based on data demonstrating the elevated temperature does not cause fires or significantly inhibit anaerobic decomposition by killing methanogens. The historical data collection at JRL indicates that a higher operating temperature of 150°F is required to collect gas generated in the landfill. Reducing the LFG flows at wellheads with elevated temperatures has not reduced gas temperatures to less than 131 °F in several areas of the waste, and based on discussions with MEDEP, anaerobic decomposition for the waste mix at JRL has been demonstrated at up to 150°F.

5.0 MAINTENANCE AND TROUBLESHOOTING

5.1 Collection System Maintenance

LFG system maintenance involves the following:

- Repairs to the conveyance pipes due to damage caused by accident, settlement, environmental factors, and aging;
- Repair or replacement of system components (e.g., wellheads, access ports, flex hoses, valves, condensate traps, etc.);
- Excavation for repair of damaged pipe and components; and
- Repairing and re-adjusting pipe supports and anchors.

In many instances, repairing the system may require shutting down the flare or certain sections of the LFG collection system. The duration of the shutdown should be kept to a minimum; where possible, the work should be scheduled to coincide with scheduled flare shutdowns.

The sulfur removal system has a 95 percent uptime requirement and records are maintained to demonstrate that there is no more than 438 hours of downtime per 12-month period.

5.2 Landfill Surface

The landfill cover is an integral part of the landfill gas management system. Proper landfill cover maintenance practices are important for effective operation and performance of the landfill gas management system. Experience has shown that in most cases, proper cover maintenance in conjunction with timely installation of active landfill gas management system components will address most landfill odor problems. A visual inspection is helpful in identifying rips, tears, pinholes, cracks, fissures, or bare spots in the synthetic and soil cover systems. Damaged areas should be evaluated and repaired as soon as practical.

5.3 Conveyance Pipe

Over time, the conveyance pipe may develop air leaks. Air leakage should be limited to the degree practicable and it is recommended that oxygen not exceed 3.5 percent by volume in landfill gas in the conveyance pipe. (Under normal operations, the landfill gas should contain no more than one percent oxygen.)

Leaks can occur in above-grade pipe systems due to pipe separations caused by thermal contraction resulting from cooling at night and during a system shutdown. Separations or damage can occur to below-grade pipe due to differential settlement. Buried pipe is also subject to expansion and contraction, but to a lesser degree because of a more uniform temperature and the anchoring effect of the soil support within the trench.

Solid conveyance pipes are typically installed with at least 5 percent slope within the landfill footprint in order to reduce the potential for liquid (condensate) blockage in pipes due to differential settlement. Increased pitch (i.e., greater than 5 percent) is provided, if practical. Other factors, such as constraints encountered during construction, may limit slopes on pipes. Blockages in pipes can be evaluated by installing access ports for monitoring. With buried systems, it may be necessary to "pot hole" (i.e., perform exploratory excavation) with a backhoe to install access ports for monitoring. If the main conveyance pipe becomes blocked or restricted with condensate, then either re-establishing the slope of the pipe or installing an additional condensate trap and drain should rectify the condition.

Landfill gas conveyance pipes will be replaced prior to final closure of the landfill, and as such, will be located just below the final cover system. These conveyance pipes, installed close to final grade, are not expected to fail. However, if replacement or access to pipes becomes necessary, repairs should be designed to reduce impacts to the final cover system and to restore the cover system to match final closure conditions. The Maine Department of Environmental Protection should be notified prior to impacting the final cover system, and repairs efforts should be documented.

5.4 Blower(s)/Flare Station Maintenance

A brief discussion of general maintenance requirements for specific equipment follows. For more detailed information refer to the applicable manufacturer's information. Table 4 describes the monitoring checklist for the blower(s)/flare station.

5.4.1 Pipe and Fittings

Conveyance pipe and fittings installed at JRL are primarily high-density polyethylene (HDPE), with Polyvinyl Chloride (PVC) pipe installed in vertical extraction wells.

Process plant pipe and fittings may consist of both steel and thermoplastic. Both are durable long-lived materials. However, thermoplastic pipe can be subject to damage from shock, strain, or heat. Thermoplastic pipe should not be used near sources of extreme heat such as the flare. Carbon steel piping can erode and corrode. Stainless steel, cast iron, and aluminum piping have all been successfully used in landfill gas applications.

Landfill gas and condensate exhibit corrosive properties. The presence of oxygen, carbon dioxide, and organic acids common to landfill gas can be present in landfill gas condensate. The combination of these constituents can promote corrosion of steel pipe that carries landfill gas condensate. The most common point of wear due to erosion or corrosion in steel pipe in landfill applications is typically at pipe elbows and other fittings that are subject to erosion and where liquid condensate accumulates. Theses pipes, where they occur, should be inspected for evidence of corrosion (leakage, particularly at fittings). Where questions of pipe integrity exist, ultrasonic thickness testing may be performed, if necessary.

5.4.2 Valves

LFG flow control valves installed at JRL are primarily gear operated butterfly valves with stem extensions. Gaskets are typically Viton. Joint hardware is typically Type 304 stainless steel. Other valve types and components may be present. Valve seats and stem seals may wear and eventually require replacement. Butterfly valves with elastometer seals, such as Buna-N or EPDM, may be affected by landfill gas. In such cases, it may be necessary to replace with Viton or Teflon™. Viton or Teflon™ valve seats in butterfly and ball valves, respectively, have demonstrated reasonably good performance.

5.4.3 Blower Maintenance

Routine maintenance for blowers and motors consists of listening for signs of abnormal operating conditions, monitoring for excessive vibration or temperature, and draining condensate from the blower housing periodically and before startup (if not automatically drained). The blower drive belt tension and wear should be checked on a monthly basis. If belts are glazed or cracked, then they should be replaced. At least one spare set of matched belts should always be on hand. On direct drive machines, flexible coupling alignment should be checked on initial setup and periodically as recommended by the manufacturer (typically quarterly to annually). Bearings should be greased or repacked according to the manufacturers' recommendations, (typically quarterly to annually). The electric drive motor, if not equipped with sealed bearings, should also be greased. Blower seals and packing should be checked periodically for leakage. If blower seals continually leak or will not last, then consult the manufacturer or try another type of seal or seal material. Consult the manufacturer's literature for detailed information on the maintenance of the blower.

5.4.4 Gas Inlet Automatic Block Valve

An automatic block valve shuts off the flow of landfill gas to the blower(s)/flare station when the flare is not operating or when a fault or shutdown is initiated. This is a butterfly type valve and should be serviced only when a need is indicated. The automatic block valve seat should maintain a gas tight seal whenever the valve is in the closed position. Refer to the manufacturer's literature for information on service.

5.4.5 Flame Arrester

A flame arrester is designed to prevent the migration of burning landfill gas from the flare backwards into the process pipe and the flare station. This condition can cause what is known as "detonation" (an explosion) or "flashback" within the pipe network. A flame arrester will only work properly if the velocity and pressure of the flashback in the pipe is within the allowable range and the flame arrester is properly assembled.

The differential pressure across the flame arrester should be checked during monthly monitoring. The normal differential pressure is typically less than 0.5 inches water column (w.c.). The differential pressure across the flame arrester should not exceed 1.0-inch w.c. If excessive differential pressure is observed, then the flame arrester should be serviced. To service the flame arrester, shut down the blower/flare station and block in the flame arrester upstream using the manual and automatic block valves (**verify the valves do not leak**). Closely follow the directions in the manufacturer's literature for maintenance and re-assembly of the flame arrester element. It is important to note that a flame arrester's effectiveness is based upon a design spacing or gap in the flame arrester element. During maintenance and re-assembly, this spacing must be maintained according to the manufacturer's original specifications if the flame arrester is to function as designed. Ensure that all parts are returned and in the proper orientation when re-assembling the flame arrester.

5.4.6 Flare

Operation and maintenance of the flare is straightforward and consists of maintaining proper fuel pressure, maintaining the igniter system, and keeping the flare drained of condensate. Proper fuel velocity, quality, mixing, and flame condition are key to performance. Also, to operate consistently, the flare burner assembly must be adequately shielded from excessive wind. Problems with flame stability in an open flare are usually caused by poor landfill gas quality.

The primary wear on flares is due to thermal stress. If the flare is operated in an imbalanced condition or at excessively high temperature, then it will exhibit signs of accelerated thermal stress. This may be indicated by wear and deformation of the burner.

The flare may require burner adjustment or modification to achieve and maintain proper combustion performance. Adjustment of the flare may involve changing an orifice or burner ring, or moving or changing a plate. Consult the manufacturer or seek qualified assistance.

Landfill gas velocity to the flare is adjusted at the blower/flare station inlet. This is accomplished by balancing the distribution of the blower's total pressure, so that there is sufficient fuel pressure at the flare. In severe cases where adjustment will not work, the flare burner or the blowers may need to be modified or replaced. **Never modify the flame arrester to increase gas velocity.**

The flame safeguard sensor system consists of an electronic controller mounted in the control panel and a thermocouple mounted near the tip of the flare. At least one spare thermocouple should be kept on the site.

A thermocouple can be checked independently with a voltmeter and thermocouple tables or with a digital thermocouple test meter or digital thermometer. Proper polarity must be observed when installing and monitoring a thermocouple. If the thermocouple is subjected to flame impingement, then its life may be shortened considerably. Thermocouples can be mounted in protective sheathing; however, this will cause some delay in response to temperature changes.

See the flare manufacturer's literature for specific component operation, maintenance, and troubleshooting information.

5.4.7 Electrical Equipment Controls and Instrumentation

Dust may accumulate in electrical cabinets and absorb moisture from the air. Over sufficient time, a conductive path can be created that can cause a failure. Connections may also become loose due to thermal expansion and contraction. Electrical service and control cabinets should be cleaned on an annual basis. Wire connections should be checked and tightened throughout the cabinets annually. Calibration and verification of instrumentation gages and thermocouples should be performed annually. Shutdown alarms and devices should be tested and the results recorded in a log. Thermocouples for sensing flare stack temperature should be maintained, and replaced when they no longer perform properly. This is normally evident by failure of the temperature controller to properly read or control the flare stack temperature (usually due to an open junction) or by loose or corroded connection terminals at the temperature transmitter, connector block, or temperature controller.

Test and recalibrate instruments, fault protection, and shutdown devices. Large breakers or fused disconnects should be disconnected under load. Ensure high voltage breakers or disconnects are "locked out" in accordance with 29 CFR 1910.147 and 1910.333 Subpart S before working on equipment associated with them. Fuses should be physically pulled to isolate equipment. When the blower/flare station will be down for major maintenance or stand down, large breakers should be locked out (i.e., physically separated and disconnected) and the fuses should be pulled.

5.4.8 Lubrication - General

Follow the manufacturer's recommendations for specific types and brands of lubricants. It is important to use the recommended type of grease and not to mix types or brands of grease. Do not over lubricate. Personnel who perform lubrication services should be knowledgeable in lubrication practices and should follow the manufacturer's instructions for lubrication requirements.

Establish an initial lubrication frequency based upon the equipment manufacturer's recommendation. It may be necessary to adjust the lubrication frequency interval based upon experience with the equipment.

5.4.9 Other Equipment Maintenance and Operating Tips

The system operator should be proactive, remain alert, and develop a habit of observing equipment.

Equipment noises (such as bearings) may be monitored using an equipment stethoscope or using a wrench or similar tool by placing it on the equipment and placing the opposite end of the tool against the bone in front of the ear to listen. It is important to develop a sense of what baseline conditions are for comparison. The smell of leaking landfill gas or burned lubricant can indicate a seal, component, or lubricant failure.

When checking motors or other electrical devices for temperature by feel, the back of the hand should be used. Approach the equipment slowly and feel for radiant heat, which would indicate a very high temperature. If the equipment is too hot to maintain hand contact, then it is at or above a threshold of about 60 to 63°C (140 to 145°F) and may be considered excessive in many cases depending upon the equipment and service. The reason for using the back of the hand is that it is more heat sensitive and in the case of electrical fault to the casing, the natural reaction will be for the muscles of the arm to contract away from the device. This can prevent electrocution.

Operating personnel should wear all cotton clothing that provides some degree of protection in gas flash fires. Some synthetics such as polyester blends will melt readily, which can be fatal. Ties or loose items (e.g., identification badges hanging around the neck, etc.) should never be worn around rotating or belt-driven equipment. All watches, rings, identification bracelets, etc., should be removed when performing electrical testing or troubleshooting.

It is important that maintenance supplies, lubricants, and spare parts be inventoried on a frequent basis to ensure that adequate stocks are maintained for when they will be needed. Supplies should be reordered and restocked as used.

5.4.10 Condensate Handling Systems

Condensate is managed using traps constructed inside the limit of waste, and condensate knockout structures constructed outside the limit of waste. The condensate knockout structures collect condensate in perimeter landfill gas conveyance pipes located outside the limit of waste and discharge the condensate into primary leachate collection system cleanouts. The condensate traps inside the limit of waste drain by gravity into primary leachate collection system cleanouts or in some cases drain back into the waste mass. The blower/flare station is fitted with various traps, drains, valves, and pumps for handling condensate. Condensate handling components installed for LFG treatment systems (i.e., Sulfatreat & Thiopaq) should also be inspected and maintained periodically. Condensate can be corrosive and the equipment should be checked carefully and frequently for the effects of corrosion. Seals, o-rings, and valves are usually high maintenance items. Refer to the manufacturer's information for maintenance of individual equipment or components.

6.0 COLD WEATHER OPERATIONAL CONSIDERATIONS

Condensate in exposed pipes and equipment or in below-grade pipes without adequate soil cover may freeze during winter operations. Care must be taken to limit the amount of condensate allowed to collect in the landfill gas system at any time. The landfill gas is a source of heat for the system. If the system shuts down for a short period during winter months, then condensate in the exposed portions of the system may freeze.

If freezing of condensate under normal operating conditions becomes a regular problem, then heat trace and insulate the affected areas.

7.0 DATA COLLECTION

Data will be collected routinely using a meter for the flare, the well field, structures, etc. These "readings" are transferred to a computer and then uploaded to a secure database website. The database can be accessed by approved landfill personnel. Data is also collected via the supervisory control and data acquisition (SCADA) system for the flare and Thiopaq system.

7.1 Data Assessment

LFG measurements at each wellhead may be compared with previous data to assess relative performance. Persons assessing the data should be aware of the normal operating range for each parameter, note changes, and assess their causes. Equipment deterioration that can be sudden or gradual may be indicated by an abnormal monitoring result. Such indications should be promptly investigated. Data should fall within established parameters for normal operating ranges by comparing monitoring data with previous readings and target criteria for pressure (vacuum), methane (45 percent or more), oxygen (five percent or less), and temperature (150°F or less).

7.2 Landfill Daily Log Book

Whenever the system is monitored for any reason, staff should make appropriate entries in the Daily Log Book stored in the landfill office.

- Name of person making the entry;
- Date and time;
- Reason for the monitoring (e.g., routine, shutdown, specific monitoring or maintenance activity, etc.);
- Reason for any shutdown;
- Actions taken or adjustments made;
- Equipment status upon leaving; and
- Unusual observations made.

The daily log is used as a record of events regarding the landfill and to communicate between operating personnel. The log entry also becomes part of the daily landfill readings.

7.3 Data Collection Routine

Data are collected manually at the individual data points (i.e., at wells on the landfill, etc.).

Equipment used for data collection includes:

- Portable Gas Analyzer;
- Calibration gases (use before going into the field);
- Data reading sheets;
- Field book;
- Site map of the data points;
- Carrying tray, toolbox, or backpack, etc.;
- Tools needed to access the system components; and
- Spare parts for maintenance such as access ports, plugs, etc.

8.0 APPROVAL PROCEDURES FOR LANDFILL MANAGEMENT SYSTEM INSTALLATION

Prior to the installation of new landfill gas management system infrastructure within the landfill, various procedures must be followed to assure that the proposed system modification is properly designed and approved. This section defines the procedures to be followed prior to the modifying the landfill gas management system.

8.1 System Design

The landfill gas management system structures, conveyance pipes, and condensate management structures at JRL are designed by Sanborn, Head & Associates, Inc. (Sanborn Head) using sound engineering principles that follow industry standard procedures. The blower/flare station is sized according to projected landfill gas flow rates.

NEWSME routinely retains Sanborn Head to prepare detailed design packages for the expansion of the landfill gas management system associated with each new operational cell to comply with Condition 15.B of Solid Waste Order #S-020700-WD-N-A. The detailed design package typically includes the following:

- A description of the basis for the design;
- Drawings;
- Specifications; and
- Quality assurance and quality control information.

The landfill gas system expansion design for each cell is based on the projected development plans as prepared by Sevee & Maher Engineers, Inc. and a yearly review of landfill gas generation rates performed by Sanborn Head. The landfill gas expansion design is then submitted to the MEDEP for review and approval.

Required changes to the existing LFG infrastructure are occasionally identified and proposed by NEWSME as the landfill gas generation rate and the disposal capacity needs are reviewed. In addition, NEWSME may propose changes to the approved landfill gas management system design to address observed conditions that may require modifications or additions to the system.

8.2 Approval Procedures

Procedures to modify approved landfill gas management system designs fall into two categories referred to as Major and Minor Revisions. A discussion of these revisions are discussed in Sections 8.3 and 8.4 below and depicted in the flow chart provided as Figure 3.

8.3 Approval of Major Revisions to Approved Design

Major revisions are defined as a modification that affects the design or operation of the gas management system and can include such projects as:

- The addition of vertical extraction wells;
- The installation or rerouting of conveyance pipes;
- Changes to the condensate management design; and
- The addition of gas collection trenches.

Major revisions to approved design projects will be handled as a Change Order pursuant to 06-096 CMR 401-3.D of the Maine Solid Waste Regulations. Prior to submitting a formal change order request, NEWSME will contact the MEDEP and describe the issue (orally) and the proposed remedy (i.e., construct additional wells). Appropriate sketches will be provided as necessary. NEWSME will identify the anticipated time frame for construction and the name of the qualifying person who will oversee the construction. Following this step, the proposed (MEDEP agreed upon) changes will be provided to Sanborn Head, who will revise the drawings to include the location of additional wells and associated conveyance pipe, to include well depth information, etc. NEWSME will submit a written change order request to the MEDEP for review and approval at least five business days prior to the planned construction, unless an alternate deadline has been agreed upon with the MEDEP. The MEDEP will issue a response to the change order request within five business days or approval of the Change Order is automatically granted.

8.4 Approval of Minor Revisions to Approved Design

Minor revisions to the design are defined as modifications that do not significantly affect the design or operation of the landfill gas management system and can include projects such as:

- Minor shifting of a previously approved trench;
- The addition of a pipe intended to bypass a "water-out" or non-functioning section of an existing trench; and
- The addition of a short stub to an existing gas collection trench.

These modifications do not typically require the installation of an additional wellhead and are often a means of addressing a concern in a section of the operational landfill area.

For major changes to the design of the landfill gas management system that require MEDEP approval, NEWSME will notify the MEDEP and describe the need for the change, the location, and how the structure will be connected to the existing infrastructure. A hand sketch will be provided as necessary.

9.0 INFRASTRUCTURE CONSTRUCTION AND DOCUMENTATION

Construction of landfill gas infrastructure will be peformed by qualified NEWSME staff and specialty contractors when needed. This section describes the components of the landfill gas management system that can be installed by NEWSME staff and the system components that require specialty contractors.

9.1 Construction Projects by NEWSME Staff

Qualified NEWSME staff is authorized to install the below listed infrastructure:

- Gas collection trenches;
- Conveyance pipe within the solid waste boundary;
- Condensate structures located within the solid waste boundary; and
- Wellheads on new wells and trenches and replacement wellhead fixtures.

NEWSME staff will follow an approved set of technical specifications for each project and qualified personnel will document the construction using field survey techniques. Following construction, field survey data will be provided to Sanborn Head so that the as-built drawings may be updated. Updated as-built drawings will be provided to the MEDEP as part of the annual report. As-built drawings pertaining to new wellhead installations will be provided to the MEDEP within 45 days of completion of work.

9.2 Construction Projects by Specialty Contractors

Specialty contractors will be retained to perform the following installations:

- Vertical extraction wells;
- Conveyance pipes outside of the solid waste boundaries; and
- Condensate pipe and structures outside of the solid waste boundary.

Following the construction of the above infrastructure, updated information will be provided to Sanborn Head and as-built plans provided to the MEDEP within 45 days of completion of work.

NEWSME staff may also elect to perform the above installations with third party oversight.

9.3 Construction Quality Assurance and Quality Control

Construction activities by NEWSME staff as listed in Section 9.1 above (with the exception of LFG conveyance pipe installations) will be overseen and documented by qualified NEWSME staff.

Construction of vertical extraction wells, conveyance pipes of 12-inches diameter or greater, and condensate pipe and structures outside of the solid waste boundary will be overseen by qualified construction quality assurance personnel separate from NEWSME and the installation contractor following an approved CQA plan. The construction will be documented and the information will be submitted to the MEDEP within 45 days of completion of work.

9.4 Licensing of LFG Infrastructure Installations

Proposals for new gas related projects will be submitted to the MEDEP in the form of a minor revision application pursuant to 06-096 CMR 400.3.B(2)(b) of the Maine Solid Waste Management Regulations, except that if a major redesign of the gas extraction system is being proposed, the MEDEP may require an amendment application be submitted. For projects related to new cell construction, including the layout for proposed gas collection trenches, the landfill gas management system design will be included with the application for the new cell construction. The MEDEP may include comments on the proposed landfill gas management system design as part of its review of the new cell design. MEDEP's review will be completed prior to the construction of the new cell.

9.5 Emergency Situations

In an after-hours emergency, such as vandalism or a catastrophic failure, that causes damage and/or shuts down the landfill gas management system, NEWSME will immediately notify the proper parties by all means (office, home, DEP spill response line) to notify them of any proposed activities associated with abating the condition. It is understood by MEDEP staff that any work required to get the landfill gas management system operating again will proceed as needed.

P:\2500s\2536.27\Source Files\Response to MEDEP Comments\Attachments\SH-8 - O+M plan\20160225 O&M plan.docx

TABLE 1

TYPICAL LANDFILL GAS CONSTITUENTS

Operations and Maintenance Manual
Juniper Ridge Landfill
Old Town, Maine

COMPONENT	**PERCENT VOLUME** **(All are stated on a dry basis except moisture.)**
Methane (CH_4)	30 to 50%.
Carbon Dioxide (CO_2)	20 to 40%.
Oxygen (O_2)	0 to 5%.
Balance Gas Including Nitrogen (N_2)	10 to 50%.
Hydrogen (H_2)	Trace to 5% plus; generally less than 1%.
Carbon Monoxide (CO)	Trace; CO is an indicator of the possible presence of a subsurface fire.
Hydrogen Sulfide (H_2S) & Other Sulfur Components	Treated to 1,000 ppmv or less (5,000 to 15,000 ppmv untreated)
Moisture	Up to 14% (increases with gas temperature).
Volatile Organic Compounds (VOCs)	Less than 2%; typically ¼ to ½%.

Note: This table represents the typical characteristics of landfill gas at the Juniper Ridge Landfill based on data collected at the site.

TABLE 2

RECOMMENDED MONITORING SCHEDULE

Operations and Maintenance Manual
Juniper Ridge Landfill
Old Town, Maine

ITEM	FREQUENCY	PARAMETER
Wellheads	Monthly	• Condition of flex hose; • Valve position; • Gas flow rate; • Static Pressure; • Percent methane; • Percent carbon dioxide; • Percent oxygen; and • Temperature of gas.
Blower/Flare Station	Weekly	• Condition of pipe and connections; • Consistency of flame; and • Functioning of ignition sparker.
Conveyance Pipe	Bi-Annual	• General condition of exposed pipe.

Notes:

1. The monitoring frequency may be reduced with approval from the Maine Department of Environmental Protection.

2. In addition to the monitoring schedule outlined above mechanical components of the blower/flare station should also be monitored and serviced in accordance with the manufacturer's instructions.

TABLE 3

GAS EXTRACTION SYSTEM MONITORING CHECKLIST

Operations and Maintenance Manual
Juniper Ridge Landfill
Old Town, Maine

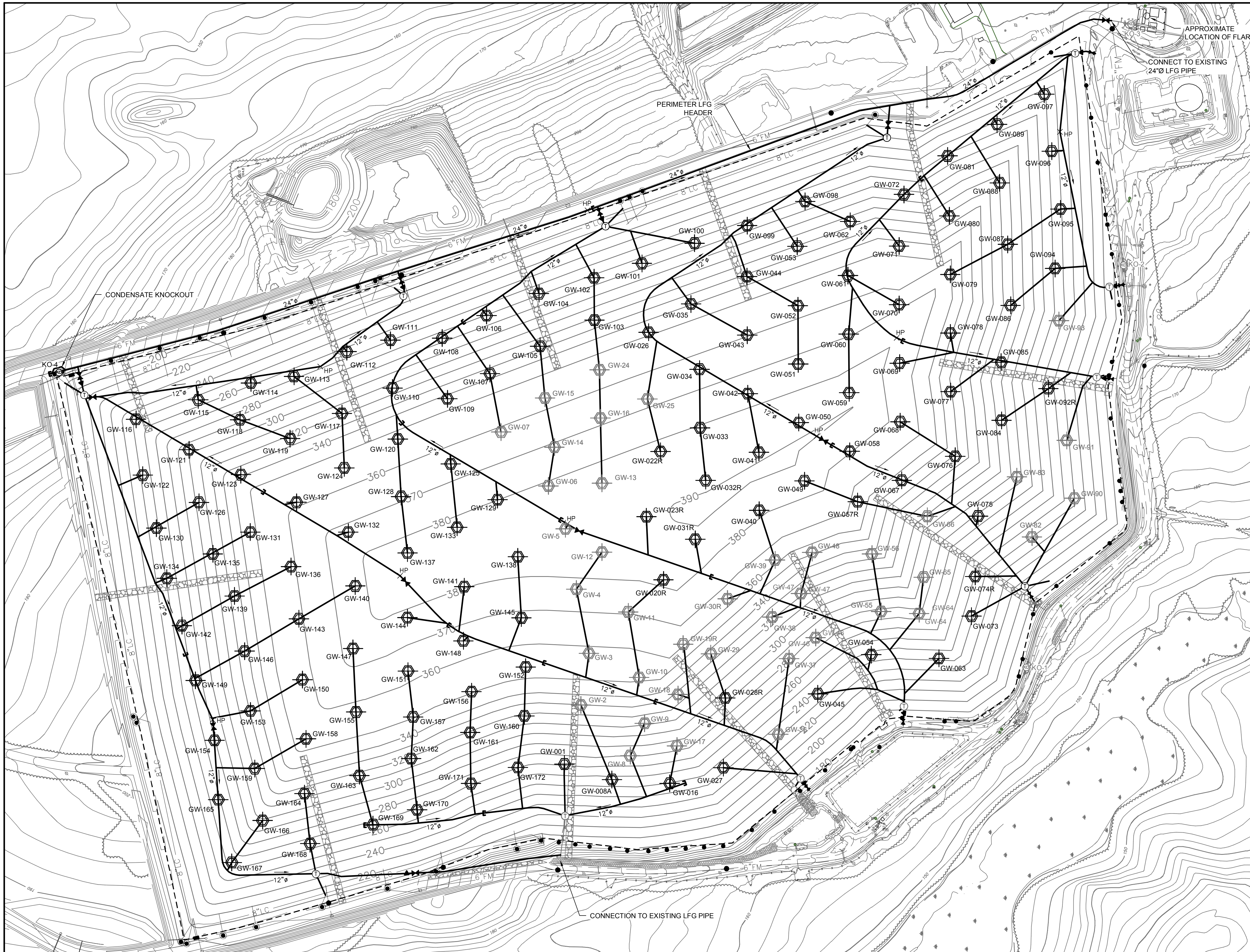
A. <u>Prior to going out onto the landfill.</u>

1. Calibrate the GEM-2000 or GEM-5000 Portable Gas Analyzer (meter) using methane, carbon dioxide, and oxygen.
2. Calibrate the pressure transducers by performing the "Zero Pressure" function.
3. Record the ambient weather conditions including:
   - Temperature;
   - Barometric pressure;
   - Wind speed and direction;
   - Precipitation amounts; and
   - Current observations (i.e., drizzling, raining, snowing).

B. <u>At each wellhead assembly.</u>

1. Connect the striped tubing with the external filter/water trap assembly from the static pressure/sampling port on the meter to the static pressure port on the wellhead assembly.
2. Connect the clear tubing between the impact pressure port on the meter and the impact pressure port on the wellhead assembly.
3. Perform the "Read Gas Levels" function on the meter. Follow instructions on the meter.
4. Record the following data on the data sheets or in the meter memory:
   - Station identification;
   - Percent methane;
   - Percent carbon dioxide;
   - Percent oxygen;
   - Percent balance;
   - Percent LEL;
   - Temperature of the gas stream;
   - Static pressure;
   - Differential pressure;
   - Gas flow rate; and
   - Control valve percent open.

5. Make adjustments to the flow rate by adjusting the wellhead control valve, if required.



NOTES:

1. THE BASE MAP SHOWN WAS PREPARED BY AERIAL SURVEY & PHOTO INC., OF NORRIDGEWOCK, MAINE. PHOTO DATE JULY 31, 2014. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE.
2. PROPOSED EXPANSION GRADES SHOWN WERE PROVIDED TO SANBORN HEAD BY SEVEE & MAHER, (SME) INC. OF CUMBERLAND, MAINE.
3. ACTUAL GRADES MAY DIFFER FROM GRADES SHOWN ON DRAWINGS AT THE TIME OF CONSTRUCTION.
4. THE EXISTING LANDFILL GAS EXTRACTION SYSTEM INFRASTRUCTURE FEATURES SHOWN ARE BASED ON A COMBINATION OF DESIGN AND AS-BUILT DOCUMENTATION AVAILABLE TO SANBORN, HEAD & ASSOCIATES, INC. (SANBORN HEAD). ACTUAL LOCATIONS OF INDIVIDUAL FEATURES MAY BE DIFFERENT THAN SHOWN.
5. THE LOCATIONS OF MANY OF THE LANDFILL DESIGN COMPONENTS SHOWN ON THIS PLAN, SUCH AS LEACHATE CLEANOUTS, STORMWATER MANAGEMENT FEATURES, AND UTILITIES, ARE BASED ON PROPOSED LOCATIONS PROVIDED TO SANBORN HEAD BY SEVEE & MAHER ENGINEERS, INC. OF CUMBERLAND, MAINE.
6. THIS PLAN IS INTENDED TO ILLUSTRATE THE PROPOSED LAYOUT OF THE LANDFILL GAS (LFG) EXTRACTION SYSTEM. ACTUAL LOCATION OF WELLS, PIPE, AND VALVES MAY CHANGE DEPENDING ON SITE CONDITIONS AND CONSTRAINTS DURING CONSTRUCTION.
7. SOLID LANDFILL GAS CONVEYANCE PIPE AND COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 6 PERCENT. PERFORATED LANDFILL GAS COLLECTION TRENCHES SHALL BE INSTALLED AT A MINIMUM SLOPE OF 2 PERCENT.
8. HDPE PIPE AND FITTINGS SHALL BE SDR-17.
9. HDPE FITTINGS SHALL BE MOLDED; FABRICATED FITTINGS SHALL NOT BE USED.

LEGEND:

EXISTING		PROPOSED
180	10 FOOT CONTOUR	370
	2 FOOT CONTOUR	
GW-11	VERTICAL LFG WELL	GW-35
12"ø	LFG CONVEYANCE PIPE (SIZE AND SLOPE DIRECTION)	12"ø
T	CONDENSATE TRAP	T
	CONTROL VALVE	
KO-1	CONDENSATE KNOCKOUT	KO-4
	LEACHATE COLLECTION CLEANOUT	
	VERTICAL RISER	
	LIMIT OF WASTE CONTAINMENT	
	CELL LIMIT	
	EDGE OF ROAD	
HP	HIGH POINT	HP
	RIPRAP-LINED DOWNCHUTE	
	TEMPORARY PIPE TERMINATION	

SANBORN HEAD

GRAPHICAL SCALE

150' 75' 0' 150' 300'

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: APRIL 2015

LFG SYSTEM EXPANSION MASTER PLAN
JUNIPER RIDGE LANDFILL
OLD TOWN, MAINE

JRL LFG INFRASTRUCTURE PLAN (FULL BUILD-OUT)

PROJECT NUMBER: 2536.27

FIGURE NUMBER: 1

MAINE STATE PLANE

CURRENT ACTIVE LFG COLLECTION SOURCES:

VERTICAL WELLS
GW-02
GW-03
GW-04
GW-05
GW-06
GW-09
GW-10
GW-11
GW-12
GW-13
GW-14
GW-15
GW-16
GW-18
GW-19R
GW-20R
GW-21
GW-22R
GW-23R
GW-24
GW-25
GW-28
GW-29
GW-30R
GW-31R
GW-32
GW-36
GW-37
GW-38
GW-39
GW-46
GW-47
GW-48
GW-54
GW-55
GW-56
GW-57
GW-64
GW-65
GW-66
GW--7
GW-74
GW-75
GW-82
GW-83
GW-90
GW-91
GW--A
GW--D
GW--E
GW--F
GW-G2
GW-H2
GW--I
GW--J
GW--K
GW--L
GW--M
GW--N
GW--O
GW--P
GW--S
GW--T

HORIZONTAL COLLECTION TRENCHES
GCT-01
GCT-17
GCT-18
GCT-201
GCT-2A1
GCT-2A2
GCT-3A1
GCT-3A2
GCT-3A3
GCT-3A4
GCT-3A5
GCT-3B1
GCT-3B2
GCT-3B3
GCT-3B4
GCT-401
GCT-401A
GCT-402
GCT-402A
GCT-403
GCT-404
GCT-404A
GCT-405A
GCT-406
GCT-406A
GCT-501
GCT-502
GCT-503
GCT-504
GCT-505
GCT-506
GCT-507
GCT-508
GCT-509
GCT-510
GCT-511
GCT-512
GCT-513
GCT-514
GCT-601
GCT-602
GCT-603
GCT-604
GCT-605
GCT-606
GCT-607
GCT-608
GCT-609
GCT-610
GCT-701
GCT-702
GCT-703
GCT-704
GCT-705
GCT-706
GCT-707
GCT-708
GCT-709
GCT-710
GCT-711
GCT-801
GCT-802
GCT-803
GCT-804
GCT-805
GCT-806
GCT-807
GCT-808
GCT-809
GCT-810
GCT-811
GCT-812
GCT-813
GCT-814
GCT-815
GCT-816
GCT-817
GCT-818
GCT-819

OTHER
3W-01
3W-02
7SOUTH
L7WEST
LC--5
LC--6
LGV401
LGV402
LGV403
LGV404
LPC-1
LPC-3
LPC-4
LPC4A
LPC-2

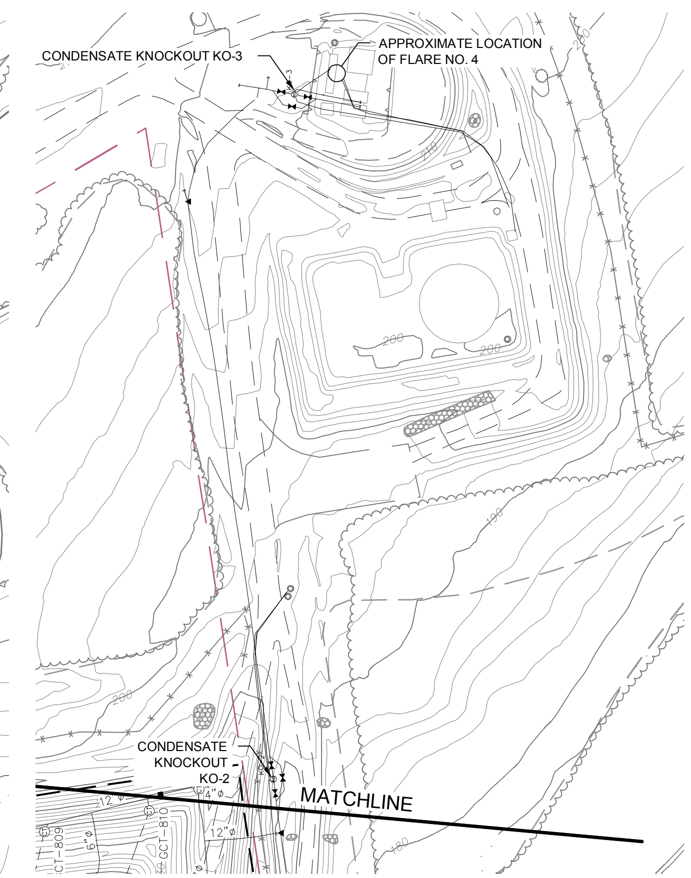
NOTES:

1. THE EXISTING LANDFILL GAS EXTRACTION SYSTEM INFRASTRUCTURE FEATURES SHOWN ARE BASED ON A COMBINATION OF DESIGN AND AS-BUILT DOCUMENTATION AVAILABLE TO SANBORN, HEAD & ASSOCIATES, INC. (SANBORN HEAD). ACTUAL LOCATIONS OF INDIVIDUAL FEATURES MAY BE DIFFERENT THAN SHOWN.
2. BASE MAP WAS PREPARED BY AERIAL SURVEY & PHOTO INC., OF NORRIDGEWOCK, MAINE. PHOTO DATE JULY 31, 2014. VERTICAL DATUM: BRASS PLUG AT PUMP STATION. HORIZONTAL DATUM: MAINE STATE COORDINATES EAST ZONE NAD 83. GROUND CONTROL BY PLISGA & DAY LAND SURVEYORS, BANGOR, MAINE.

LEGEND:

EXISTING

- 190 — 10-FOOT CONTOUR
- 2-FOOT CONTOUR
- LIMIT OF WASTE CONTAINMENT
- CELL LIMIT
- EDGE OF ROAD
- LANDFILL GAS CONVEYANCE PIPE
- LANDFILL GAS COLLECTION TRENCH (PERFORATED PIPE)
- LIMIT OF MARSH
- FENCE LINE
- GW-9 — LANDFILL GAS EXTRACTION WELL
- GCT-21 — COLLECTION TRENCH WELLHEAD
- COLLECTION TRENCH TERMINATION
- PIPE END CAP
- LEACHATE COLLECTION PIPE CLEANOUT
- LEACHATE COLLECTION INLET
- LANDFILL GAS EXTRACTION WELLHEAD
- RIPRAP AREAS



SANBORN HEAD

GRAPHICAL SCALE
100' 50' 0 100' 200'

NO.	DATE	DESCRIPTION	BY

DRAWN BY: R. CLAY
DESIGNED BY: R. CLAY
REVIEWED BY: E. STEINHAUSER
PROJECT MGR: E. STEINHAUSER
PIC: E. STEINHAUSER
DATE: APRIL 2015

LFG SYSTEM EXPANSION MASTER PLAN

JUNIPER RIDGE LANDFILL

OLD TOWN, MAINE

JRL LFG INFRASTRUCTURE PLAN (AS-BUILT - CELLS 1-8)

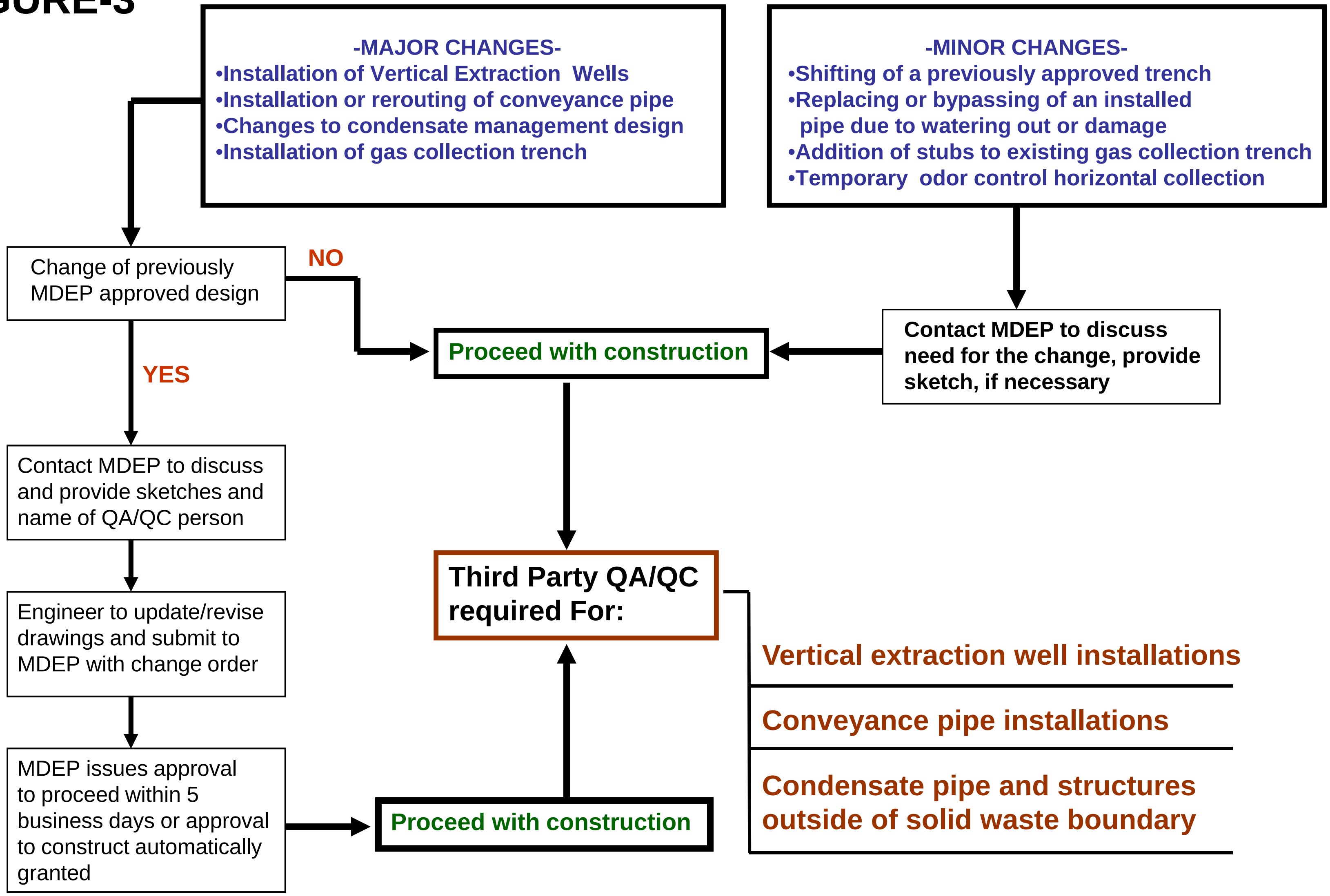
PROJECT NUMBER: 2536.27

FIGURE NUMBER: 2

IMAGES: D:\Steinhauser-ME.tif
C:\program files (x86)\dotsoft\toolpac\WIPEOUT.BMP

XREFS: P:\2500s\2536.27\Graphics Files\CAD\LFG Master Plan\XRefs\OT-LF-07-31-14.dwg
S:\CONDATA\2300s\2343.01\Graphics Files\CAD\Work\Xrefs\20150114-LFG-ASBUILT-100.dwg
P:\2500s\2536.27\Graphics Files\CAD\LFG Master Plan\XRefs\20111026 PERIMETER HEADER AB.dwg

FIGURE-3



-PROTOCOL FOR LFG SYSTEMS INSTALLATION-