Prepared for:

Sarasota County

Solid Waste Operations

Central County Solid Waste Disposal Complex

4000 Knights Trail Road
Nokomis, Florida 34275

RESPONSE TO RAI NO. 1

APPLICATION FOR A PERMIT TO CONSTRUCT FLEXIBLE LEACHATE STORAGE CONTAINERS AT CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX

Prepared by:

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28 March 2007

Mr. Steven G. Morgan
Solid Waste Section
Florida Department of Environmental Protection
Southwest District
13051 North Telecom Parkway
Temple Terrace, Florida 33637

Subject: Response to Request for Additional Information No. 1
Sarasota CCSWDC Flexible Leachate Storage Containers Construction
Pending Permit No.: 130542-005-SC/08
Sarasota County, Florida

Dear Mr. Morgan:

On behalf of Sarasota County (County), Geosyntec Consultants (Geosyntec) has prepared this letter to address the Florida Department of Environmental Protection's (Department's) first request for additional information (RAI) for approval of the application to construct flexible leachate storage containers (FLSCs) at Central County Solid Waste Disposal Complex (CCSWDC) located in Sarasota County, Florida. The RAI was addressed to Mr. Frank Coggins of County in a letter dated 13 December 2006, which is included as Attachment 1.

On 22 February 2007, Dr. Juan Quiroz and Mr. Ayushman Gupta of Geosyntec met with Ms. Susan Pelz, Mr. Steve Morgan and Mr. John Morris of the Department to discuss several RAI No. 1 comments. The RAI comments below are addressed accordingly and reference the February 2007 meeting with the Department is noted as needed.

This response is intended to supplement the Permit Application to Construct submitted by Geosyntec on 13 November 2006 on behalf of County. Each RAI comment has been reproduced in italic font below and the corresponding response is given in normal font. In this response, deletions to the original document have been shown with a strikethrough and additions have been shown with an underline.

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are

included as Attachment 2. The Engineering Report (without appendices) titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included as Attachment 3.

GENERAL

1. ***Rule 62-701.320(8), F.A.C.*** *Please publish the attached Notice of Application and provide proof of publication to the Department.*

   Response 1:

   In accordance with Rule 62-701.320(8), F.A.C., the Notice of Application was published in the 20 March 2007 issue of the Sarasota Herald-Tribune, a local newspaper of general circulation in Sarasota County. The signed Affidavit of Publication from Sarasota Herald-Tribune is included in Attachment 23.

2. ***Rule 62-701.730(4) (b), F.A.C.*** *Responses to each of the items in John Morris' December 11, 2006 memorandum (attached) are required. You may call Mr. Morris at (813) 632-7600, extension 336, to discuss the items in his memorandum.*

   Response 2:

   The responses to each of Mr. Morris' items are provided at the end of this itemized list provided by the Department, under the heading titled "DEP Form No. 62-701.900(1), Solid Waste Management Facility Form."

3. ***Rule 62-701.410 (2) (e), F.A.C.*** *Please provide foundation bearing capacity and subgrade settlements analyses for the FLSC in accordance with Rule 62-701.410 (2) (e), F.A.C.*

   Response 3:

   On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting, it was agreed that foundation bearing capacity for the FLSCs was inherently addressed in the perimeter berm slope stability analyses which were provided in Appendix F of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal

Complex," dated November 2006. As a result, additional stability analyses for the FLSCs were not required.

During the 22 February 2007 meeting with the Department, it was agreed that the subgrade settlement analyses for the FLSCs would be provided for two points along the cell floor of the FLSCs. The differential settlement between the high-point and the low point of the FLSC cell floor would be evaluated to verify that grade reversal will not occur due to settlement of the subsurface soils. The resulting subgrade settlement calculations for the FLSCs are provided in Attachment 4. Based on the cell floor settlement calculations, the differential settlement (and thereby change in constructed slope) is negligible.

ENGINEERING REPORT (RULE 62-701.320(7) (d), F.A.C.)

4. ***§1.1:*** *Please provide a copy of the pending ERP permit for the storm water management system modification of the facility.*

Response 4:

A copy of the pending Environmental Resource Permit (ERP) application is included in Attachment 5. The ERP application was prepared by Geosyntec, on behalf of County, and submitted to the Department on 14 March 2007.

5. ***§3.2:***

   a. *The reference to the FLSC facility being built-up relative to the existing ground as shown on Sheet 3 of the permit drawings appears to be a typographic error. Please revise to reference Sheet 5 of the drawings.*

Response 5.a:

The Engineering Report (without appendices) titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included as Attachment 3. Section 3.2 was modified to appropriately reference Sheet 5 of the Permit Drawings, and is included in the revised Engineering Report.

b. *Please provide the supporting calculations for the stated 300,000 gallon storage capacity of each FLSC.*

Response 5.b:

The supporting calculations for the stated 300,000 gallon storage capacity of each FLSC cell are provided in Attachment 6.

6. *§3.3:*

a. *Neither the perimeter drainage channel nor weir details on the permit drawings or this section show or explain how the impacted stormwater is pumped from the drainage canal to the impacted stormwater pipeline. Please revise this section and the permit drawings to address this discrepancy.*

Response 6.a:

The Engineering Report (without appendices) titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included as Attachment 3. Section 3.3 was modified to explain how the impacted storm water is pumped from the drainage channel to the impacted storm water pipeline, and is included in the revised Engineering Report.

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. A note was added to Sheet 2 and a detail was added to Sheet 10 of the revised Permit Drawings to explain (and show) how the impacted storm water is pumped from the drainage channel to the impacted storm water pipeline.

b. *Please revise this section to explain how stormwater that accumulates on the FLSC top liner will be removed without damaging the top liner and revise the appropriate construction drawings accordingly to depict the stormwater removal mechanism.*

Response 6.b:
The Engineering Report (without appendices) titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included as Attachment 3. Section 3 was modified to explain how storm water that accumulates on the FLSC top liner will be removed without damaging the top liner, and is included in the revised Engineering Report.

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. A note was added to Sheet 5 of the revised Permit Drawings to explain how storm water that accumulates on the FLSC top liner will be removed without damaging the top liner.

7. *§4.5: Since a leak in the primary and secondary sump indicates a leak in the FLSC container may be occurring, please provide an explanation and justification for pumping the leaked leachate back into the FLSC.*

Response 7:
The Engineering Report (without appendices) titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included as Attachment 3. Section 4.5 was modified to explain and justify the rationale for pumping the leaked leachate back into the FLSC, and is included in the revised Engineering Report.

APPENDIX A – FDEP FORM 62-701.900(6)

8. ***Rule 62-701.320 (7) (b), F.A.C.*** *Application Form #62-701.900(6): Please address the following comments regarding the permit application form and provide a revised application form with the following information, where applicable:*

   a. *§B.1. This application is for construction of the FLSC only. Please revise the narrative description in this section accordingly.*

Response 8.a:
Section B.1, page 6 of the permit application form, FDEP Form #62-701.900(1) dated November 2006, was revised to indicate that the submitted permit application is for construction of the FLSCs only. The revised page is included in Attachment 7.

b. §D.1. The FLSC is a solid waste management unit and therefore the siting prohibitions are applicable to the FLSC. Please revise this section accordingly and address and confirm that the siting prohibitions in Rule 62-701.300 (2), F.A.C. will not be violated by the proposed construction or operation of the FLSC.

Response 8.b:
Section D.1, page 11 of the permit application form, FDEP Form #62-701.900(1) dated November 2006, was revised to address and confirm that the siting prohibitions in Rule 62-701.300(2) will not be violated by the proposed construction or operation of the FLSC facility. The revised page is included in Attachment 7.

The Engineering Report (without appendices) titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included as Attachment 3. A new section, Section 3.5, was added to address and confirm that the siting prohibitions in Rule 62-701.300(2) will not be violated by the proposed construction or operation of the FLSC facility, and is included in the revised Engineering Report.

Appendix B – CONSTRUCTION DRAWINGS (RULE 62-701.(9), F.A.C.)

Please provide the following additional information and revisions to the facility Construction Drawings. The drawings will be reviewed in their entirety after the responses to this request for information. Some comments related to the drawings are difficult to explain, and should be discussed at the meeting requested at the end of this letter.

9. ***Sheet 2 of 13 – Site Development Plan***

   a. *The reference to Detail 12 being located on Sheet 12 is incorrect. Please correct this detail reference. Detail 11 is not located on Sheet 12 and does not appear to be provided in the construction drawings. Please provide Detail 11.*

   Response 9.a:
   The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The reference for Detail 12 was corrected to Sheet 13 of the revised Permit Drawings. The label for Detail 11 was added to Sheet 12 of the revised Permit Drawings. It is noted that the typical surface water drainage channel cross section is applicable to both Details 9 and 11 on Sheet 2 of the revised Permit Drawings.

10. ***Sheet 3 of 13 – Base Grading Plan***

   a. *Please provide a table of the elevations at the control points shown on this plan sheet.*

   Response 10.a:
   The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. A table with coordinates (Northings and Eastings) and elevations is included on Sheet 3 of the revised Permit Drawings.

   b. *Please explain the design rationale for having the crest elevation of the division berm 1 foot below the perimeter and separator berm crest elevation.*

   Response 10.b:
   In the unlikely event that the capacity of an individual FLSC cell is exceeded, the crest elevation of the division berms between cells that contain either leachate or impacted storm water is maintained one foot lower than the perimeter and separator berms (see Sheet 5 of the revised Permit Drawings included in Attachment 2). Under these extreme

circumstances, excess leachate (or impacted strom water) can overflow into the adjacent cell and still be contained.

11. ***Sheet 4 of 13 – Final Grading Plan***

a. Please provide section details of the liner system at the interface between the division berm and the perimeter and separator berm.

Response 11.a:

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2.

On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting, it was agreed that section details of the liner system at the interface between the division berm and the perimeter and separator berm were not required. Sections A and B as shown on Sheets 4 and 5 of the revised Permit Drawings provide adequate details.

12. ***Sheet 6 of 13 – Liner System Details I***

a. Detail 1:

1) Please provide section details of the liner system configuration at the elevation of the liner system/gas vent, both with and without a gas vent.

Response 12.a.1:

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2.

On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. As agreed to in the meeting, a detail showing the liner system

configuration without the gas vent has been provided on Sheet 6 of the revised Permit Drawings.

2) *The GCL appears to be located outside of the 2' x2' anchor trench. Please verify this and explain this configuration.*

Response 12.a.2:
The correct anchor trench configuration with respect to the GCL is presented in Detail 1 on Sheet 6 of the revised Permit Drawings included in Attachment 2. As presented in Appendix E of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, the anchor trench geosynthetic pullout calculations for the liner system do not assume that the GCL extends horizontally along the bottom of the anchor trench.

3) *It appears that the bottom FLSC liner will remain exposed between the top/bottom FLSC extrusion weld and the anchor trench. Please verify this and explain this configuration.*

Response 12.a.3:
The bottom geomembrane layer of the FLSCs will remain exposed from the upper-most extrusion weld to the anchor trench as shown in Detail 1 on Sheet 6 of the revised Permit Drawings included in Attachment 2. This configuration does not negatively impact the integrity of the FLSC liner system since it will be completely covered by the FLSC geomembrane layers.

4) *Please explain the significance of the 3' area identified at the toe of slope of the FLSC.*

Response 12.a.4:
The double-sided drainage geocomposite layer extends 3 ft up the side slope of the FLSC perimeter berms as shown in Detail 1 on Sheet 6 of the revised Permit Drawings included in Attachment 2. The objective of the proposed configuration

is to provide maximum drainage layer coverage beneath the primary liner (i.e., geomembrane layer).

b. Details 2 & 3:

1) It appears that the bottom FLSC geomembrane liner and the primary geomembrane liner will be installed directly on top of the sump gravel. Please verify this and explain how damage to the geomembrane will be prevented and/or revise applicable details accordingly.

Response 12.b.1:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. As agreed to in the meeting, an 8 oz/yd^2 geotextile protective layer has been incorporated in Details 2 and 3 on Sheet 6 of the revised Permit Drawings included in Attachment 2. The geotextile layer will be placed on top of the sump gravel beneath the geomembrane liner.

2) It appears that the geocomposite drainage layers are not attached or anchored at their end point. Please verify this and explain how the geocomposite drainage layers will remain in place.

Response 12.b.2:
As presented in Details 2 and 3 on Sheet 6 of the revised Permit Drawings included in Attachment 2, the double-sided drainage geocomposite layers on the side slopes are not physically anchored at their end point. Downward sliding of the geocomposite layer is not anticipated when the FLSC is filled with liquid because the hydrostatic pressure along the slope will be applied perpendicular to the slope, thereby providing a confining stress to hold the geocomposite layer in-place. Conversely, when the FLSC is empty, downward slippage of the geocomposite layer is not expected since the downward tangential force along the geocomposite layer is negligible due to a no-load condition.

13. ***Sheet 7 of 13 – Liner System Details II***

a. Details 4 & 5:

1) It appears that the perforated HDPE pipes are not wrapped within the gravel sump area. Please verify this and explain how clogging of the pipes by the gravel sump material will be prevented.

Response 13.a.1:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The HDPE pipes within the sump area, as shown on Details 4 and 5 of Sheet 7 of the revised Permit Drawings, will be placed within the gravel layer.

On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. As agreed to in the meeting, pipe perforation sizing calculations were performed to evaluate the maximum allowable perforation diameter in the leachate sump pipes that will prevent gravel from passing through. The detailed perforation calculations are provided in Attachment 8. Based on No. 4 stone (proposed sump drainage gravel), the maximum allowable perforation diameter in the leachate sump pipes is 0.84 inches, which is greater than the proposed ⅝-inch diameter holes presented in Details 13 and 14 on Sheet 13 of the revised Permit Drawings included in Attachment 2. Therefore the proposed ⅝-inch diameter holes in the leachate sump pipes are adequate.

2) Please provide a detail of the perforated end caps.

Response 13.a.2:

A detail for the perforated end caps shown in Details 4 and 5 on Sheet 7 of the revised Permit Drawings included in Attachment 2 has been included as Detail 14 on Sheet 13 of the revised Permit Drawings.

b. Detail 7:

1) From Details 4 & 5, depending on where on the side slopes Section 7 is located, either perforated primary and secondary outflow and instrumentation pipes are installed on the top of the geocomposite drainage layer or perforated outflow pipes and solid instrumentation pipes are installed directly on top of geomembrane. Please verify where on the side slope Section 7 is located and revise this figure accordingly.

Response 13.b.1:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting the discussion below was provided in response to the Department's comment above.

The location of Section 7 is correctly presented on Sheet 5 of the revised Permit Drawings included in Attachment 2. The location of Section 7 has also been added to Sheet 7 of the revised Permit Drawings. Detail 7, also on Sheet 7 of the revised Permit Drawings, reflects a view from the top of the separator berm looking down along the side slope. As such, the primary and secondary leachate pipes within the liner system, as well as the FLSC pipes are shown in Detail 7.

*14. **Sheet 8 of 13 – FLSC Piping Layout***

a. As depicted on this plan sheet, it does appear that impacted stormwater could be pumped into and out of the leachate FLSCs, as is indicated in Section 3.2 of the Engineering Report. Please explain.

Response 14.a:
In the event that additional impacted storm water storage capacity is required, impacted storm water within FLSCs 1A and 1B can be pumped to FLSCs 2A and 2B designated for leachate. This would only occur if the impacted storm water within FLSCs 1A and 1B requires treatment since only leachate can be pumped into FLSCs 2A and 2B.

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. A check valve and butterfly valve connecting the impacted storm water outflow pipe and the leachate inflow pipe was added as shown on Sheet 8 of the revised Permit Drawings. (Note: The check valve and butterfly valve previously connecting the impacted storm water inflow pipe and the leachate inflow pipe were removed since they are not required.)

b. Please revise this plan sheet to include the 4" level transducer pipe depicted on Sheet 10.

Response 14.b:
The 4 inch level transducer pipe is depicted on Sheet 8 of the revised Permit Drawings included in Attachment 2, and is identified as the FLSC Instrumentation Pipe.

15. ***Sheet 10 of 13 – Leachate Management System Mechanical Flow Schematic***

a. Please revise the technical specification to specify the 4" SDR 17 leachate transducer pipe depicted on this sheet.

Response 15.a:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The technical specification of SDR 17 for the 4 inch leachate transducer pipe as depicted on Sheet 10 of the revised Permit Drawings was corrected to SDR 11 consistent with the specification established on Sheet 7 of the revised Permit Drawings.

b. The symbol, which appears to depict the submersible pump in the primary and secondary sumps, is inconsistent with the symbol for this pump on Sheet 9. Please revise to correct this discrepancy, as applicable.

Response 15.b:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting, the symbol discrepancy depicted on Sheet 9 of the revised

Permit Drawings included in Attachment 2 was discussed. The symbol depicted on Sheet 9 of the revised Permit Drawings represents an external strainer. The symbol for the external strainer has been modified on Sheet 8 of the revised Permit Drawings to be consistent with that presented on Sheet 9. It is noted that the symbol in question does not include a submersible pump, but rather a casing to accommodate a portable submersible pump as needed.

16. ***Sheet 11 of 13 – Leachate Management System Process and Instrumentation Schematic***

a. The "LAH", "MAH", and "FAL" identifications on this plan sheet are not included in the instrumentation identification table on Sheet 9. Please revise to correct this discrepancy, as applicable.

Response 16.a:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The LAH, MAH and FAL identifications depicted on Sheet 11 of the revised Permit Drawings have been incorporated on the Instrument Identification Table on Sheet 9. Note (a) on Sheet 9 provides further information on the identification convention regarding switches and alarm devices.

17. ***Sheet 13 of 13 – Miscellaneous Details***

a. Please provide a detail showing how impacted leachate is transferred from the perimeter drainage channel to the impacted stormwater pipeline.

Response 17.a:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. A description of how the impacted storm water is transferred from the drainage channel to the impacted storm water pipeline was provided in Response 6.a. above. A note was added to Sheet 2 and a detail was added to Sheet 10 of the revised Permit Drawings to explain (and show) how the impacted storm water is pumped from the drainage channel to the impacted storm water pipeline.

Appendix D – Conveyance Pipe Stability Calculation Package, Rules 62-701.320(7)(e) and 62-701.400(4)(a), F.A.C.

The calculations provided in Appendix D including several references to supporting documents that were the source of assumptions, referenced values, and equations utilized for the calculations. However copies of the relevant sections of many of those documents were not provided and therefore the Department was unable to verify the validity of the assumptions, values, and equations utilized in those calculations. Please provide copies of the relevant sections of all references utilized in each of the calculations. The calculations in Appendix D will be reviewed in their entirety upon receipt of the supporting references and the information requested below.

18. The pipe stability calculations do not appear to account for potential loss of strength due to pipe perforations. Please explain and provide revised calculations that account for pipe perforation, as applicable.

Response 18:
As presented in Appendix D of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, the pipe stability calculations evaluated the most critical pipe and corresponding loading. The critical pipe and loading were identified as the solid 6-inch diameter SDR 11 HDPE conveyance pipe and a 55 psi traffic load, respectively.

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The perforated pipes associated with the FLSC facility include 4, 6 and 18-inch diameter SDR 11 HDPE pipes as presented on Sheets 6 and 7 of the revised Permit Drawings. Additional pipe stability calculations have been performed for these perforated pipes and are included in Attachment 9. The calculated results for wall crushing, wall buckling, ring deflection, and bending strain indicate that the perforated 4, 6 and 18-inch diameter SDR 11 HDPE pipes provide adequate structural stability when subjected to an expected loading within the FLSCs of approximately 4.5 psi.

Copies of the relevant sections of all the references utilized in each of the calculations are also provided at the end of Attachment 9.

*19. **Pipe Data**: Based on the inner diameter (5.349 in.) and wall thickness (0.602 in.) provided in Attachment 1 for a 6" SDR-11 pipe, the outer diameter reported in this and other sections of Appendix D (6.625 in.) appears to be in error. Please revise this section and the pipe stability calculations provided accordingly, where applicable.*

Response 19:
The correct SDR 11 pipe data is presented in Appendix D of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006. The minimum wall thickness is 0.602 inches, the average inner pipe diameter is 5.349 inches, and the nominal outer pipe diameter is 6.625 inches per the manufacturer's data presented in Attachment 1 of Appendix D. The actual wall thickness may vary slightly but a minimum of 0.602 inches will be provided by the manufacturer. The calculations provided in Appendix D of the November 2006 Engineering Report are consistent with manufacturer specifications. As a result, revised pipe stability calculations were not performed.

*20. **Wall Crushing:** Based on the compressive strength value (1600 psi) provided in Attachment 1 for HDPE pipe, the compressive strength value reported in this and other sections of Appendix D (1500 psi) appears to be in error. Please revise this section and the pipe stability calculations provided accordingly, where applicable.*

Response 20:
The approximate compressive strength of the HDPE pipe is 1,600 psi, as indicated by the data presented in Attachment 1 of Appendix D of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006. A lower compressive strength value of 1,500 psi was utilized and resulted in a calculated factor of safety of 5.5 against wall crushing. For a compressive strength of 1,600 psi, the re-calculated factor of safety is 5.8. As such, the assumed 1,500 psi compressive strength of the HDPE pipe is conservative, and revised calculations are not required.

21. ***Wall Buckling:*** *The assumed values for Young's modulus and Poisson's ratio appear to be interpolated from the Selig reference provided in Attachment 2, assuming 90% standard Proctor compaction. However Specification 2200-3.07E. indicates that the general fill and subgrade will be compacted to 95% standard Proctor. Please explain this apparent discrepancy and revise this section and the pipe stability calculations provided accordingly, where applicable. Please explain the assumed "average value" for the "Empirical factor."*

Response 21:
The general fill and subgrade will be compacted to 95% standard Proctor as specified in Section 02200 of the Technical Specifications presented in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006. Young's modulus and Poisson's ratio were interpolated from the Selig [1990] reference assuming a lower standard Proctor compaction effort of 90%, and resulted in a calculated factor of safety of 7.4 against wall buckling. For a Young's modulus and Poisson's ratio corresponding to a 95% standard Proctor compaction effort, the re-calculated factor of safety is 10.7. As such, the assumed Young's modulus and Poisson's ratio corresponding to a 90% standard Proctor compaction effort are conservative, and revised calculations were not performed.

As indicated by Selig [1990], the empirical factor (k) can vary from 0.7 to 2.3 with k equal to 1.5 as a representative value.

22. ***Summary:*** *The construction drawings appear to indicate that the 4" SDR-11 HDPE pipes will be constructed adjacent to the 6" pipes within the FLSC. Therefore it does not appear that the 4" pipes "will be subjected to a substantially smaller loading stress...." Please provide pipe stability calculation for the 4" pipe.*

Response 22:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting, the stress applied to the 4 and 6-inch diameter SDR 11 HDPE pipes within the FLCSs, as presented in Appendix D of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, was discussed. The calculations provided in Appendix D of the November 2006 Engineering Report evaluated the 6-inch

diameter SDR 11 HDPE pipe subjected to a traffic load of 55 psi. The 6-inch diameter pipe is the only pipe subjected to the traffic loading (55 psi) and was thereby identified as a more critical pipe.

The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The side-by-side 4 and 6-inch diameter SDR 11 HDPE pipes within the FLSC sumps (see Sheets 6 and 7 of the revised Permit Drawings) will be subjected to a loading of about 4 psi, as presented in Appendix D of the November 2006 Engineering Report. Stability calculations for the perforated 4 and 6-inch diameter SDR 11 HDPE leachate sump pipes have been provided in Attachment 9 as discussed in Response 18 above.

Appendix E – Anchor Trench Design Calculation Package

23. ***HDPE Geomembrane Material Properties:*** *The tensile strength utilized for the anchor trench calculations (90 lb/in) is inconsistent with that specified in Specification 2770-Table 2770-1 (72 lb/in). Please revise the anchor trench calculations or the referenced specification to address this discrepancy.*

Response 23:
Section 02770 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. As discussed in Appendix E of the November 2006 Engineering Report, the tensile strength of the geomembrane should be in accordance with GRI Test Method GM-13. The tensile strengths of the geomembrane at break and yield in Table 02770-1 of Section 02770 were corrected to 90 lb/in and 126 lb/in, respectively. In addition, the ASTM standard for tensile properties of the geomembrane was incorrectly referenced in Section 02770, Part 1.03.A. The reference to the correct ASTM standard, ASTM D 6693, has been provided in Part 1.03.A of Section 02770.

The geomembrane seam properties are also presented in Table 02770-2 of Section 02770 of the Technical Specifications included in Appendix H of the November 2006 Engineering Report. Since the geomembrane seam properties are a function of the tensile strengths identified above,

the seam properties were revised accordingly in accordance with GRI Test Method GM-19. Part 1.03.B of Section 02770 has been updated to include the reference for GRI Test Method GM-19. The revised Table 02770-2 is included in Attachment 10.

(Note: Table 02770-1 and 2 are referenced in Attachment B of the CQA Plan included as Appendix I of the engineering report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006.)

*24. **Attachment 2 – Typical Interface Friction Values:** Please provide copies of the references sources for the assumed interface friction values provided in this Attachment.*

Response 24:
The references cited in Attachment 2 of Appendix E of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, have been provided in Attachment 11.

Appendix F – Perimeter Berm Stability Calculation Package, Rule 62-701.410, F.A.C.

*25. **FLSC Configuration:** This section indicates that the FLSC perimeter berm has an 8-foot wide crest while the Representative Cross Section shown in Attachment 1 and the perimeter berm stability calculations in Appendix F assume a 7-foot wide crest. The construction drawings show 8-foot wide crest on the perimeter berm and division berm and a 12-foot wide crest on the separator berm. Please revise this section, the calculations in Appendix F, and/or the construction drawings, as applicable based on the perimeter crest widths proposed for the FLSC.*

Response 25:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. The FLSC perimeter, division and separator berm widths are correct as shown in the revised Permit Drawings, i.e., 8-ft, 8-ft and 12-ft wide respectively.

On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting the slope stability analysis of the perimeter berm as presented in Appendix F of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, was discussed. The slope stability analysis of the perimeter berm assumed a top crest width of 7 ft as noted in the above comment. However, upon further inspection of the stability analysis graphical output as presented in Attachment 2 of Appendix F of the November 2006 Engineering Report, the critical failure surface starts along the central portion of the assumed 7-ft wide top crest area of the perimeter berm. This indicates that perimeter berm stability for the case analyzed is not sensitive to the width of the top crest of the berm. In other words, if the width of the perimeter berm is increased to 8 ft or 12 ft, the location of the failure surface will not change and the minimum factor of safety will remain the same. Moreover, the reported minimum factor of safety of 2.84 is indicative of a very stable configuration and exceeds the typical regulatory requirement of 1.5.

Since Comments 27.a and b (see below) require a revised perimeter berm slope stability analysis and based on the discussion above, only the 8-ft wide top crest berm configuration has been analyzed. The results of the perimeter berm slope stability analyses are included in Attachment 12, and indicate a minimum calculated factor of safety of 2.84 for the revised configuration.

*26. **Method of Analysis:** Please provide a copy of the "sliding block methodology" reference utilized for the sliding block analysis.*

Response 26:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting the sliding block methodology as presented in Appendix F of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, was discussed. It was noted that sliding block analyses are commonly used to evaluate the stability of gravity dams. A reference that outlines sliding analyses for dam design has been provided in Attachment 13.

27. ***Attachment 2 – Rotational Foundation Stability Analysis:***

a. The slide analysis information indicates that the unit weight for the berm material was assumed to be 115 lb/ft³. Please explain this discrepancy and revise the rotational stability analysis and/or Attachment 1, as applicable.

Response 27.a:
The correct unit weight of the berm material is 120 pcf as presented in the hand-drawn cross section in Attachments 1 and 2 of Appendix F of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006. The perimeter berm stability analysis presented in Attachment 2 of Appendix F of the November 2006 Engineering Report was revised accordingly. The hand-drawn cross section in Attachment 1 of Appendix F of the November 2006 Engineering Report was also revised to reflect the correct top crest width of the perimeter berm as discussed in Comment 25 of this RAI. The results of the revised perimeter berm slope stability analysis have been provided in Attachment 12. Note that the revised analysis also addresses Comments 25 and 27.c of this RAI.

b. Please explain the "Hu" value and the rationale for the value assumed.

Response 27.b:
The Hu coefficient, as defined in SLIDE (slope stability software), is simply a factor between 0 and 1, by which the vertical distance from a point in the soil (e.g. the center of a slice base) to a Water Surface (either a Water Table or Piezometric Line) is multiplied to obtain the pressure head. Hu equal to 1 would indicate hydrostatic conditions and can be used where the Water Surface is horizontal. Where the Water Surface is inclined, setting Hu equal to 1 will provide a conservative (low) estimate of the safety factor, since in general this will overestimate the true pore pressure. In most cases, the user will simply set Hu equal to 1, because this represents the worst case scenario (maximum pore pressure). Additional information on the Hu coefficient in SLIDE is provided in Attachment 14.

c. The assumed water table elevation of 17.5 NGVD in the Rotational stability analysis appears to be inconsistent with the 16.5 NGVD water table elevation reported throughout

the remainder of this application. Please revise the rotational stability analysis accordingly.

Response 27.c:
The correct water table elevation is 16.5 NGVD as presented in the hand-drawn cross section in Attachment 2 of Appendix F of the engineering report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006. The perimeter berm stability analysis presented in Attachment 2 of Appendix F of the November 2006 Engineering Report was revised accordingly. The results of the revised perimeter berm slope stability analysis have been provided in Attachment 12. Note that the revised analysis also addresses Comments 25 and 27.a of this RAI.

Appendix G – Liner System Leakage and Lateral Drainage Capacity Calculation Package,

Rule 62-701.400, F.A.C.

The calculations provided in Appendix G including several references to supporting documents that were the source of assumptions, referenced values, and equations utilized for the calculations. However copies of the relevant sections of many of those documents were not provided and therefore the Department was unable to verify the validity of the assumptions, values, and equations utilized in those calculations. Please provide copies of the relevant sections of all references utilized in each of the calculations. The calculations in Appendix G will be reviewed in their entirety upon receipt of the supporting references and the information requested below.

28. Sheet 7 of the construction drawings depicts the bottom FLSC liner installed directly on top of the primary leak detection outflow and instrumentation pipes on the FLSC side slopes. Please explain how this liner system configuration is considered in the liner leakage calculations.

Response 28:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. Sheet 7 of the revised Permit Drawings correctly depicts the bottom

of the FLSC liner on top of the primary leak detection outflow and instrumentation pipes along the FLSC side slopes.

On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting it was agreed that the liner system configuration as presented on Sheet 7 of the revised Permit Drawings is adequate and revised calculations were not required.

Attachment 15 provides copies of the relevant sections of all the references utilized in each of the leakage and lateral drainage calculations.

29. Please provide leachate collection system filter fabric (geotextile) design calculations.

Response 29:
The Permit Drawings titled "Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida," dated November 2006, have been revised and are included as Attachment 2. As identified on Sheets 6 and 7, a geotextile will be placed on top of the drainage gravel beneath the geomembrane layer, and a double-sided drainage geocomposite will be placed along the base of the FLSC sumps below the drainage gravel.

An 8 oz/yd^2 geotextile has been specified for this proposed construction project including the upper and lower geotextiles of the double-sided drainage geocomposite as presented in the Technical Specifications included as Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006.

Since the geotextile will not be in contact with any soils other than the gravel within the sump, filter design calculations (for the geotextile) are not required.

Appendix H – Technical Specifications, Rules 62-701.400(3), (7) and (8)

Please revise the Technical Specifications and/or other referenced application documents, as appropriate, to address the following comments and/or inconsistencies.

30. Please provide the Technical Specifications for "Concrete" referenced in Section 12 of the Construction Quality Assurance (CQA) Plan.

Response 30:
The Technical Specifications and CQA Plan were included as Appendices H and I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006. Section 03300 provides the technical specifications for Concrete as referenced in Section 12 of the CQA Plan and has been provided in Attachment 16.

31. ***Section 02200 - Earthwork***

a. §1.04.A. The referenced Sections 2230 and 2240 in this section were not provided. Please revise this section or provide these specification sections, as applicable.

Response 31.a:
Section 02200 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 17. Part 1.04.A of Section 02200 was modified to remove references to Sections 2230 and 2240 which are not a part of the proposed FLSC facility construction project.

b. §1.050B. Please indicate who will provide equipment and labor to assist the CQA Consultant.

Response 31.b:
Section 02200 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 17. Part 1.05.B of Section 02200 was modified to indicate that the Contractor will provide equipment and labor to assist the CQA Consultant.

c. §2.01.A. & 3.06.B. Please identify the borrow source for fill material for this project.

Response 31.c:
Section 02200 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 17. Parts 2.01.A and 3.06.B of Section 02200 have been revised to indicate that the fill materials are expected to be obtained from existing on-site borrow pits and/or stockpiles at this time.

d. §3.05. Please note that dewatering may require an Industrial Waste Permit from the Department. Please specify who will be responsible for obtaining any necessary dewatering permits from the Department.

Response 31.d:
The FLSC facility will be built-up relative to existing ground, and any excavations below ground will be limited. The impacted storm water conveyance pipeline will be installed in a shallow trench along the side slope of the existing perimeter access road. As such, de-watering activities, if any, for the project will be very limited and localized and an Industrial Water Permit will not be required.

e. §3.07.A Please specify that stones or ruts shall be no larger than 1", consistent with Section 7.4 of the CQA Plan.

Response 31.e:
Section 02200 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 17. Parts 3.07.C and E of Section 02200 have been modified to be consistent with Section 7.4 of the CQA Plan, which states that the prepared subgrade shall not contain loose stones or ruts greater than 1 inch in depth.

32. ***Section 02240 - Geocomposite***

a. §1.04.A. The referenced Tale 02740-1 is missing from this section. Please provide.

Response 32.a:
Section 02740 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 18. Table 02740-1 was inadvertently omitted from the submittal, the corresponding page has been provided.

b. §2.05. Please specify the storage limits for the geocomposite consistent with Section 9.2 of the CQA Manual.

Response 32.b:
Section 02740 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 18. Part 2.05.D has been added to Section 02740 to address storage limit requirements for the geocomposite consistent with Section 9.2 of the CQA Plan.

c. §3.02.B.1. The bottom layer overlap specified in this section is inconsistent with that specified in Section 9.5 of the CQA Plan.

Response 32.c:
Section 02740 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 18. Part 3.02.B.1 of Section 02740 has been modified to clarify geotextile bottom overlap requirements consistent with Section 9.5 of the CQA Plan.

Note that the CQA Plan included in Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised and is included (without attachments) in Attachment 20. Section 9.5 of the CQA Plan was modified to clarify geonet overlap requirements consistent with Part 3.02.C of Section 02740.

33. ***Section 02270 - Geomembrane***

a. §3.03.C.5.e. Allowance for wrinkles of up to 4 inches of does not appear to provide for "intimate contact" as specified in this section. Please explain and revise this section accordingly.

Response 33.a:
In general, during the hottest portion of the day, some wrinkling may occur due to thermal expansion of the HDPE material. During cooler portions of the day (e.g., mornings and late afternoon), the HDPE material cools and any wrinkles present during the hotter portion of the day disappear to restore the intimate contact with the subgrade. The intent of Part 3.03.C.5.e of Section 02270 is to minimize wrinkles, and it is the Engineer's experience that a 4 inch tolerance on wrinkles during the hottest portion of the day is acceptable.

b. §3.03.C.5.e. Geomembrane installation shall not occur during non-daylight hours and shall not be approved by the Engineer. Please revise this section accordingly.

Response 33.b:
Section 02770 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. Part 3.03.C.7 has been incorporated to Section 02770 indicating that geomembrane installation shall not occur during non-daylight hours.

c. §3.04.D.1. Please specify the geomembrane panel overlap consistent with Section 6.7.5 of the CQA Plan.

Response 33.c:
Section 02770 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. Part 3.04.D.1 of Section 02770 has been modified to specify a minimum finished panel overlap of 4 inches consistent with Section 6.7.5 of the CQA Plan.

d. §3.04.E.3. Please specify that seam will be aligned with no "fishmouths."

Response 33.d:
Section 02770 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. Part 3.04.E.3 of Section 02770 has been modified to specify that seams will be aligned with no "fishmouths."

e. §3.04.J.2. The sampling and testing methods specified in this section are inconsistent with those specified in Section 6.7.9.3 of the CQA Plan.

Response 33.e:
Section 02770 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. Part 3.04.J.2 of Section 02770 was modified to be consistent with the sampling and testing methods specified in Section 6.7.9.3 of the CQA Plan.

f. Table 02770-1 Please specify the Oxidative Induction Time property for the geomembrane. The tensile strength (at break) property provided appears incorrect. Please verify and revise, as appropriate.

Response 33.f:
Section 02770 of the Technical Specifications included in Appendix H of the engineering report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. Table 02770-1 of Section 02770 was modified to specify the Oxidative Induction Time property for the geomembrane. Part 1.03.B of Section 02770 has been updated to include to the appropriate ASTM test methods, i.e., ASTM D 3895 and ASTM D 5885. The tensile strength properties of the geomembrane were addressed in Response 23 of this RAI.

(Note: Oxidative Induction Time property for the geomembrane is referenced in Table 6-1 of the CQA Plan included as Appendix I of the Engineering Report titled "Application

for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006.)

g. Table 02770-2 Please specify seam shear strength properties that are at least 90% of the minimum yield strength for the geomembrane, in accordance with Rule 62-701.400(2)(d), F.A.C.

Response 33.g:
Seam shear strengths were addressed in Response 23 of this RAI, and Table 02770-2 has been included in Attachment 10. The specified seam shear strength (120 lb/in) is approximately 95% of the minimum yield strength (126 lb/in) for the geomembrane material as presented in Table 02770-1 which is also included in Attachment 10.

34. ***Section 16651 – Control Panel Fabrication***

a. §2.02. The reference to "two" FLSC in this section appears to be inconsistent with the four proposed in this application.

Response 34.a:
Section 16651 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 19. Part 2.02 of Section 16551 was modified clarify the reference to "two" FLSCs. There will be two Control Panels: (i) one for the two FLSC cells that will store leachate, and (ii) one for other the two FLSC cells that will store impacted storm water. A total of four FLSC cells will be constructed.

Appendix I – Construction Quality Assurance Plan, Rules 62-701.400(3), (7) & (8)

Please revise the CQA Plan and/or other referenced application documents, as appropriate, to address the following comments and/or inconsistencies.

35. ***Section 3 – Project Organization and Personnel***

a. §3.9. Please specify that the geosynthetics installer obtains samples as required by the CQA Plan, under the direction of CQA personnel.

Response 35.a:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 3.9 of the CQA Plan has been modified to specify that the geosynthetics installer will obtain samples as required by the CQA Plan and Technical Specifications under the direction of CQA personnel.

36. ***Section 4 - Documentation***

a. §4.6. Please specify that copies of photographs referenced in Section 4.3 will be part of the Certification Report.

Response 36.a:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Sections 4.3 and 4.6 of the CQA Plan have been modified to specify that copies of the referenced documentation photographs will be part of the Certification Report.

37. ***Section 6 - Geomembrane***

a. §6.7.2. No "alternate process" for seaming has been specified in the Technical Specifications. Please revise this section to eliminate this option or provide technical specifications for "alternate processes."

Response 37.a:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 6.7.2 of the CQA Plan has been modified to eliminate alternate seaming processes.

b. §6.7.4. Please specify that seam will be aligned with no "fishmouths."

Response 37.b:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 6.7.3 of the CQA Plan has been modified to specify that seams will be aligned with no "fishmouths."

c. §6.7.7. Geomembrane seaming shall not occur during non-daylight hours. Please revise this section accordingly.

Response 37.c:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 6.7.7 of the CQA Plan has been modified to specify that geomembrane seaming shall not occur during non-daylight hours.

d. §6.7.8. Please provide technical specifications for spark testing.

Response 37.d:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 6.7.8 of the CQA Plan has been modified to eliminate spark testing.

e. §6.7.9.5. Please specify that all five destructive test specimens shall pass laboratory CQA testing consistent with Section 02770-3.04.J.3. of the Technical Specifications.

Response 37.e:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 6.7.9.5 of the CQA Plan has been modified to clarify testing

of the destructive specimens by stating that: "A passing test shall meet or exceed the minimum required values in at least four out of five specimens, and the fifth specimen shall meet or exceed 80% of the minimum required values. In the event that the CQA destructive testing sample fails, the archived sample may be tested…" in accordance with GRI Test Method GM-19.

f. Table 6-1 Please revise this table to indicate that a minimum of one conformance test per 100,000 square feet of material shall be conducted for geomembrane/ geocomposite interface shear strength.

Response 37.f:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting, it was agreed that only one interface friction conformance test will be performed to confirm interface friction values utilized in the anchor trench pullout calculations.

The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. One geomembrane-geomembrane interface friction test has been included in Table 6-1 of the CQA Plan. The geomembrane-geomembrane interface was selected as the critical interface of the FLSC liner system in the anchor trench pullout calculations.

Section 02770 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 10. Part 1.03 of Section 02770 has been updated accordingly and Part 2.03.C has been added to Section 02770 to address the CQA conformance testing details associated with the geomembrane-geomembrane interface friction testing.

38. ***Section 8 - Geotextiles***

a. §8.2. This section is inconsistent with Technical Specification 02720-2.05.C. that specifies that geotextile rolls shall not be stored for greater than 6 months.

Response 38.a:
Section 02720 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 21. Part 2.05.C of Section 02720 has been modified to be consistent with Section 8.2 of the CQA Plan.

b. §8.3. This section is inconsistent with the Construction Drawings, which appears to indicate that geotextiles will not be anchored in the anchor trench.

Response 38.b:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Sections 8.3.5 and 8.6 of the CQA Plan have been modified to be consistent with the revised Permit Drawings (see Attachment 2) which indicate that geotextiles will not be anchored in the anchor trench.

c. §8.6. Please revise Technical Specification 02720 to provide specifications for equipment ground pressure of geotextile overlying geomembrane as indicated in this section, as appropriate.

Response 38.c:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and replacement pages are included in Attachment 20. For construction of the FLSC facility, geotextiles will not be placed directly on geomembranes; Section 8.6 of the CQA Plan has been modified accordingly.

Section 02720 of the Technical Specifications included in Appendix H of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 21. Part 3.04.E has been added to Section 02720 to provide specifications for acceptable ground pressures applied on the geotextile by construction equipment.

d. Table 8-1 Please revise this table to correct the reference (5) typographic error.

Response 38.d:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Table 8.1 of the CQA Plan has been revised to correct the reference (5) typographic error. The correct reference number is (4).

39. ***Section 9 - Geocomposites***

a. §9.4. This section is inconsistent with the Construction Drawings, which appears to indicate that the geocomposite will not be anchored in the anchor trench.

Response 39.a:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 9.4 of the CQA Plan has been modified to be consistent with the revised Permit Drawings (see Attachment 2) which indicate that the geocomposite will not be anchored in the anchor trench.

b. §9.5. This section is inconsistent with Technical Specification 02740-3.02.C., which specifies that adjacent geonet edges will overlap a minimum of 4 inches and Technical Specification 02740-3.02.B.2., which specifies that horizontal seams can be 1/3 up a greater than 10H:1V side slope.

Response 39.b:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Section 9.5 of the CQA Plan has been modified to be consistent with Parts 3.02.C and 3.02.B.2 of Section 02740.

c. Table 9-1 Please revise this table to indicate that a minimum of one conformance test per 100,000 square feet of material shall be conducted for geomembrane/ geocomposite interface shear strength.

Response 39.c:
On 22 February 2007, Geosyntec met with the Department to discuss several RAI No. 1 comments. During the meeting, it was agreed that only one interface friction conformance test will be performed to confirm interface friction values utilized in the anchor trench pullout calculations. The geomembrane-geomembrane interface was identified as the critical interface in the pullout calculations. As such geomembrane-geocomposite interface friction testing will not be performed.

40. ***Section 10 – Pipes and Fitting***

a. §10.1. Technical Specification 02715 does not appear to provide specification for FLSC gas system installation, as described in this section. Please explain and revise, as appropriate.

Response 40.a:
The CQA Plan included as Appendix I of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, has been revised accordingly and is included in Attachment 20. Reference to the FLSC gas management system in Section 10.1 of the CQA Plan has been removed since it will not be required for the FLSC facility. The proposed gas (air) vents for the FLSCs consist of a vent hole and a geomembrane flap as presented on Sheet 6 of the revised Permit Drawings (see Attachment 2).

DEP FORM NO. 62-701.900(1), SOLID WASTE MANAGEMENT FACILITY PERMIT FORM

SECTION B – DISPOSAL FACILITY GENERAL INFORMATION

*1. **B.13.:** The "Yes" response on this item of the application form is inconsistent with the same item of the application form received September 20, 2002 that was associated with the renewal of the operations permit for the facility (permit #130542-022-SO). In the event that a Declaration to the Public has been filed with the Sarasota County Clerk's office that meets the requirements of Rule 62-701.610(5), F.A.C., please submit a certified copy of the declaration. In the event that a Declaration to the Public has not been filed for the facility, please submit a revised application form for this item that indicates a "No" response.*

Response 1 (Section B.13):
A Declaration to the Public has not been filed with the Sarasota County Clerk's office. FDEP Form 62-701.900(1) has been revised accordingly and replacement pages are provided in Attachment 7.

*2. **B.17.:** Please provide the basis for the indication that the water table in the vicinity of the flexible leachate storage containers occurs at an elevation of 16 feet NGVD. In the event that this ground water elevation is based on the un-numbered figure included in Appendix C entitled "Monitoring Well Construction Details MW-13" (an approximate ground elevation of 20 feet and depth to water measurement at the time of well installation), please submit additional characterization of the occurrence of ground water at well MW-13 including but not limited to: surveyed top of casing elevation to the nearest 0.01 foot NGVD; surveyed ground surface elevation to the nearest 0.01 foot NGVD; depth to ground water surface below the top of casing measured to the nearest 0.01 foot; and, total well depth below the top of casing measured to the nearest 0.01 foot. Please also submit the details of the well development activities conducted as well MW-13 to demonstrate there is a good connection with the surficial aquifer and that the resultant ground water level measurements are representative of site conditions.*

Response 2 (Section B.17):
Surveying of the monitoring well to the nearest 0.01 ft has been scheduled for early April 2007 at which time the well will be re-developed and ground water measurements taken. Upon completion of the monitoring well activities, the requested information will be forwarded to the Department.

SECTION M – WATER QUALITY AND LEACHATE MONITORING REQUIREMENTS
(Rule 62-701.510, F.A.C.)

*3. **M.1.a.:** Please note that sufficient hydrogeological information in the vicinity of the proposed leachate storage containers shall be required to support future modification of the existing monitoring plan for the facility to accommodate the operation of these leachate storage containers. As no routine ground water level measurements are conducted at the portion of the facility where these proposed leachate storage containers are located, the collection additional information is required to supplement available information. Please conduct ground water level measurements at all existing monitor wells, piezometers and staff gauges listed in permit #130542-002-SO and at new well MW-13 at least at a monthly frequency and prepare ground water surface contour maps for each set of water level data to demonstrate the direction of ground water flow. Please submit revisions to Section 3.4 of the "Engineering Report" to specify the direction of ground water flow at the proposed leachate containers determined from these supplemental water level measurements. Please also submit a revised application form for this item that refers to Section 3.4 of the "Engineering Report."*

Response 3 (Section M.1.a):
Supplemental ground water information from October 2006 to February 2007 in the vicinity of the existing ground water monitoring network associated with the existing landfill site to the north of the proposed FLSC facility has been provided by Mr. Paul A. Wingler of Sarasota County Solid Waste Operations in Attachment 22. The attached ground water contour maps indicate that the ground water flow direction is in a southwesterly direction.

Monthly ground water measurements at all existing monitoring wells, piezometers and staff gauges listed in Permit #130542-002-SO and at the monitoring of the well (identified as MW-13) adjacent to the proposed FLSC facility will be performed to demonstrate the direction of ground water flow. Monthly monitoring will commence in April 2007, and the required information will be forwarded to the Department. FDEP Form 62-701.900(1) has been revised accordingly and replacement pages are provided in Attachment 7.

Section 3.4 of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, was revised accordingly (see Attachment 3).

4. ***M.1.c.(6):***

a. The indication in Section 3.4 of the "Engineering Report" that well MW-13 is 12 feet deep and is screened from 7 to 12 feet below grade appears to be inconsistent with the un-numbered figure included in Appendix C entitled "Monitoring Well Construction Details MW-13" which indicates well MW-13 is 10 feet deep and is screened from 5 to 10 feet below grade. Please review this apparent inconsistency and submit revisions, as appropriate. Please also submit a revised application form for this item that refers to Section 3.4 of the "Engineering Report."

Response 4.a (Section M.1.c.(6)):
MW-13 is 10-ft deep and screened from 5 to 10 ft below grade. Section 3.4 of the Engineering Report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006, was revised accordingly (see Attachment 3). FDEP Form 62-701.900(1) has been revised accordingly and replacement pages are provided in Attachment 7.

b. Please note that Rule 62-701.510(3)(d)4, F.A.C., requires the following: "Wells monitoring the unconfined water table shall be screened so that the water table can be sampled at all times. The applicant shall provide technical justification for the actual screen length chosen." The suitability of well MW-13 to meet the requirements of the cited rule will depend on the construction details (requested in comments #2 and #4.a.) and the results of supplemental water level measurement conducted at the facility (requested in comment #3). It is understood that changes to the monitoring plan are not part of this construction permit application but would be associated with a future application for minor modification of permit #130542-002-SO to authorize the operation of the proposed leachate containers. ***This comment is presented for informational purposes and does not require a response.***

Response 4.b (Section M.1.c.(6)):
No response required.

CLOSURE

If you have any questions or require additional information, please do not hesitate to contact either of the undersigned at (813) 558-0990.

Sincerely,

Juan D. Quiroz

Juan D. Quiroz, Ph.D., P.E.
Project Engineer

A Gupta

Ayushman Gupta, P.E.
Senior Engineer

Attachments

Copies to: Frank Coggins, Sarasota County

ATTACHMENT 1

Department of Environmental Protection

Jeb Bush
Governor

Southwest District
13051 North Telecom Parkway
Temple Terrace, FL 33637-0926
Telephone: 813-632-7600

Colleen M. Castille
Secretary

Mr. Frank Coggins, Manager
Sarasota County Solid Waste Operations
4000 Knights Trail Road
Nokomis, Fl. 34275

December 13, 2006

RE: Sarasota CCSWDC Flexible Leachate Storage Containers Construction Pending Permit No.: 130542-005-SC/08, Sarasota County

Dear Mr. Coggins:

This is to acknowledge receipt of your application dated November 9, 2006 (received November 13, 2006) prepared by GeoSyntec Consultants, to construct a flexible leachate storage container (FLSC) system (at the solid waste management facility referred to as the Sarasota County Central Solid Waste Disposal Complex.

This letter constitutes notice that a permit will be required for your project pursuant to Chapter(s) 403, Florida Statutes.

Your application for a permit is incomplete. This is the Department's first request for information. Please provide the information listed below promptly. Evaluation of your proposed project will be delayed until all requested information has been received.

GENERAL:

1. The requested information and comments below do not repeat the information submitted by the applicant. However, every effort has been made to concisely refer to the section, page, drawing detail number, etc. where the information has been presented in the original submittal.

2. Please submit **4 copies** of all requested information. Please specify if revised information is intended to supplement, or replace, previously submitted information. Please submit all revised plans and reports as a complete package. For revisions to the narrative reports, deletions may be struckthrough (~~struckthrough~~) and additions may be shaded Shaded or similar notation method. This format will expedite the review process. Please include revision date on all revised pages.

3. Please provide a summary of all revisions to drawings, and indicate the revision on each of the applicable plan sheets. Please use a consistent numbering system for drawings. If new sheets must be added to the original plan set, please use the same numbering system with a prefix or suffix to indicate the sheet was an addition, e.g. Sheet 1A, 1B, P1-A, etc.

4. Please be advised that although some comments do not explicitly request additional information, the intent of all comments shall be to request revised calculations, narrative, technical specifications, QA documentation, plan sheets, clarification to the item, and/or other information as appropriate. **Please be reminded that all calculations must be signed and sealed by the registered professional engineer (or geologist as appropriate) who prepared them.**

The following information is needed in support of the solid waste application [Chapter 62-701, Florida Administrative Code (F.A.C.)]:

1. **Rule 62-701.320(8), F.A.C.** Please publish the attached Notice of Application and provide proof of publication to the Department.

2. **Rule 62-701.730(4)(b), F.A.C.** Responses to each of the items in John Morris' December 11, 2006 memorandum (attached) are required. You may call Mr. Morris at (813) 632-7600, extension 336, to discuss the items in his memorandum.

3. **Rule 62-701.410(2)(e), F.A.C.** Please provide foundation bearing capacity and subgrade settlements analyses for the FLSC in accordance with Rule 62-701.410(2)(e), F.A.C.

ENGINEERING REPORT (RULE 62-701.320(7)(d), F.A.C.)

4. **§1.1:** Please provide a copy of the pending ERP permit for the storm water management system modification at the facility.

5. **§3.2:**

   a. The reference to the FLSC facility being built-up relative to the existing ground as shown on Sheet 3 of the permit drawing appears to be a typographic error. Please revise to reference Sheet 5 of the drawings.

   b. Please provide the supporting calculations for the stated 300,000 gallon storage capacity of each FLSC.

6. **§3.3:**

   a. Neither the perimeter drainage channel nor weir details on the permit drawings or this section show or explain how the impacted stormwater is pumped from the drainage canal to the impacted stormwater pipeline. Please revise this section and the permit drawings to address this discrepancy.

   b. Please revise this section to explain how stormwater that accumulates on the FLSC top liner will be removed without damaging the top liner and revise the appropriate construction drawings accordingly to depict the stormwater removal mechanism.

7. **§4.5:** Since a leak in the primary and secondary sump indicates a leak in the FLSC container may be occurring, please provide an explanation and justification for pumping the leaked leachate back into the FLSC.

APPENDIX A - FDEP FORM 62-701.900(6)

8. **Rule 62-701.320(7)(b), F.A.C.** Application Form #62-701.900(6): Please address the following comments regarding the permit application form and provide a revised application form with the following information, where applicable:

   a. §B.1. This application is for construction of the FLSC only. Please revise the narrative description in this section accordingly.

   b. §D.1. The FLSC is a solid waste management unit and therefore the siting prohibitions are applicable to the FLSC. Please revise this section accordingly and address and confirm that the siting prohibitions in Rule 62-701.300(2), F.A.C. will not be violated by the proposed construction or operation of the FLSC.

Appendix B - CONSTRUCTION DRAWINGS (RULE 62-701.730(9), F.A.C.)

Please provide the following additional information and revisions to the facility Construction Drawings. The drawings will be reviewed in their entirety after the responses to this request for information. Some comments related to the drawings are difficult to explain, and should be discussed at the meeting requested at the end of this letter.

9. **Sheet 2 of 13 - Site Development Plan**

a. The reference to Detail 12 being located on Sheet 12 is incorrect. Please correct this detail reference. Detail 11 is not located on Sheet 12 and does not appear to be provided in the construction drawings. Please provide Detail 11.

10. **Sheet 3 of 13 - Base Grading Plan**

a. Please provide a table of the elevations at the control points shown on this plan sheet.

b. Please explain the design rationale for having the crest elevation of the division berm 1 foot below the perimeter and separator berm crest elevation.

11. **Sheet 4 of 13 - Final Grading Plan**

a. Please provide section details of the liner system at the interface between the division berm and the perimeter and separator berm.

12. **Sheet 6 of 13 - Liner System Details I**

a. Detail 1:

1) Please provide section details of the liner system configuration at the elevation of the liner system/gas vent, both without and without a gas vent.

2) The GCL appears to be located outside of the 2'x2' anchor trench. Please verify this and explain this configuration.

3) It appears that the bottom FLSC liner will remain exposed between the top/bottom FLSC extrusion weld and the anchor trench. Please verify this and explain this configuration.

4) Please explain the significance of the 3' area identified at the toe of slope of the FLSC.

b. Details 2 & 3:

1) It appears that the bottom FLSC geomembrane liner and the primary geomembrane liner will be installed directly on top of the sump gravel. Please verify this and explain how damage to the geomembrane will be prevented and/or revise applicable details accordingly.

2) It appears that the geocomposite drainage layers are not attached or anchored at their end point. Please verify this and explain how the geocomposite drainage layers will remain in place.

13. **Sheet 7 of 13 - Liner System Details II**

a. Details 4 & 5:

1) It appears that the perforated HDPE pipes are not wrapped within the gravel sump area. Please verify this and explain how clogging of the pipes by the gravel sump material will be prevented.

2) Please provide a detail of the perforated end caps.

b. Detail 7:

1) From Details 4 & 5, depending on where on the side slopes Section 7 is located, either perforated primary and secondary outflow and instrumentation pipes are installed on top of the geocomposite drainage layer or perforated outflow pipes and solid instrumentation pipes are installed directly on top of geomembrane. Please verify where on the side slope Section 7 is located and revise this figure accordingly.

14. **Sheet 8 of 13 - FLSC Piping Layout**

a. As depicted on this plan sheet, it does appear that impacted stormwater could be pumped into and out of the leachate FLSCs, as is indicated in Section 3.2 of the Engineering Report. Please explain.

b. Please revise this plan sheet to include the 4" level transducer pipe depicted on Sheet 10.

15. **Sheet 10 of 13 - Leachate Management System Mechanical Flow Schematic**

a. Please revise the technical specification to specify the 4" SDR 17 leachate transducer pipe depicted on this sheet.

b. The symbol, which appears to depict the submersible pump in the primary and secondary sumps, is inconsistent with the symbol for this pump on Sheet 9. Please revise to correct this discrepancy, as applicable.

16. **Sheet 11 of 13 - Leachate Management System Process and Instrumentation Schematic**

a. The "LAH", "MAH", and "FAL" identifications on this plan sheet are not included in the instrumentation identification table on Sheet 9. Please revise to correct this discrepancy, as applicable.

17. **Sheet 13 of 13 - Miscellaneous Details**

a. Please provide a detail showing how impacted leachate is transferred from the perimeter drainage channel to the impacted stormwater pipeline.

Appendix D - Conveyance Pipe Stability Calculation Package, Rules 62-701.320(7)(e) and 62-701.400(4)(a), F.A.C.

The calculations provided in Appendix D including several references to supporting documents that were the source of assumptions, referenced values, and equations utilized for the calculations. However copies of the relevant sections of many of those documents were not provided and therefore the Department was unable to verify the validity of the assumptions, values, and equations utilized in those calculations. Please provide copies of the relevant sections of all references utilized in each of the calculations. The calculations in Appendix D will be will be reviewed in their entirety upon receipt of the supporting references and the information requested below.

18. The pipe stability calculations do not appear to account for potential loss of strength due to pipe perforations, Please explain and provide revised calculations that account for pipe perforation, as applicable.

19. **Pipe Data:** Based on the inner diameter (5.349 in.) and wall thickness (0.602 in.) provided in Attachment 1 for a 6" SDR-11 pipe, the outer diameter reported in this and other sections of Appendix D (6.625 in.) appears to be in error. Please revise this section and the pipe stability calculations provided accordingly, where applicable.

20. **Wall Crushing:** Based on the compressive strength value (1600 psi) provided in Attachment 1 for HDPE pipe, the compressive strength value reported in this and other sections of Appendix D (1500 psi) appears to be in error. Please revise this section and the pipe stability calculations provided accordingly, where applicable.

21. **Wall Buckling:** The assumed values for Young's modulus and Poisson's ratio appear to be interpolated from the Selig reference provided in Attachment 2, assuming 90% standard Proctor compaction. However Specification 2200-3.07E. indicates that the general fill and subgrade will be compacted to 95% standard Proctor. Please explain this apparent discrepancy and revise this section and the pipe stability calculations provided accordingly, where applicable. Please explain the assumed "average value" for the "Empirical factor."

22. **Summary:** The construction drawings appear to indicate that the 4" SDR-11 HDPE pipes will be constructed adjacent to the 6" pipes within the FLSC. Therefore it does not appear that the 4" pipes "will be subjected to a substantially smaller loading stress…." Please provide pipe stability calculation for the 4" pipe.

Appendix E - Anchor Trench Design Calculation Package

23. **HDPE Geomembrane Material Properties:** The tensile strength utilized for the anchor trench calculations (90 lb/in) is inconsistent with that specified in Specification 2770-Table 2770-1 (72 lb/in). Please revise the anchor trench calculations or the referenced specification to address this discrepancy.

24. **Attachment 2 - Typical Interface Friction Values:** Please provide copies of the references sources for the assumed interface friction values provided in this Attachment.

Appendix F - Perimeter Berm Stability Calculation Package, Rule 62-701.410, F.A.C.

25. **FLSC Configuration:** This section indicates that the FLSC perimeter berm has an 8-foot wide crest while the Representative Cross Section shown in Attachment 1 and the perimeter berm stability calculations in Appendix F assume a 7-foot wide crest. The construction drawings show 8-foot wide crest on the perimeter berm and division berm and a 12-foot wide crest on the separator berm. Please revise this section, the calculations in Appendix F, and/or the construction drawings, as applicable based on the perimeter crest widths proposed for the FLSC.

26. **Method of Analysis:** Please provide a copy of the "sliding block methodology" reference utilized for the sliding block analysis.

27. **Attachment 2 - Rotational Foundation Stability Analysis:**

a. The slide analysis information indicates that the unit weight for the berm material was assumed to be 115 lb/ft^3, while the Representative Cross Section in Attachment 1 indicates that the unit weight of the berm material is 120 lb/ft^3. Please explain this discrepancy and revise the rotational stability analysis and/or Attachment 1, as applicable.

b. Please explain the "Hu" value and the rationale for the value assumed.

c. The assumed water table elevation of 17.5 NGVD in the rotational stability analysis appears to be inconsistent with the 16.5 NGVD water table elevation reported throughout the remainder of this application. Please revise the rotational stability analysis accordingly.

Appendix G - Liner System Leakage and Lateral Drainage Capacity Calculation Package, Rule 62-701.400,F.A.C.

The calculations provided in Appendix G including several references to supporting documents that were the source of assumptions, referenced values, and equations utilized for the calculations. However copies of the relevant sections of many of those documents were not provided and therefore the Department was unable to verify the validity of the assumptions, values, and equations utilized in those calculations. Please provide copies of the relevant sections of all references utilized in each of the calculations. The calculations in Appendix G will be will be reviewed in their entirety upon receipt of the supporting references and the information requested below.

28. Sheet 7 of the construction drawings depicts the bottom FLSC liner installed directly on top of the primary leak detection outflow and instrumentation pipes on the FLSC side slopes. Please explain how this liner system configuration is considered in the liner leakage calculations.

29. Please provide leachate collection system filter fabric (geotextile) design calculations.

Appendix H - Technical Specifications, Rules 62-701.400(3), (7) and (8)

Please revise the Technical Specifications and/or other referenced application documents, as appropriate, to address the following comments and/or inconsistencies.

30. Please provide the Technical Specifications for "Concrete" referenced in Section 12 of the Construction Quality Assurance (CQA) Plan.

31. **Section 02200 - Earthwork**

a. §1.04.A. The referenced Sections 2230 and 2240 in this section were not provided. Please revise this section or provide these specification sections, as applicable.

b. §1.05.B. Please indicate who will provide equipment and labor to assist the CQA Consultant.

c. §2.01.A. & 3.06.B. Please identify the borrow source for fill material for this project.

d. §3.05. Please note that dewatering may require an Industrial Waste Permit from the Department. Please specify who will be responsible for obtaining any necessary dewatering permits from the Department.

e. §3.07.A. Please specify that stones or ruts shall be no larger than 1", consistent with Section 7.4 of the CQA Plan.

32. **Section 02240 - Geocomposite**

a. §1.04.A. The referenced Table 02740-1 is missing from this section. Please provide.

b. §2.05. Please specify the storage limits for the geocomposite consistent with Section 9.2 of the CQA Manual.

c. §3.02.B.1. The bottom layer overlap specified in this section is inconsistent with that specified in Section 9.5 of the CQA Plan.

33. **Section 02270 - Geomembrane**

a. §3.03.C.5.e. Allowance for wrinkles of up to 4 inches of does not appear to provide for "intimate contact" as specified in this section. Please explain and revise this section accordingly.

b. §3.03.C.5.e. Geomembrane installation shall not occur during non-daylight hours and shall not be approved by the Engineer. Please revise this section accordingly.

c. §3.04.D.1. Please specify the geomembrane panel overlap consistent with Section 6.7.5 of the CQA Plan.

d. §3.04.E.3. Please specify that seam will be aligned with no "fishmouths."

e. §3.04.J.2. The sampling and testing methods specified in this section are inconsistent with those specified in Section 6.7.9.3 of the CQA Plan.

f. Table 02770-1 Please specify the Oxidative Induction Time property for the geomembrane. The tensile strength (at break) property provided appears incorrect. Please verify and revise, as appropriate.

g. Table 02770-2 Please specify seam shear strength properties that are at least 90% of the minimum yield strength for the geomembrane, in accordance with Rule 62-701.400(2)(d), F.A.C.

34. **Section 16651 - Control Panel Fabrication**

a. §2.02. The reference to "two" FLSC in this section appears to be inconsistent with the four proposed in this application.

Appendix I - Construction Quality Assurance Plan, Rules 62-701.400(3), (7) & (8)

Please revise the CQA Plan and/or other referenced application documents, as appropriate, to address the following comments and/or inconsistencies.

35. **Section 3 - Project Organization and Personnel**

a. §3.9. Please specify that the geosynthetics installer obtains samples as required by the CQA Plan, under the direction of CQA personnel.

36. **Section 4 - Documentation**

a. §4.6. Please specify that copies of photographs referenced in Section 4.3 will be part of the Certification Report.

37. **Section 6 - Geomembrane**

a. §6.7.2. No "alternate process" for seaming has been specified in the Technical Specifications. Please revise this section to eliminate this option or provide technical specifications for "alternate processes."

b. §6.7.4. Please specify that seam will be aligned with no "fishmouths."

c. §6.7.7. Geomembrane seaming shall not occur during non-daylight hours. Please revise this section accordingly.

d. §6.7.8. Please provide technical specifications for spark testing.

e. §6.7.9.5. Please specify that all five destructive test specimens shall pass laboratory CQA testing consistent with Section 02770-3.04.J.3. of the Technical Specifications.

f. Table 6-1 Please revise this table to indicate that a <u>minimum</u> of one conformance test per 100,000 square feet of material shall be conducted for geomembrane/geocomposite interface shear strength.

38. **Section 8 - Geotextiles**

a. §8.2. This section is inconsistent with Technical Specification 02720-2.05.C. that specifies that geotextile rolls shall not be stored for greater than 6 months.

b. §8.3. This section is inconsistent with the Construction Drawings, which appears to indicate that geotextiles will not be anchored in the anchor trench.

c. §8.6. Please revise Technical Specification 02720 to provide specifications for equipment ground pressure of geotextile overlying geomembrane as indicated in this section, as appropriate.

d. Table 8-1 Please revise this table to correct the reference (5) typographic error.

39. **Section 9 - Geocomposites**

a. §9.4. This section is inconsistent with the Construction Drawings, which appears to indicate that the geocomposite will not be anchored in the anchor trench.

b. §9.5. This section is inconsistent with Technical Specification 02740-3.02.C., which specifies that adjacent geonet edges will overlap a minimum of 4 inches and Technical Specification 02740-3.02.B.2., which specifies that horizontal seams can be 1/3 up a greater than 10H:1V side slope.

c. Table 9-1 Please revise this table to indicate that a minimum of one conformance test per 100,000 square feet of material shall be conducted for geomembrane/geocomposite interface shear strength.

40. **Section 10 - Pipes and Fitting**

a. §10.1. Technical Specification 02715 does not appear to provide specification for FLSC gas system installation, as described in this section. Please explain and revise, as appropriate.

Please **respond within 45 days** after you received this letter, responding to all of the information requests and indicating when a response to any unanswered questions will be submitted. If the response will require longer than 45 days to develop, you should develop a specific timetable for the submission of the requested information for Department review and consideration. Pursuant to the provisions of Rule 62-4.055(1), F.A.C., if the Department does not receive a timely, complete response to this request for information the Department may issue a final order denying your application. A denial for lack of information or response will be unbiased as to the merits of the application. The applicant may reapply as soon as the requested information is available.

You are requested to submit 4 copies of your response to this letter as one complete package with an original and two copies of all correspondence (with one copy sent to Ms. Susan Pelz). It is recommended that you may want to contact the Department to set up a meeting to discuss this letter and subsequent submittals. Please contact me at (813) 632-7600 ext. 385 to schedule the meeting.

Sincerely,

Steven G. Morgan
Solid Waste Section
Southwest District

SM/sgm
Attachments
cc: Ayushman Gupta, P.E., GeoSyntec Consultants, 14055 Riveredge Dr., Suite 300, Tampa, Fl. 33637 w/attachments
Richard Tedder, FDEP Tallahassee, w/attachments
Fred Wick, FDEP, Tallahassee, w/attachments
John Morris, P.G., FDEP Tampa w/attachments
Susan Pelz, P.E., FDEP Tampa

Memorandum

Florida Department of Environmental Protection

TO:	Steve Morgan
FROM:	John R. Morris, P.G. JRM
DATE:	December 11, 2006
SUBJECT:	Sarasota Central Solid Waste Disposal Complex Flexible Leachate Storage Containers, Pending Construction Permit #130542-005-SC Environmental Monitoring Review Comments (RAI #1)
cc:	Susan Pelz, P.E.

I have reviewed portions of the materials submitted to the Department in support of the referenced application for the construction permit associated with the proposed flexible leachate storage containers that were received on November 13, 2006. My review focused on the hydrogeologic and environmental monitoring aspects of the application. Please have the applicant submit responses to the following review comments that provide revised submittals, or replacement pages to the submittals, that use a ~~strike through~~ and underline format, or similar format, to facilitate review. Please also have the applicant include the revision date as part of the header/footer for all revised pages (text, figures, tables, appendices, forms and site plans). The information requests have been referenced to sections of the permit application and are also referenced to the sections of the supporting documents where appropriate, as presented below:

DEP FORM NO. 62-701.900(1), SOLID WASTE MANAGEMENT FACILITY PERMIT FORM
SECTION B – DISPOSAL FACILITY GENERAL INFORATION

1. **B.13.:** The "Yes" response on this item of the application form is inconsistent with the same item of the application form received September 20, 2002 that was associated with the renewal of the operations permit for the facility (permit #130542-002-SO). In the event that a Declaration to the Public has been filed with the Sarasota County Clerk's office that meets the requirements of Rule 62-701.610(5), F.A.C., please submit a certified copy of the declaration. In the event that a Declaration to the Public has not been filed for the facility, please submit a revised application form for this item that indicates a "No" response.

2. **B.17.:** Please provide the basis for the indication that the water table in the vicinity of the flexible leachate storage containers occurs at an elevation of 16 feet NGVD. In the event that this ground water elevation is based on the un-numbered figure included in Appendix C entitled "Monitoring Well Construction Details MW-13" (an approximate ground elevation of 20 feet and depth to water measurement at the time of well installation), please submit additional characterization of the occurrence of ground water at well MW-13 including but not limited to: surveyed top of casing elevation to the nearest 0.01 foot NGVD; surveyed ground surface elevation to the nearest 0.01 foot NGVD; depth to ground water surface below the top of casing measured to the nearest 0.01 foot; and, total well depth below the top of casing measured to the nearest 0.01 foot. Please also submit the details of the well development activities conducted at well MW-13 to demonstrate there is a good connection with the surficial aquifer and that the resultant ground water level measurements are representative of site conditions.

SECTION M – WATER QUALITY AND LEACHATE MONITORING REQUIREMENTS
(Rule 62-701.510, F.A.C.)

3. **M.1.a.:** Please note that sufficient hydrogeological information in the vicinity of the proposed leachate storage containers shall be required to support future modification of the existing monitoring plan for the facility to accommodate the operation of these leachate storage containers. As no routine ground water level measurements are conducted at the portion of the facility where these proposed leachate storage containers are located, the collection additional information is required to supplement available information. Please conduct ground water level measurements at all existing monitor wells, piezometers and staff gauges listed in permit #130542-002-SO and at new well MW-13 at least at a monthly frequency and prepare ground water surface contour maps for each set of water level data to demonstrate the direction of ground water flow. Please submit revisions to Section 3.4 of the "Engineering Report" to specify the direction of ground water flow at the proposed leachate containers determined from these supplemental water level measurements. Please also submit a revised application form for this item that refers to Section 3.4 of the "Engineering Report."

4. **M.1.c.(6):**

a. The indication in Section 3.4 of the "Engineering Report" that well MW-13 is 12 feet deep and is screened from 7 to 12 feet below grade appears to be inconsistent with the un-numbered figure included in Appendix C entitled "Monitoring Well Construction Details MW-13" which indicates well MW-13 is 10 feet deep and is screened from 5 to 10 feet below grade. Please review this apparent inconsistency and submit revisions, as appropriate. Please also submit a revised application form for this item that refers to Section 3.4 of the "Engineering Report."

b. Please note that Rule 62-701.510(3)(d)4, F.A.C., requires the following: "Wells monitoring the unconfined water table shall be screened so that the water table can be sampled at all times. The applicant shall provide technical justification for the actual screen length chosen." The suitability of well MW-13 to meet the requirements of the cited rule will depend on the construction details (requested in comments #2 and #4.a.) and the results of supplemental water level measurements conducted at the facility (requested in comment #3). It is understood that changes to the monitoring plan are not part of this construction permit application but would be associated with a future application for minor modification of permit #130542-002-SO to authorize the operation of the proposed leachate containers. **This comment is presented for informational purposes and does not require a response.**

I can be contacted at 813-632-7600, extension 336, to discuss the comments in this memorandum.
jrm

62-110.106(5). Notices: General Requirements.
Each person who files an application for a Department permit or other notice as may publish or be required to publish a notice of application or other notice as set forth below in this section. Except as specifically provided otherwise in this paragraph, each person publishing such a notice under this section shall do so at his own expense in the legal advertisements section a newspaper of general circulation (i.e., one that meets the requirements of sections 50.011 and 50.031 of the Florida Statutes) in the county or counties in which the activity will take place or the effects of the Department's proposed action will occur, and shall provide proof of the publication to the Department within seven days of the publication.

62-110.106(6). If required, the notice shall be published by the applicant one time only within fourteen days after a complete application is filed and shall contain the name of the applicant, a brief description of the project and its location, the location of the application file, and the times when it is available for public inspection. The notice shall be prepared by the Department and shall comply with the following format:

State of Florida
Department of Environmental Protection
Notice of Application

The Department announces receipt of an application for permit to construct a flexible leachate storage container (FLSC) system, subject to Department rules, at the solid waste management facility referred to as the Sarasota County Central Solid Waste Disposal Complex, located at 4000 Knights Trail Road, Nokomis, Sarasota County, Florida.

This application is being processed and is available for public inspection during normal business hours, 8:00 a.m. to 5:00 p.m., Monday through Friday, except legal holidays, at the Department of Environmental Protection, Southwest District Office, 13051 North Telecom Parkway, Temple Terrace, Florida 33637-0926.

ATTACHMENT 2

REVISED PERMIT DRAWINGS

The Permit Drawings entitled *Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County, Florida* have been revised as needed per RAI No. 1. The revised Permit Drawings are included under a separate cover.

ATTACHMENT 3

Prepared for

Sarasota County

Solid Waste Operations

Central County Solid Waste Disposal Complex

4000 Knights Trail Road
Nokomis, Florida 34275

APPLICATION FOR A PERMIT TO CONSTRUCT FLEXIBLE LEACHATE STORAGE CONTAINERS AT CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX

Prepared by

GeoSyntec Consultants
14055 Riveredge Drive, Suite 300
Tampa, Florida 33637

Project Number FL1109

November 2006
Revised March 2007

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APPENDICES

APPLICATION FOR A PERMIT TO CONSTRUCT FLEXIBLE LEACHATE STORAGE CONTAINERS AT CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX

1. INTRODUCTION

1.1 Terms of Reference

GeoSyntec Consultants (GeoSyntec) has prepared this permit application for the construction and operation of a flexible leachate storage container (FLSC) facility at Central County Solid Waste Disposal Complex (CCSWDC) located in Sarasota County, Florida (west of I-75 and approximately 4 miles northwest of Nokomis). This permit application is submitted to Florida Department of Environmental Protection, Southwest District (FDEP) on behalf of Sarasota County Solid Waste Operations (Sarasota County).

In May 2006, Mr. Ayushman Gupta and Mr. Erik Nelson of GeoSyntec met with Ms. Susan Pelz, Mr. Steve Morgan and Mr. Roger Evans of FDEP regarding the permitting requirements for the proposed FLSC facility. In the meeting, it was agreed that two applications will be submitted to permit the construction and operation of the FLSCs. First, a solid waste application will be submitted to construct the FLSC facility. To operate the FLSCs, a second application will be submitted to modify the existing operation plan for CCSWDC (to include the operations of the FLSCs). In addition, an Environmental Resource Permit (ERP) for the FLSC facility is currently being prepared by GeoSyntec for submittal to FDEP.

The required permit application form, *FDEP Form 62-701-900(1) – Application to Construct, Operate, Modify or Close a Solid Waste Management Facility*, has been completed and is included in Appendix A. This permit application was prepared by Mr. Juan D. Quiroz, Ph.D., P.E. and Mr. Ayushman Gupta, P.E. of GeoSyntec.

1.2 Site Information

CCSWDC is located on a 6,150-acre property at the north end of Knights Trail Road in Nokomis, Sarasota County, Florida in Sections 1 through 4 and 9 through 16 of Township 38S and Range 19E. CCSWDC currently consists of a Class I solid waste landfill. The conceptual master plan for the CCSWDC includes a landfill footprint area of 268 acres, a total waste disposal capacity of about 40 million cubic yards, and an operating life of more than 40 years. The planned CCSWDC landfill development will occur in five phases. In October 1993, FDEP approved the construction permit for Phase 1 of the

CCSWDC landfill, which was subsequently renewed in July 1997. The total landfill footprint of Phase 1 is approximately 60 acres which are divided equally into five 12-acre cells. The operation of Phase 1 started in June 1998.

At present, leachate from Phase 1 is pumped from leachate sumps (located along the northern end of each landfill cell) to a 6 inch diameter high density polyethylene (HDPE) leachate transmission line (i.e., forcemain). The existing leachate transmission line is approximately one mile long and carries leachate from the landfill cells to the site's leachate storage facility located south of the existing landfill. The leachate storage facility consists of a double-contained concrete storage tank. The open-top, cylindrical leachate storage tank has an inside diameter of 100 ft and height of 30 ft, corresponding to a total storage capacity of about 1.8 million gallons. The secondary containment tank has an inside diameter of 130 ft and height of 19 ft. The leachate is currently trucked from the storage tank to a wastewater treatment plant for disposal.

The storm water management system for the site generally consists of sheet flow to surface water drainage channels and then to storm water ponds located throughout the site. Specifically, storm water from Phase 1 and future landfill phases is conveyed via surface water drainage channels (and culverts) along the interior side slope of the existing landfill area perimeter roads to Storm Water Pond Nos. 1 and 2 located northwest and southwest of Phase 1, respectively.

1.3 Organization of the Report

This report is organized into five sections. Following this introductory section:

- Section 2 provides the project background and basis for this permit application;
- Section 3 presents the FLSC facility, specifically the layout and configuration and general operation procedures;
- Section 4 presents the FLSC facility design evaluation and associated calculation packages;
- Section 5 outlines the construction quality assurance (CQA) plan and technical specifications; and
- Section 6 provides a summary of the FLSC design and permit application.

2. PROJECT BACKGROUND

In the last few years Sarasota County has experienced extreme rainfall events that have required more leachate storage capacity than is currently available at CCSWDC to maintain the required minimum head on the bottom liner system of Cells 1 through 4 within the Phase 1 landfill. Under emergency situations, additional leachate storage capacity was potentially provided by Cell 5, a lined landfill cell. However, as waste filling activities progress into Cell 5, an alternative leachate storage system is required for emergency situations.

In addition, the extreme rainfall events have occasionally caused leachate breakouts (from Phase 1) that have impacted storm water in the surface water drainage channel north of Phase 1. In the past, any storm water that may have been impacted by the leachate breakouts was contained within the northern surface water drainage channel until water quality testing was completed to verify whether the "impacted" storm water needs to be treated or could be safely discharged to Storm Water Pond No. 1. As a result, an improved impacted storm water emergency storage system is also desired by Sarasota County.

3. FLEXIBLE LEACHATE STORAGE CONTAINER FACILITY

3.1 Overview

Sarasota County proposes to construct and operate a FLSC facility at CCSWDC to store additional leachate and impacted storm water at the site in emergency situations. The FLSC facility will provide a leachate storage capacity of approximately 600,000 gallons. Accordingly, the total on-site leachate storage capacity will increase from 1.8 million gallons to 2.4 million gallons. The FLSC facility will also provide a 600,000 gallon storage capacity for impacted storm water.

Additional details regarding the proposed FLSC facility are provided in the proposed Permit Drawings titled *Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex, Sarasota County*, dated November 2006, which are included in Appendix B of this permit application. The following sections discuss the details associated with the layout, configuration and operation of the FLSC facility.

3.2 Layout and Configuration

As shown on Sheet 2 of the Permit Drawings, the proposed FLSC facility will be located at the northwest corner of the open field that lies south of the existing Phase 1 landfill and west of the on-site maintenance building area. The FLSC facility was situated

such that it could tie-in to the existing leachate forcemain that runs in a southerly direction from Phase 1 to the existing on-site leachate storage facility. To convey impacted storm water the FLSCs, an approximately 7,950-ft long, 6-inch diameter HDPE conveyance pipe will be installed along the interior side slope of the site's perimeter road from Storm Water Pond No. 1 to the proposed FLSC facility location (see Sheet 2 of the Permit Drawings). The conveyance pipe will eventually cross under the access road that lies just north of the proposed location of the FLSC facility. In addition, weir structures will be constructed at the inlet locations to Storm Water Ponds Nos. 1 and 2. These structures will prevent discharge of impacted storm water into the respective ponds from Phase 1 and future landfill phases.

The FLSC facility will be built-up relative to existing ground, as shown on Sheet ~~3~~ 5 of the Permit Drawings. A perimeter berm approximately 7 ft high with 3H:1V side slopes and an 8-ft wide crest will encompass the FLSCs. Interior division berms will also be constructed to provide separation between each individual FLSC unit. The FLSC facility has a footprint of approximately two acres and consists of four individually lined and sealed storage containers. Each FLSC has a maximum storage capacity of approximately 300,000 gallons. Two of the containers (FLSCs 2A and 2B) will provide leachate storage, while the remaining two containers (FLSCs 1A and 1B) will provide impacted storm water storage under emergency situations. FLSCs 1A and 1B will only be used for impacted storm water. However, FLSCs 2A and 2B may also be used for impacted storm water, if needed.

As shown on Sheet 6 of the Permit Drawings, the FLSCs will be constructed of 60-mil high density polyethylene (HDPE) textured geomembrane. The maximum liquid level within each container is approximately 5 ft. Each FLSC is designed as an individual unit with a double-liner system and individual leachate and leakage collection sumps. The double-liner system consists of the following, from top to bottom:

- Double-sided drainage geocomposite (with 8 oz/yd^2 geotextile on both sides);
- 60-mil HDPE textured geomembrane;
- Double-sided drainage geocomposite;
- 60-mil HDPE textured geomembrane; and
- Geosynthetic clay liner (GCL).

3.3 Operation

In general, the FLSC facility will be utilized only in emergency situations when: (i) additional leachate storage capacity is temporarily required; and (ii) potentially impacted storm water is required to be temporarily stored (until water quality testing is completed). The FLSC facility will tie-in to the existing leachate forcemain that conveys leachate from Phase 1 to the existing on-site leachate storage facility. A system of check valves and inflow/outflow pipes will be utilized to convey leachate and/or impacted storm water to and from the FLSC facility (see Sheets 8 through 11). Specific procedures have been developed for each circumstance, and are summarized below.

If additional leachate storage capacity is required, leachate flow can be diverted from the existing leachate forcemain to the designated FLSCs. Once the downstream leachate storage facility is restored to normal operating conditions and can accommodate the leachate volume temporarily stored within the FLSCs, the leachate will be pumped back into the existing leachate management system for disposal. The FLSCs will be emptied and remain empty until additional, emergency storage capacity is required.

If storm water is potentially impacted from Phase 1 or future landfill phases, the weir structures located at the inlet locations of Storm Water Ponds Nos. 1 and 2 (see Sheet 2 of the Permit Drawings) will be raised to prevent discharge to the respective ponds and contain the impacted storm water within the surface water drainage channels. A submersible pump will be placed on the concrete pad (see Sheet 12 of the Permit Drawings) on the side of the weir where the impacted water is contained. The pump will be connected to the impacted storm water pipeline at the adjacent cleanout locations shown on Sheet 2 of the Permit Drawings. The impacted storm water will then be pumped via the impacted storm water pipeline to the FLSC facility, and sampled for water quality testing. If water quality testing indicates that the "impacted" storm water can be safely discharged, then it will be pumped from the containers to the surface water drainage channel north of and across the road from the FLSC facility. This surface water drainage channel will eventually discharge to Storm Water Pond No. 2. If water quality testing indicates that the impacted storm water requires treatment, then it will be pumped into the existing leachate management system for disposal. The FLSCs will be emptied and remain empty until additional, emergency storage capacity is required.

Ponded storm water that accumulates on top of the FLSCs and does not evaporate will be removed, as needed, using a small submersible pump. The pump will be lowered into position on top of the FLSC using an extension rod such that the top geomembrane layer of the FLSC is not damaged. Storm water will be pumped to the outer slope of the perimeter berm.

Additional operation details of the FLSCs will be provided in a second (i.e., permit modification) application that will be submitted to modify the operations plan for CCSWDC and include the operations of the FLSCs.

3.4 Subsurface Investigation and Monitoring Well Installation

A subsurface investigation was conducted to evaluate the subsurface profile and corresponding geotechnical properties of the foundation soils in support of the proposed FLSC facility. One soil boring, designated as GB-1, was performed at approximately the center of the FLSC facility footprint. Continuous Standard Penetration Tests (SPTs) with a split-spoon barrel were conducted in the hollow-stem augered borehole to provide N-values (blows/ft) and a continuous visual examination of the soil profile. The depth of the SPTs and split-spoon sampling in the boring was continued until refusal (i.e., blows/ft greater than 50) at approximately 20 ft below ground surface. The borehole was subsequently backfilled with Bentonite pellets. The boring was performed on 1 August 2006 by National Environmental Technology, Inc. (Dover, Florida) under the field direction/monitoring of GeoSyntec personnel. The soil boring log is included in Appendix C of this permit application.

The foundation soils beneath the FLSCs generally consist of loose to medium dense fine sands and silty sands. The observed ground water table at the time of the boring was about 3.5 ft below the ground surface. Laboratory geotechnical testing was performed on select soil samples obtained during soil boring GB-1. The laboratory geotechnical tests performed consisted of grain size analyses, which were used to classify the soils and confirm the visual descriptions presented in the soil boring logs. The testing was performed by Excel Geotechnical Testing, Inc. (Roswell, Georgia). The results of the laboratory geotechnical tests are provided in Appendix C of this permit application.

The ground water at the site generally flows in a southwest direction. A shallow monitoring well, designated as MW-13, was installed approximately 50 ft southwest of the outer toe of the proposed FLSC facility perimeter berm (see Sheet 3 of the Permit Drawings), and is located at an assumed down-gradient location with respect to general ground water flow at the site. The monitoring well is approximately 10 ~~12~~-ft deep with a screen length of 5 ft along the lower portion of the well (i.e., from a depth of approximately 5 ~~7~~ ft to 10 ~~12~~ ft below the existing ground surface). The monitoring well was installed on 1 August 2006 by National Environmental Technology, Inc. (Dover, Florida) under the field direction/monitoring of GeoSyntec personnel. The monitoring well installation log and well completion report is provided in Appendix C of this permit application. Monthly ground water measurements at all existing monitoring wells, piezometers and staff gauges

listed in Permit #130542-002-SO and at the monitoring of the well (identified as MW-13) adjacent to the proposed FLSC facility will be performed to confirm the direction of ground water flow. Monthly monitoring will commence in April 2007, and the required information will be forwarded to the Department.

3.5 Prohibitions

The FLSC facility satisfies FDEP's siting criteria as stated in Rule 62-701.300(2), F.A.C. Leachate or impacted storm water will not be stored or placed:

- in an area where geological formations or other subsurface features will not provide adequate support (stability of the FLSCs is discussed in detail in Appendix F of this permit application);
- within 500 ft of any existing or approved potable water well or within 1,000 ft of any community water supply well;
- in de-watered pits;
- in a natural or artificial body of water;
- in an area subject to frequent and periodic flooding;
- within 200 feet of a wetland (or body of water) except where the facility is designed with permanent leachate control methods, which will result in compliance with water quality standards and criteria (liner system leakage calculations for the FLSCs are provided in Appendix G); or
- on the right-of-way of any public highway, road, or alley.

4. DESIGN EVALUATIONS

4.1 Overview

The following design aspects were evaluated in support of the proposed FLSC facility: (i) impacted storm water conveyance pipe stability; (ii) liner system anchor trench design; (iii) perimeter berm stability; and (iv) liner system leakage and lateral drainage capacity. A summary of each evaluation is provided below.

4.2 Conveyance Pipe Stability

The structural stability of the impacted storm water conveyance pipe and FLSC facility leachate pipes (see Sheets 2 and 3 of the Permit Drawings) was evaluated with respect to

applied overburden and/or traffic loading. The pipe stability analyses are presented in the calculation package titled *Pipe Stability Evaluation*, which is included in Appendix D of this permit application. Based on the pipe stability calculations that consider wall crushing, wall buckling, excessive ring deflection and excessive bending strain, the proposed conveyance pipes provide adequate structural stability with respect to the applied external loads.

4.3 Liner System Anchor Trench Design

The adequacy of the liner system anchor trench design was evaluated for the FLSC facility. As presented on Sheet 6 of the Permit Drawings, the anchor trench located along the top crest of the perimeter berm will be constructed to hold in-place the liner system geosynthetics. The liner system anchor trench design evaluation is presented in the calculation package titled *Anchor Trench Design Evaluation*, which is included in Appendix E of this permit application. Based on the anchorage calculations and FLSC loading conditions, the proposed anchor trench depth of 2 ft is adequate relative to geosynthetic pullout resistance.

4.4 Perimeter Berm Stability

The impact of the FLSCs on the global stability of the perimeter berms was evaluated. Two analyses were performed: (i) sliding stability along the base of the berm; and (ii) rotational (foundation) slope stability of the berm. The stability analyses are presented in the calculation package titled *Perimeter Berm Stability*, which is included in Appendix F of this permit application. The results of the sliding stability analysis indicate that the perimeter berm provides adequate buttressing for the proposed FLSCs. Similarly, the results of the foundation slope stability analysis indicate that the perimeter berm and subsurface soils provide adequate foundation support for the FLSCs.

4.5 Liner System Leakage and Lateral Drainage Capacity

The rate of leakage through the FLSCs, and primary and secondary liner systems (see Sheet 6 of the Permit Drawings) was evaluated. These leakage rates were then utilized to evaluate the conveyance capacity of the proposed primary and secondary leachate collection layers such that specified maximum allowable heads on the liner were not exceeded. Finally, the sump pumps were sized accordingly to prevent head build-up within each leachate collection layer.

The liner system leakage and lateral drainage capacity calculations are presented in the calculation package titled *Liner Leakage and Lateral Drainage Capacity Evaluation*,

which is included in Appendix G of this permit application. Based on the liner system leakage calculations, the actual leakage rate through the secondary liner is negligible since the FLSC facility will be used for a limited time under emergency situations only. In addition, the maximum calculated head-on-liner values are less than or equal to the specified maximum allowable heads that were limited to: (i) 12 inches for the primary leachate collection layer; and (ii) the thickness of the lateral drainage layer for the secondary leachate collection layer. These head-on-liner results indicate that the lateral drainage capacity of the proposed leachate collection layers is adequate.

The sumps will be instrumented with leak detection transducers that will activate an alarm light; and pumping of the sumps will be performed on an as needed basis. If a leak is detected in the primary or secondary sump, the accumulated leachate will be pumped back into the respective FLSC. Since the proposed FLSCs will be used for a limited time under emergency situations only, any leachate that is re-introduced back into the FLSC from the respective primary or secondary sump will not increase the amount of total calculated leakage through the FLSC liner system as presented in Appendix G. As such adequate leachate storage and containment is still provided by the FLSCs. Additional leachate sump operational details are provided in the liner system leakage and lateral drainage capacity calculation package (Appendix G).

5. TECHNICAL SPECIFICATIONS AND CONSTRUCTION QUALITY ASSURANCE PLAN

It is assumed that the FLSCs will be constructed with high quality materials, that good construction practices will be followed, and that a very good construction quality assurance (CQA) program will be implemented. The *Technical Specifications* for all construction materials are presented in Appendix H, and the *CQA Plan* is presented in Appendix I.

ATTACHMENT 4

FLSC Cell Floor Subgrade Settlement Calculations

A Gupta
3/29/07

Geosyntec consultants

Written by: JUAN QUIROZ Date: 26/03/07 (DD MM YY) Reviewed by: _____ Date: __/__/__ (DD MM YY)

Client: SARASOTA Co Project: FLSC Project/Proposal No. FL1109 Task No. _____

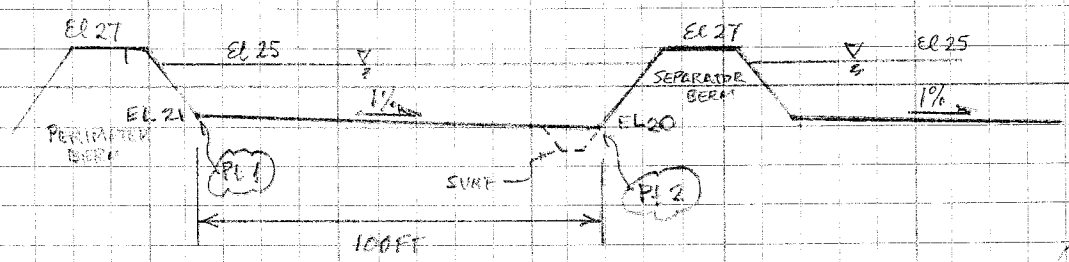
SUBGRADE SETTLEMENT OF FLSC CELL FLOOR

References: 1. Lambe, T.W. and Whitman, R.V. Soil Mechanics. John Wiley and Sons, Inc., New York, 1969.

2. Schmertmann, J.H. "Static Cone to Compute Settlement Over Sand," Journal of the Soil Mechanics and Foundation Division ASCE, Vol. 96, No. SM3, pp. 1011-1043, 1970.

Given:

Typical FLSC section (see sheets 4 & 5 Permit Dwgs)



* Refer to attached subsurface profile and Soil Boring GB-1

From App. C of the November 2006 FLSC Engineering Report

Req'd: Calculate: 1. Total settlement of Points 1 and 2

2. Differential settlement

3. Post-Settlement slope along FLSC cell floor.

Geosyntec consultants

Written by: JUAN QUIROZ Date: 26/03/07 (DD MM YY) Reviewed by: _____ Date: __/__/__ (DD MM YY)

Client: SARASOTA CO Project: FLSC Project/Proposal No. FL1109 Task No. _____

Sol'n:

(1) Elastic Settlement can be calculated using one-dimensional stress-strain relationships [Lambe and Whitman, 1969]:

$$\Delta s = \sum \Delta \ell \frac{\Delta \sigma}{D}$$

Δs = elastic settlement

$\Delta \ell$ = incremental layer thickness

D = constrained modulus of elasticity $= \frac{E_s(1-\mu)}{(1+\mu)(1-2\mu)}$

E_s = soil modulus of elasticity $= 8 \times N$ (units = ton/ft²) where N = BLOWS/FT

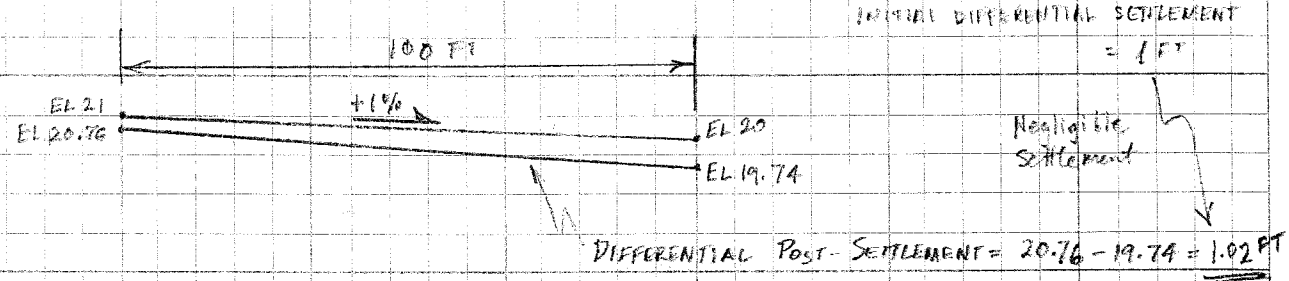
PER SOIL BORING GB-1

μ = Poisson's ratio (= 0.3 to 0.45 for cohesionless soils)

$\Delta \sigma$ = incremental vertical stress

See attached spreadsheet calculations for total settlement values at Points 1 & 2.

(2) DIFFERENTIAL POST-SETTLEMENT



(3) POST-SETTLEMENT SLOPE (NEGATIVE SLOPE INDICATES GRADE REVERSAL)

$$S = \frac{20.76' - 19.74'}{100'} = +1.02\%$$ W/POSITIVE DRAINAGE MAINTAINED TOWARD ~~THE SUMP. (I.E., NO GRADE REVERSAL)~~

Subgrade Settlement Below Cell Floor Point 1
Flexible Leachate Storage Containers
Contral County Solid Waste Disposal Complex - Sarasota County, Florida

Layer No.	Soil Type	Elev. Top of Layer (ft)	Elev. Bottom of Layer (ft)	ΔI Layer Thickness (ft)	Mid-Depth Below Grade (ft)	Δσ Vertical Stress (psf)	Average SPT N-value	E_s (1) Modulus of Elasticitv (psf)	D Constrained Modulus of Elasticity (psf)	ε (Strain)	ΔSettlement (ft)	ΔSettlement (in)
1	Loose Fine Sand & Silty Sand	21	10	11	15.5	262	8	128000	189651	0.001	0.015	0.18
2	Medium Dense Silty Sand	10	0	10	5.0	262	24	384000	568952	0.0005	0.005	0.06
					Weighted Average =			249905			**Total =**	**0.24**

Notes: (1) The Modulus of Elasticity of the soil (E) is based on the relation presented by Schmertman (1970) for sands (i.e., E = 8*N where N is the SPT N-value).
(2) The calculations are based on the following input parameters:

Poisson Ratio (μ) =	0.33	
Unit Weight of Water (γ_{water}) =	62.4	pcf
Height of Water (H_{water}) =	4.2	ft
Vertical Stress Increment ($\Delta\sigma$) =	262	psf

Subgrade Settlement Below Cell Floor Point 2
Flexible Leachate Storage Containers
Contral County Solid Waste Disposal Complex - Sarasota County, Florida

Layer No.	Soil Type	Elev. Top of Layer (ft)	Elev. Bottom of Layer (ft)	ΔI Layer Thickness (ft)	Mid-Depth Below Grade (ft)	Δσ Vertical Stress (psf)	Average SPT N-value	E_s (1) Modulus of Elasticity (psf)	D Constrained Modulus of Elasticity (psf)	ε (Strain)	ΔSettlement (ft)	ΔSettlement (in)
1	Loose Fine Sand & Silty Sand	20	10	10	15.0	312	8	128000	189651	0.002	0.016	0.20
2	Medium Dense Silty Sand	10	0	10	5.0	312	24	384000	568952	0.001	0.005	0.07
					Weighted Average =			256000			**Total =**	**0.26**

Notes: (1) The Modulus of Elasticity of the soil (E) is based on the relation presented by Schmertman (1970) for sands (i.e., E = 8*N where N is the SPT N-value).
(2) The calculations are based on the following input parameters:

Poisson Ratio (μ) =	0.33	
Unit Weight of Water (γ_{water}) =	62.4	pcf
Height of Water (H_{water}) =	5	ft
Vertical Stress Increment ($\Delta\sigma$) =	312	psf

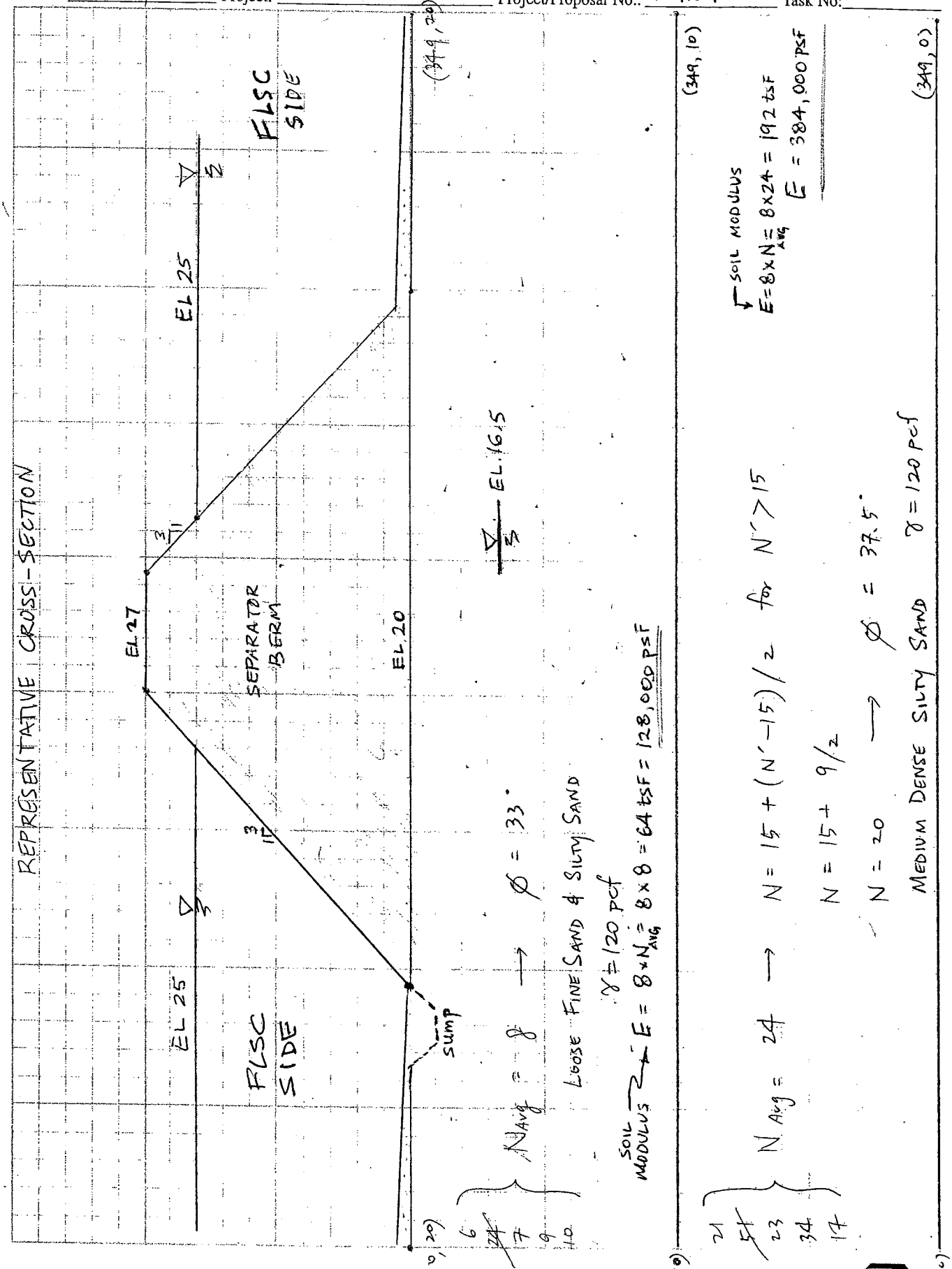
FLSC Subsurface Information

GEOSYNTEC CONSULTANTS

Page _____ of _____

Written by: JUAN QUIROZ Date: 07/03/23 (YY/MM/DD) Reviewed by: _____ Date: ___/___/___ (YY/MM/DD)

Client: Sarasota Project: FLSC Project/Proposal No.: FL1109 Task No: _____



GEOLOGIC SOIL BORING DETAIL GB-1

PROJECT: CCSWDD-SARASOTA CO.	DATE COMPLETED: AUGUST 2006
PROJECT NO.: FL0819.02	CONTRACTOR: NET
DATE STARTED: 1 AUGUST 2006	DRILLER: NA
DRILL RIG: NA	SCALE: 1"=5'
METHOD: AUGER BIT	DRAWN BY: ENDRE CSORDAS
GROUND ELEV.: 20 FT	DRAWING NO.: FL0819.01F002
WATER AT INSTALL: 3.5 FT	CHECKED BY: JUAN QUIROZ
GEOLOGIST: JOE TERRY	SHEET: 1 OF: 1

DEPTH (FEET)	% FINES	UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL	BLOWS / 6 IN	N-VALUE	REC / ATT	LITHOLOGICAL DESCRIPTION
	NA	SP	1 2 4 6	6	95%	LIGHT TO DARK BROWN FINE SAND W/GRAY AND BLACK ORGANICS (ROOTS) MOIST.
	10.3	SP-SM	14 14 10 9	24	100%	LIGHT TO DARK BROWN FINE SAND (FEW ROOTS TOWARDS THE TOP), MOIST TO SATURATED
5			3 3 4 5	7	100%	DARK TO LIGHT BROWN FINE SAND, SATURATED
	10.2	SP-SM	4 3 6 8	9	100%	LIGHT BROWN AND GRAY SILTY FINE SAND W/ TRACE CLAY, WET
10	19.7	SM	2 5 5 5	10	80%	GRAY SILTY FINE SAND, WET
	56.3	CL	6 6 15 30	21	90%	LIGHT BROWN AND GRAY VERY FINE SAND W/LITTLE GRAVEL (AT 11.5 FT) AND TRACE CLAY AT 12 FT, MOIST
			8 12 39 50	51	98%	LIGHT BROWN TO TAN SILTY CLAY, MOIST
15			10 8 15 26	23	90%	TAN SILTY CLAY AND LITTLE FINE MATERIAL (GRAVEL) TOWARDS BOTTOM, MOIST
	NA	ML-CL	27 20 14 11	34	100%	LIGHT BROWN AND TAN FINE SAND SILT W/LITTLE TO SOME CLAY (GREENEISH-BROWN) AND LITTLE GRAVEL, MOIST
			5 7 10 25	17	90%	TAN MEDIUM TO FINE SAND, SOME SILT AND CLAY (GREENISH GREEN); LIME ROCK (YELLOWISH) W/SILT AND SAND TOWARDS BOTTOM
25.25 20			50			REFUSAL AT 20.25 FT W/50 BLOWS PER FT FOR 3"

NO WELL INSTALLED

TOTAL DEPTH = 20.25 Feet

N-VALUE

GEOSYNTEC CONSULTANTS
TAMPA, FLORIDA

FIGURE NO.	–
PROJECT NO.	FL0819.02
FILE NO.	FL0819.01F002

ATTACHMENT 5

Prepared for

Sarasota County Solid Waste Operations
Central County Solid Waste Disposal Complex
4000 Knights Trail Road
Nokomis, FL 34275

APPLICATION FOR AN ENVIRONMENTAL RESOURCES PERMIT FOR CONSTRUCTION OF FLEXIBLE LEACHATE STORAGE CONTAINERS AT CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX

Prepared by

Geosyntec consultants

14055 Riveredge Drive, Suite 300
Tampa, FL 33637

Project Number FL0819
March 2007

TRANSMITTAL LETTER

Geosyntec consultants

14055 Riveredge Drive
Suite 300
Tampa, FL 33637

813-558-0990
813-558-9726

14 March 2007

Mr. Douglas Hyman, P.E.
Florida Department of Environmental Protection
Southwest District
13051 N. Telecom Pkwy
Temple Terrace, Florida 33637-0926

Subject: Environmental Resources Permit Application
Flexible Leachate Storage Containers
Central County Solid Waste Disposal Complex
Sarasota County, Florida

Dear Mr. Hyman:

Transmitted herewith are four copies of the Environmental Resources Permit Application for the above referenced facility. Geosyntec Consultants is submitting the application on behalf of the County of Sarasota.

If you, or your staff, have any questions or need additional information, please feel free to contact the undersigned.

Sincerely,

Erik J. Nelson, P.E.
Senior Engineer

Enclosures

J:\TWP\FL\FL0819-Flex Bags Sarasota\ERP\Reviewed\Transmittal.doc

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INTRODUCTION

INTRODUCTION

1. TERMS OF REFERENCE

On behalf of Sarasota County Solid Waste Operations (County), Geosyntec Consultants (Geosyntec) has prepared this Environmental Resources Permit (ERP) application for the construction of flexible leachate storage containers (FLSCs) at the existing Central County Solid Waste Disposal Complex (CCSWDC). The proposed FLSC facility will provide additional (emergency) leachate storage capacity as well as storage capacity for potentially impacted storm water that may be generated within the landfill portion of the site. As part of this project, storm water control weirs will be constructed in the existing perimeter storm water drainage channel around the landfill to provide a means of stopping potentially impacted storm water from entering the on-site storm water ponds, specifically Storm Water Pond Nos. 1 and 2. A pipeline will also be constructed at the site to convey impacted storm water from the weir structures to the FLSC facility.

In July 2006, Mr. Ayushman Gupta and Mr. Juan Quiroz of Geosyntec met with Mr. Doug Hyman and Ms. Allyson Minik of Florida Department of Environmental Protection (FDEP) regarding the permitting requirements for the proposed FLSC facility. In the meeting, it was agreed that an ERP application will be submitted to permit the construction of the FLSCs. Note that a solid waste application to construct the FLSC facility was submitted to FDEP on 9 November 2006, and is currently under review.

The remainder of this application presents a description of the proposed construction and operation of the FLSC facility. This ERP application has been prepared to meet the requirements of the FDEP Form No. 62-343.900(1) titled *Joint Environmental Resource Permit Application* (dated 3 October 1995). This ERP application addresses Sections A, C, and H of the form, which are the sections applicable to this project. This permit application was prepared by Mr. Erik Nelson, P.E. and reviewed by Mr. Juan D. Quiroz, Ph.D., P.E. of Geosyntec.

2. SITE LOCATION

CCSWDC is located in Sarasota County, Florida, east of Interstate Highway 75, and northeast of Nokomis, Florida. The CCSWDC site is located in Sections 2, 3, 10, and 11 of Township 38 South, Range 19 East. The site location is shown in Figure 1. The main entrance of the facility is located at latitude 27° 11' 34", longitude 82° 23' 48", on highway

U.S. 441. The center of the landfill footprint is located at latitude 27° 11' 47" and longitude 82° 23' 09".

3. PROJECT SUMMARY

The proposed FLSCs are designed to provide additional (emergency) leachate storage capacity, as well as storage capacity for potentially impacted storm water. The storage capacity of each FLSC is approximately 300,000 gallons. Four individually lined FLSCs will be constructed at the site. Two of the FLSCs will be constructed to temporarily store leachate and two will be constructed to temporarily store potentially impacted storm water. Piping within the FLSC facility will be constructed to prevent cross-contamination of potentially impacted storm water by leachate. Piping for the FLSC facility will tie directly into the leachate transmission line that runs from the existing landfill to the on-site leachate storage tank. Valves will be installed at this connection to direct incoming leachate to the appropriate FLSCs, as needed. Similarly, valves will be installed to direct outgoing leachate or impacted storm water to the existing leachate transmission line. Storm water that is not impacted will be released to the surface water management system. Each FLSC will be equipped with a submersible pump capable of pumping leachate from the FLSC facility to the existing leachate storage tank or pumping impacted storm water from the FLSC facility to the adjacent surface water management system. The FLSC area is not intended for truck load out. Tankers transporting leachate for treatment will be loaded at the existing leachate storage tank. .

Each FLSC will be constructed of two high density polyethylene (HDPE) geomembrane sheets welded together at the edges to create a bladder-type storage system. Each bladder will supported by perimeter berms constructed of compacted soil. The area formed by the soil berms will be lined and will provide secondary containment for the FLSC. Four bladders will be constructed at the FLSC facility. Sheet 3 of 13 of the Permit Drawings presents the proposed layout for the FLSCs. A copy of the Permit Drawings are included as Appendix A. Each bermed cell area will be lined with a primary HDPE geomembrane liner and secondary composite liner system that consists of an HDPE geomembrane underlain by a geosynthetic clay liner (GCL). A geocomposite drainage layer will be placed between the bottom liner of the FLSC and the primary HDPE geomembrane liner. A second geocomposite drainage layer will be installed between the primary and secondary liner systems. The permit drawings attached to this ERP application provide a detailed description of each component.

Operation of the FLSCs will be done manually. In the event that additional leachate storage capacity is required, the main leachate transmission line from the active landfill cells will be closed off and leachate will be directed to the leachate FLSCs. Each FLSC

will be equipped with a submersible pump which can be used to empty the FLSCs. Leachate in the FLSCs will only be able to be pumped to the existing leachate tank through a pipeline from the leachate FLSCs to the main leachate transmission line.

If storm water from active portions of the existing landfill is potentially impacted, the affected portion of the perimeter storm water drainage channel will be blocked off at the nearest down gradient weir structure. A portable submersible pump will pump impacted storm water collected behind the weir structure through a buried pipeline to the storm water FLSCs. A sample of the storm water will be collected and sent for water quality analyses. If the analytical test results indicate that the storm water can be safely discharged then the storm water will be pumped into the storm water drainage channel located across the road to the north of the FLSC. That section of the drainage channel discharges into Storm Water Pond No. 2. If the analytical data indicates that the storm water cannot be safely discharged, the storm water can either be pumped directly to the existing leachate storage tank or transferred to an adjacent FLSC designated for leachate.

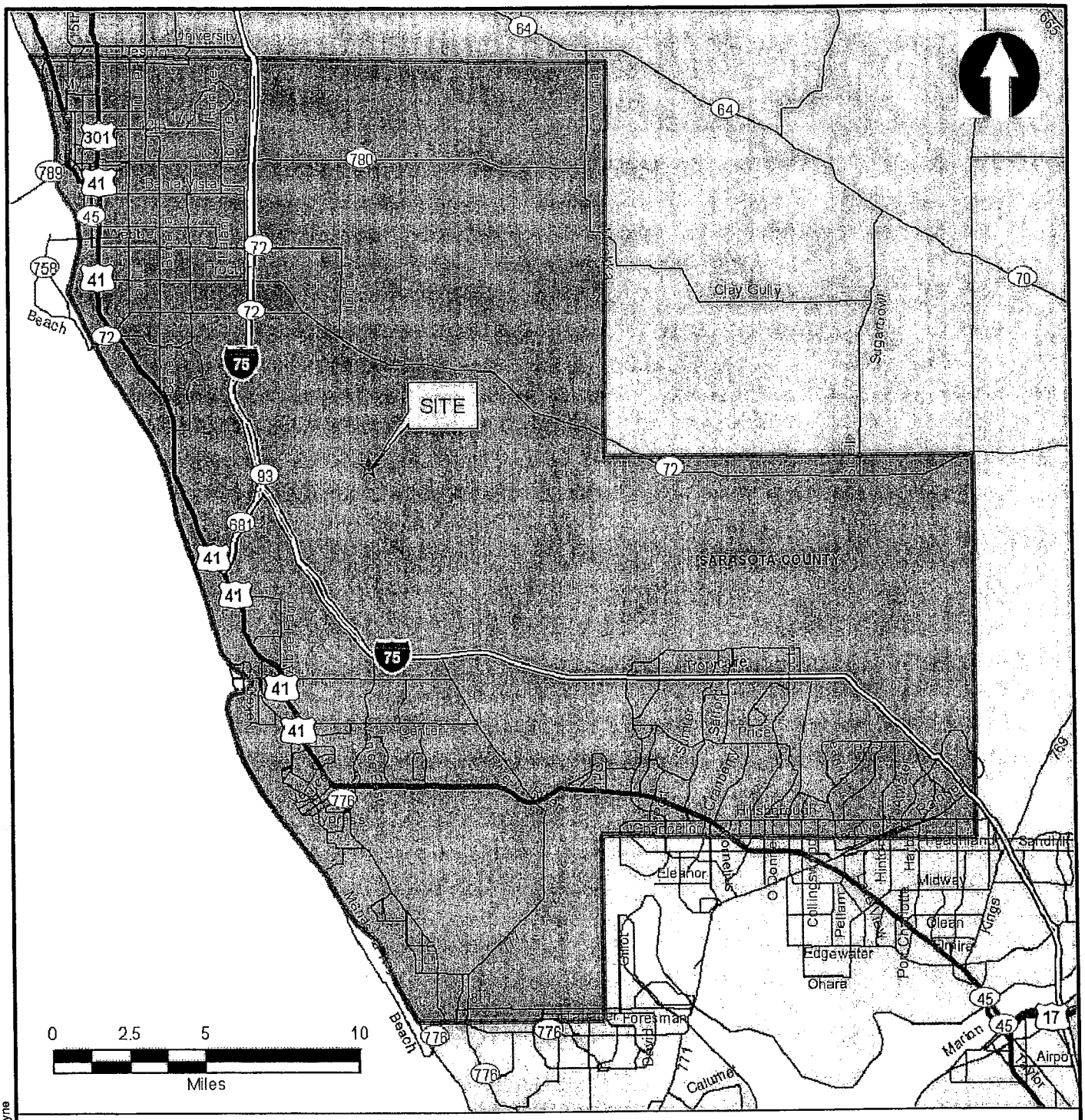
4. ERP SUMMARY

This ERP application describes the impacts of construction of the FLSC facility, and associated impacted storm water pipeline and weir structures on the existing landfill facility. All features of this project will be located within the boundaries of the currently permitted and developed portions of the site. No off-site construction activities are proposed as part of this project. Therefore, no impacts to wetlands, or other undisturbed areas of the site have been proposed.

Any run-off generated during and after construction activities are complete will be routed through the existing storm water management system. The only impervious area created by this project corresponds to the top of the FLSCs. The total area of impervious area created will be approximately 1.5 acres. Precipitation that falls onto the FLSC will be retained on top of the FLSCs. The top liner of the FLSCs will provide a separation barrier between the contents within the FLSCs and the storm water collected on top of it. Therefore, storm water collected on top of the FLSC will be considered clean water and can be released directly to the storm water drainage channel adjacent to the FLSC facility. The storm water will be pumped off of the top of the FLSCs by trash pumps or small submersible pumps if necessary.

5. APPLICATION ORGANIZATION

The organization of this permit application generally follows FDEP Form No. 62-343.900(1). Based on the type and size of proposed development, the County is required to submit Sections A, C, and H of the form. The remainder of this document presents the information required by Sections A, C, and H of the form. Section A provides a description of the project and general information regarding the facility. Section C provides brief descriptions of the proposed construction, surface-water and wetlands impacts. Section H provides more detail regarding the proposed construction and its affect on wetlands, and storm water management. A copy of the Permit Drawings submitted as part of the solid waste application to FDEP on 9 November 2006 is also included as part of this ERP application.



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Geosyntec consultants

TAMPA, FL

DATE:	13 MAR 2007	FILE NO.	FL0819.02F005
PROJECT NO.	FL0819	FIGURE NO.	1

APPLICATION FORM

SECTION A

BASIC APPLICATION FORM

SECTION A

FOR AGENCY USE ONLY

ACOE Application	DEP/WMD Application
Date Application Received	Date Application Received
Proposed Project Lat.	Fee Received $
Proposed Project Long.	Fee Receipt

PART 1:
Are any of the activities described in this application proposed to occur in, on, or over wetlands or other surface waters?
☐ yes ☒ no
Is this application being filed by or on behalf of a government entity or drainage district? ☒yes ☐no

PART 2:

A. Type of Environmental Resource Permit Requested (check at least one). See Attachment 2 for thresholds and descriptions.

☐ Noticed General - include information requested in Section B.
☐ Standard General (Single Family Dwelling) - include information requested in Sections C and D.
☐ Standard General (all other Standard General projects) - include information requested in Sections C and E.
☐ Individual (Single Family Dwelling) - include information requested in Sections C and D.
☒ Individual (all other Individual projects) - include information requested in Sections C and E.
☐ Conceptual - include information requested in Sections C and E.
☐ Mitigation Bank Permit (construction) - include information requested in Sections C and F. (If the proposed mitigation bank involves the construction of a surface water management system requiring another permit defined above, check the appropriate box and submit the information requested by the applicable section.)
☐ Mitigation Bank (conceptual) - include information requested in Sections C and F.

B. Type of activity for which you are applying (check at least one)

☐ Construction or operation of a new system, other than a solid waste facility, including dredging or filling in, on or over wetlands and other surface waters.
☒ Construction, expansion or modification of a solid waste facility.
☐ Alteration or operation of an existing system which was not previously permitted by a WMD or DEP.
☐ Modification of a system previously permitted by a WMD or DEP.
Provide previous permit numbers:___

☐ Alteration of a system	☐ Extension of permit duration
☐ Abandonment of a system	☐ Construction of additional phases of a system
☐ Removal of a system	

C. Are you requesting authorization to use Sovereign Submerged Lands?
☐yes ☒no
(See Section G and Attachment 5 for more information before answering this question.)

D. For activities in, on,or over wetlands or other surface waters, check type of federal dredge and fill permit requested:
☒Individual ☐Programmatic General ☐General
☐Nationwide ☐Not Applicable

E. Are you claiming to qualify for an exemption? ☐yes ☒no
If yes, provide rule number if known. ___

PART 3: A. OWNER(S) OF LAND	B. ENTITY TO RECEIVE PERMIT (IF OTHER THAN OWNER)
Name Sarasota County	Name N/A
Title and Company Sarasota County Solid Waste Division	Title and Company
Address 4000 Knights Trail Road	Address
City, State, Zip Nokomis, FL 34275	City, State, Zip
Telephone and Fax 941 861-1571	Telephone and Fax
C. AGENT AUTHORIZED TO SECURE PERMIT	**D. CONSULTANT (IF DIFFERENT FROM AGENT)**
Name ERik J. Nelson, P.E.	Name N/A
Title and Company Sr. Engineer, GeoSyntec Consultants	Title and Company
Address 14055 Riveredge Drive, Suite 300	Address
City, State, Zip Tampa, FL 33637	City, State, Zip
Telephone and Fax 813-558-0990 Fax: 813-558-9726	Telephone and Fax

PART 4: (Please provide metric equivalent for federally funded projects):

A. Name of Project, including phase if applicable: Flexible Leachate Storage Containers, Central County Solid Waste Disposal Complex

B. Is this application for part of a multi-phase project?
☐yes ☒no

C. Total applicant-owned area contiguous to the project?
6150 ac.; N/A ha.

D. Total area served by the system: 268 ac.; N/A ha.

E. Impervious area for which a permit is sought: 1.5 ac.; N/A ha.

F. Volume of water that the system is capable of impounding:
3.7 ac. ft.; N/A m

G. What is the total area of work in, on, or over wetlands or other surface waters?
N/A ac.; N/A ha. N/A sq. ft.; N/A sq. m.

H. Total volume of material to be dredged: 0 yd; N/A m

I. Number of new boat slips proposed: 0 wet slips; 0 dry slips

PART 5:

Project location (use additional sheets if needed):
County(ies)Sarasota

Section(s) 1-4, 9-16	Township 38 South	Range 19 East
Section(s)	Township	Range
Section(s)	Township	Range

Land Grant name, if applicable: N/A

Tax Parcel Identification Number: N/A

Street AddressRoador other location:4000 Knights Trail Road,

City, Zip Code, if applicable: Nokomis, Fl 34275

PART 6: Describe in general terms the proposed project, system, or activity.

Construct and operate flexible leachate storage containers to receive and manage potentially impacted stormwater and excess leachate that may be generated at the site.

PART 7:

A. If there have been any pre-application meetings, including on-site meetings, with regulatory staff, please list the date(s), location(s), and names of key staff and project representatives.

11 July 2006, Allyson Minick, Doug Hyman

B. Please identify by number any MSSW/Wetland Resource/ERP/ACOE Permits pending, issued or denied for projects at the location, and any related enforcement actions.

Agency	Date	No.\Type of Application	Action Taken

C. Note: The following information is required for projects proposed to occur in, on or over wetlands that need a federal dredge and fill permit or an authorization to use state owned submerged lands. Please provide the names, addresses and zip codes of property owners whose property directly adjoins the project (excluding application) and/or (for proprietary authorizations) is located within a 500 ft. radius of the applicant's land. Please attach a plan view showing the owner's names and adjoining property lines. Attach additional sheets if necessary.

1. N/A	2. N/A
3. N/A	4. N/A
5. N/A	6. N/A
7. N/A	8. N/A

PART 8:

A. By signing this application form, I am applying, or I am applying on behalf of the applicant, for the permit and any proprietary authorizations identified above, according to the supporting data and other incidental information filed with this application. I am familiar with the information contained in this application and represent that such information is true, complete and accurate. I understand this is an application and not a permit, and that work prior to approval is a violation. I understand that this application and any permit issued or proprietary authorization issued pursuant thereto, does not relive me of any obligation for obtaining any other required federal, state, water management district or local permit prior to commencement of construction. I agree, or I agree on behalf of the applicant, to operate and maintain the permitted system unless the permitting agency authorizes transfer of the permit to a responsible operation entity. I understand that knowingly making any false statement or representation in this application is a violation of Section 373.430, F.S. and 18 U.S.C. Section 1001.

Erik J. Nelson, P.E.
Typed/Printed Name of Applicant (If no Agent is used) or Agent (If one is so authorized below)

Signature of Applicant/Agent — Date
Sr. Engineer, GeoSyntec Consultants
(Corporate Title if applicable)

AN AGENT MAY SIGN ABOVE ONLY IF THE APPLICANT COMPLETES THE FOLLOWING:

B. I hereby designate and authorize the agent listed above to act on my behalf, or on behalf of my corporation, as the agent in the processing of this application for the permit and/or proprietary authorization indicated above; and to furnish, on request, supplemental information in support of the application. In addition, I authorize the above-listed agent to bind me, or my corporation, to perform any requirements which may be necessary to procure the permit or authorization indicated above. I understand that knowingly making any false statement or representation in this application is a violation of Section 373.430, F.S. and 18 U.S.C. Section 1001.

Frank Coggins	Frank Coggins	2-12-2007
Typed/Printed Name of Applicant	Signature of Applicant	Date

Manager of Solid Waste Operations
(Corporate Title if applicable)

Please note: The applicant's original signature (not a copy) is required above.

PERSON AUTHORIZING ACCESS TO THE PROPERTY MUST COMPLETE THE FOLLOWING:

C. I either own the property described in this application or I have legal authority to allow access to the property, and I consent, after receiving prior notification, to any site visit on the property by agents or personnel from the Department of Environmental Protection, the Water Management District and the U.S. Army Corps of Engineers necessary for the review and inspection of the proposed project specified in this application. I authorize these agents or personnel to enter the property as many times as may be necessary to make such review and inspection. Further, I agree to provide entry to the project site for such agents or personnel to monitor permitted work if a permit is granted.

Frank Coggins	Frank Coggins	2-12-2007
Typed/Printed Name of Applicant	Signature of Applicant	Date

Manager of Solid Waste Operations
(Corporate Title if applicable)

SECTION C

NOTICE OF RECEIPT OF APPLICATION

SECTION C

Environmental Resource Permit Notice of Receipt of Application

Note: this form does not need to be submitted for noticed general permits.
This information is required in addition to that required in other sections of the application. Please submit five copies of this notice of receipt of application and all attachments with the other required information. Please submit all information on 8 1/2" x 11" paper.

Project Name	Flexible Leachate Storage Containers Central County Solid Waste Disposal Complex
County	Sarasota County, Florida
Owner	Sarasota County)
Applicant:	Sarasota County
Applicant's Address:	4000 Knights Trail Road, Nokomis, Florida 34275

1. Indicate the project boundaries on a USGS quadrangle map. Attach a location map showing the boundary of the proposed activity. The map should also contain a north arrow and a graphic scale; show Section(s), Township(s), and Range(s); and must be of sufficient detail to allow a person unfamiliar with the site to find it.

SEE FIGURE C-1

2. Provide the names of all wetlands, or other surface waters that would be dredged, filled, impounded, diverted, drained, or would receive discharge (either directly or indirectly), or would otherwise be impacted by the proposed activity, and specify if they are in an Outstanding Florida Water or Aquatic Preserve:

N/A

3. Attach a depiction (plan and section views), which clearly shows the works or other facilities proposed to be constructed. Use multiple sheets, if necessary. Use a scale sufficient to show the location and type of works.

SEE APPENDIX A - PERMIT DRAWINGS.

4. Briefly describe the proposed project (such as "construct dock with boat shelter", "replace two existing culverts", "construct surface water management system to serve 150 acre residential development"):

Install and operate Flexible Leachate Storage Containers (FLSC) to contain and manage potentially impacted storm water and excess leachate generated at the site.

5. Specify the acreage of wetlands or other surface waters, if any, that are proposed to be filled, excavated, or otherwise disturbed or impacted by the proposed activity:

filled 0 ac.; 0 excavated ac.;

other impacts 0 ac.

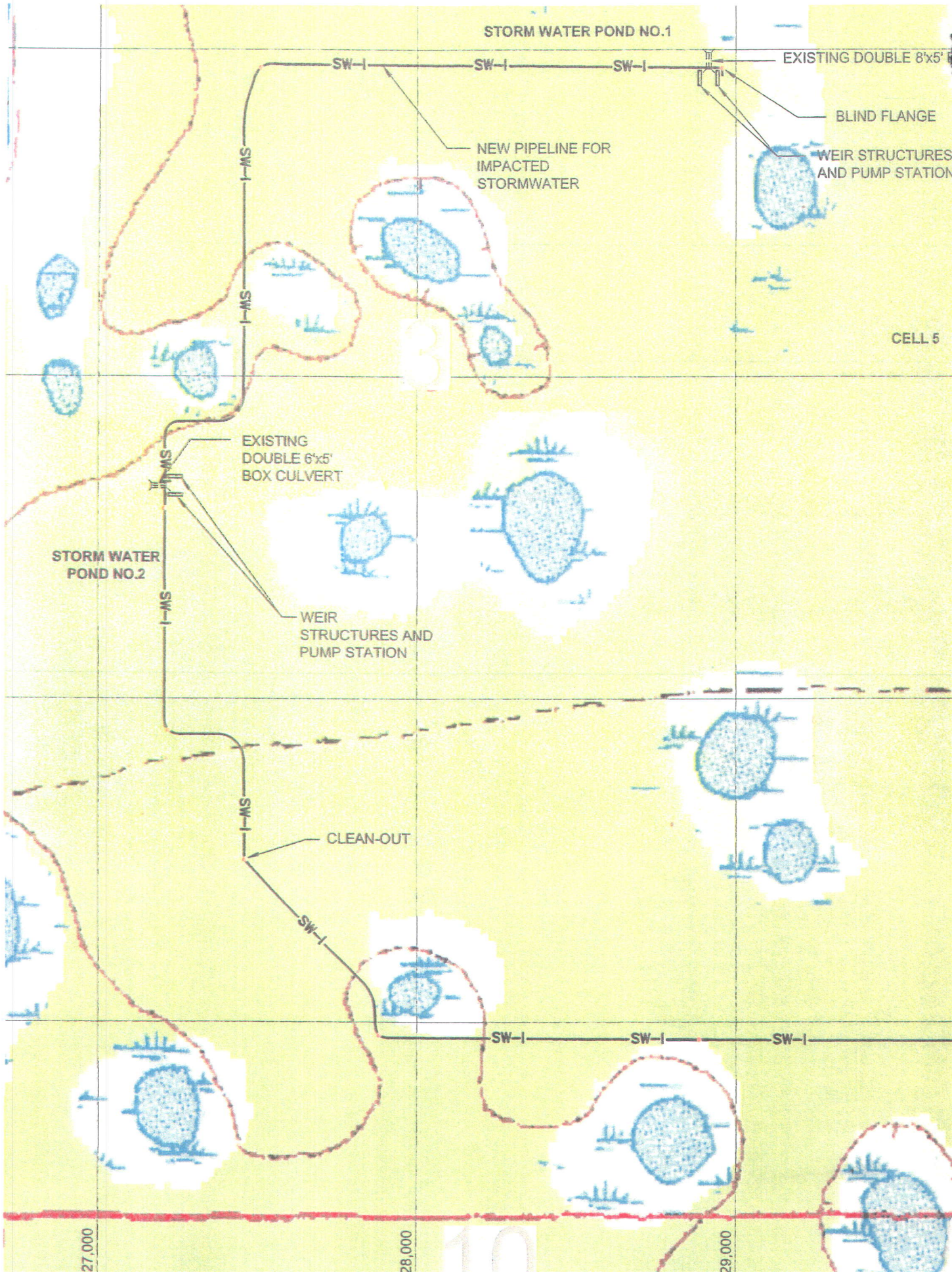
6. Provide a brief statement describing any proposed mitigation for impacts to wetlands and other surface waters (attach additional sheets if necessary):

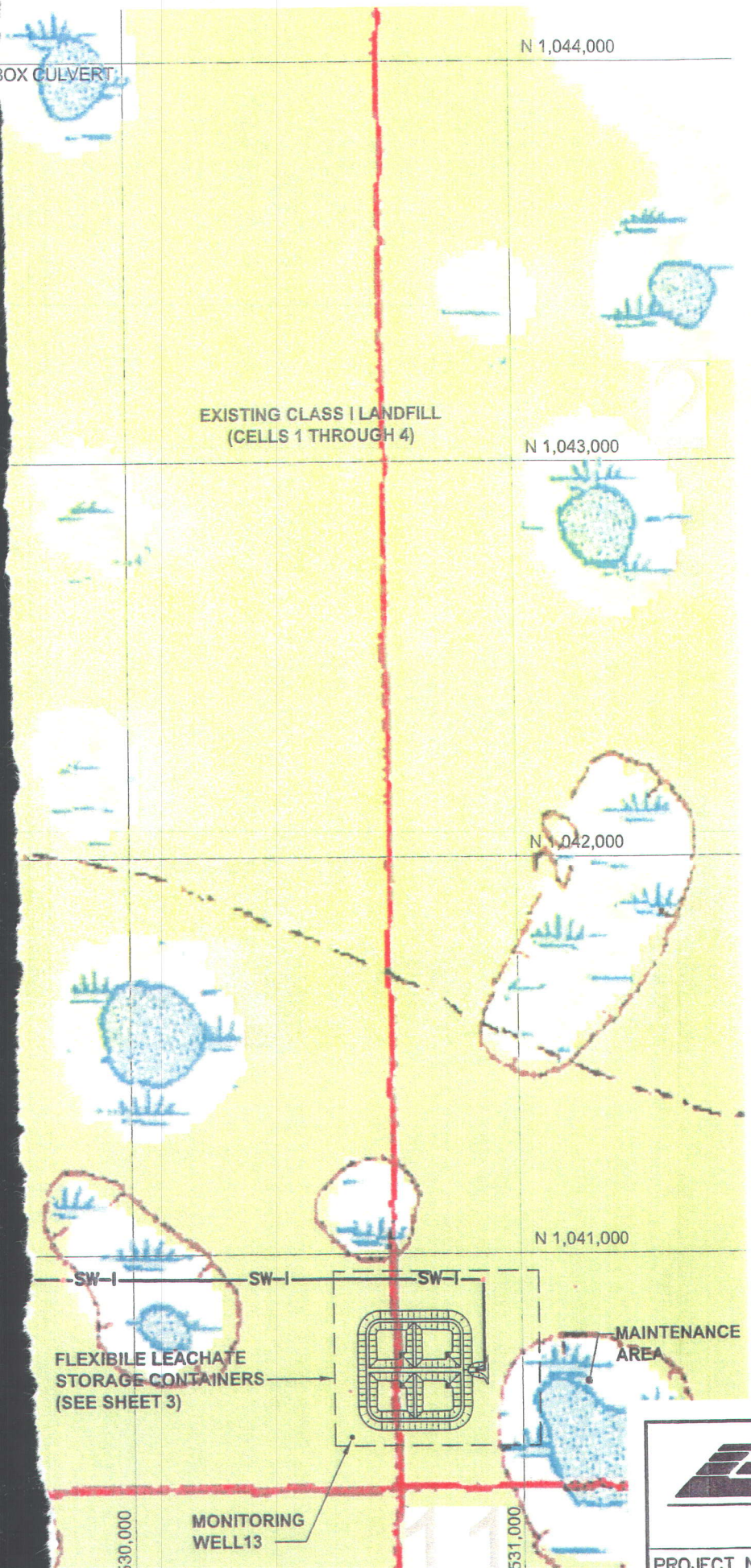
The FLSC will be constructed inside of the previously permitted and developed landfill complex, therefore no wetlands will be affected by the proposed development.

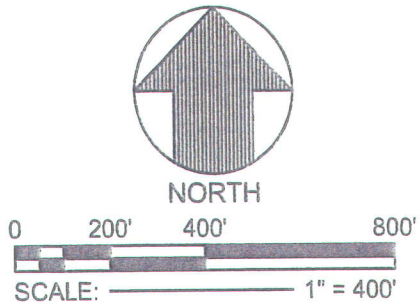
FOR AGENCY USE ONLY

Application Name:
Application Number:
Office where the application can be inspected:

Note to Notice recipient: The information in this notice has been submitted by the applicant and has not been verified by the agency. It may be incorrect, incomplete or may be subject to change.


STORM WATER POND NO.1
SW-I
SW-I
SW-I
EXISTING DOUBLE 8'x5'
BLIND FLANGE
NEW PIPELINE FOR IMPACTED STORMWATER
WEIR STRUCTURES AND PUMP STATION
SW-I
SW-I
CELL 5
EXISTING DOUBLE 6'x5' BOX CULVERT
SW-I
STORM WATER POND NO.2
SW-I
WEIR STRUCTURES AND PUMP STATION
SW-I
CLEAN-OUT
SW-I
SW-I
SW-I
SW-I
27,000
28,000
29,000





NOTES:

1. NORTHING AND EASTING COORDINATES SHOWN REFLECT FLORIDA STATE PLANE WEST ZONE NORTH AMERICAN DATUM OF 1983 (NAD83) AS ESTABLISHED BY OTHERS AND PROVIDED BY THE CLIENT.
2. BASE MAP SHOWN IS USGS 7.5' QUADRANGLE "LAUREL" OBTAINED FROM FDEP WEBSITE WWW.LABINS.ORG
3. MONITORING WELL 13 WAS INSTALLED BY NATIONAL ENVIRONMENTAL TECHNOLOGY, INC. (DOVER, FLORIDA) ON 1 AUGUST 2006.
4. SITE DEVELOPMENT IS LOCATED IN TOWNSHIP 38 SOUTH, RANGE 19 EAST.

SITE DEVELOPMENT PLAN

GEOSYNTEC CONSULTANTS
TAMPA, FLORIDA

PROJECT NO.	FL0819.02	FIGURE NO.	C-1

SECTION H

INFORMATION FOR GENERAL ENVIORNMENTAL RESOURCE PERMITS FOR MINOR SURFACE WATER SYSTEMS

SECTION H

INFORMATION FOR GENERAL ENVIRONMENTAL RESOURCE PERMITS FOR MINOR SURFACE WATER SYSTEMS

To obtain a General Permit for a Minor Surface Water Management System, the project must meet all of the requirements of Section A, Part 1 **OR** one of the requirements of Section A, Part 2 and both of the requirements of Section A, Part 3. Indicate which thresholds apply to your project and submit the information requested in Section B.

A. Project Thresholds

Part 1.

_X_ The total land area does not equal or exceed 10 acres;
_X_ The area of impervious surface will not equal or exceed two acres;
N/A Any activities to be conducted in, on or over wetlands or other surface waters will consist of less than 100 square feet of dredging or filling;
_X_ The activities will not utilize pumps for stormwater management;
_X_ The activities will not utilize storm drainage facilities larger than one 24 inch diameter pipe or its hydraulic equivalent;
_X_ Discharges from the site will meet State water quality standards, and the surface water management system will meet the applicable technical criteria for stormwater management in the Basis of Review;
N/A The proposed building floors will be above the 100-year flood elevations;
_X_ The surface water management system can be effectively operated and maintained, and;
_X_ The proposed activities will not cause significant adverse impacts to occur individually or cumulatively.

Part 2.

___ 40D-4.051(3) - NORMAL AND NECESSARY FARMING AND FORESTRY

Part 3.

___ Discharges from the site will meet State water quality standards, and the surface water management system will meet the applicable technical criteria for stormwater management in the Basis of Review described in Rule 40D-4.091(1), and

___ The Surface Water Management System can be effectively operated and maintained.

B. Technical and Legal Information

1. *Provide a copy of the boundary survey and/or a legal description and acreage of the total land area of contiguous property owned or controlled by the applicant, including the project site.*

A copy of the Property Boundary Survey has been provided as Figure G-8, originally prepared by CDM.

2. *Provide recent aerials, legible for photo interpretation with a scale of 1" = 400' or more detailed, with total land, project area and any on-site wetlands delineated.*

Sheet 2 of 13 of the Permit Drawings, Appendix A, is based on a recent aerial photograph of the site. The project location as well as the location of other features at the Central County Solid Waste Disposal Complex are shown on this sheet.

3. *Provide a detailed topographic map (with contours) of the site and adjacent hydrologically related area. The location and description of bench marks (minimum of one per major water control structure) should be included.*

Sheets 3 and 4 of 13 of the permit drawings, Appendix A, provide the most recent topography for the FLSC facility.

4. *Describe the location, size (in acres) and type of any on-site wetlands or other surface waters.*

All wetland areas on the property are outside of the permitted, developed area of the facility. No wetland impacts are proposed for the development, construction or operation of the FLSCs.

5. *Provide the project site development plan and acreage of the total area of impervious surface.*

The site development plan is shown on Sheets 2 through 4 of 13 of the Permit Drawings, Appendix A. The total area of impervious surface that will be created by the proposed construction is less than 1.5 acres. The proposed construction is such that any precipitation landing on the impervious areas will be contained within the impervious area. Collected storm water from the top of the FLSC may be released to the adjacent storm water drainage channel at a later date if necessary.

6. *Provide the Surface Water Management System design plans, calculations and reports signed and sealed by a Florida Registered Professional Engineer, as required by law.*

Calculations pertaining to the proposed construction are provided in the Solid Waste Application to Construct FLSCs at CCSWDC prepared by Geosyntec consultants (November 2006). The Solid Waste Application was submitted to FDEP in November 2006. Additional detail regarding construction is provided on the Project Drawings also prepared by Geosyntec consultants

7. *Provide construction drawings signed and sealed by the design engineer showing the location and details of the Surface Water Management System including but not limited to any preserved wetlands, lakes, culverts, pipes, under drains, exfiltration trenches, discharge structures, pumps and related facilities such as paving, grading and erosion or sediment control measures to be employed.*

Construction drawings have been provided and are attached to his document as Appendix A.

8. *Indicate type of water quality treatment system used:*

____ *Man-made wet detention* ____ *Off-line retention (Dry pond)*

____ *On-line effluent filtration (side bank or under drain filters)*

____ *Off-line underground exfiltration system* ____ *On-line retention (Dry pond)*

____ *Wet detention utilizing natural wetlands* XX *Other (explain)*

Storm water that is collected on top of the FLSC will be separated from the leachate and potentially impacted storm water contained inside of the FLSC. Storm water accumulated on top of the upper FLSC liner will be considered un-impacted water and will be pumped into the adjacent storm water management ditch at the site. Potentially impacted storm water that is suitable for release will be pumped across the roadway to the north and released back to the perimeter drainage channel around the landfill area.

9. *If a Water Use Permit has been issued for the project, state the permit number.*

N/A

10. *Indicate how any existing wells located within the project site will be utilized or abandoned.*

No supply wells will be utilized or abandoned as part of this project. One monitoring well will be installed approximately 50 feet southwest of the FLSC facility. See Section 3.4 of the Solid Waste Application.

11. *Provide a letter or other current evidence of potential acceptance by the operation and maintenance entity, if the entity is to be a public body such as a city or drainage district. If the entity is a homeowners or other association, final draft documents verifying either the present or imminent existence of such an organization and its ability to accept operation and maintenance responsibility are required.*

Sarasota County has prepared a letter indicating their acceptance responsibility for operation and maintenance of the proposed FLSC facility, Attachment H-1.

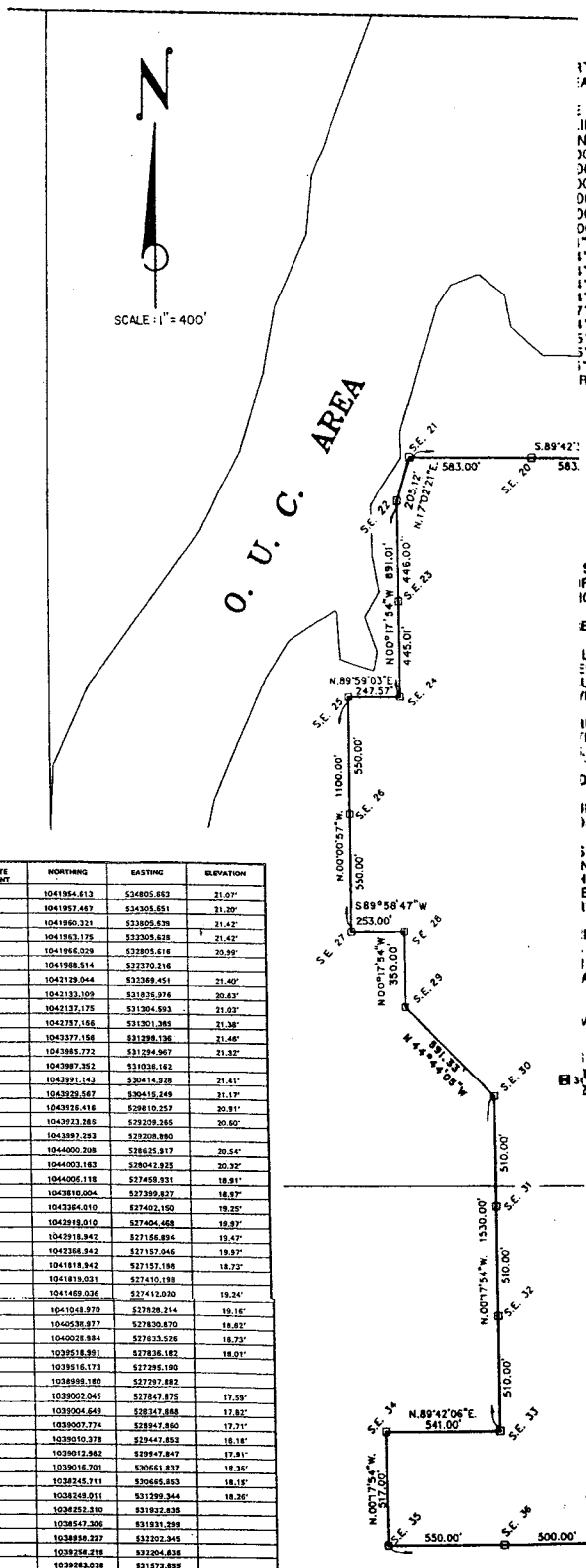
DESCRIPTION

CENTRAL COUNTY SOLID WASTE COMPLEX

(PROPOSED SPECIAL EXCEPTION AREA #1248)

ITIONS OF SECTIONS 2, 3, 10, AND 11, TOWNSHIP 38 SOUTH, :AST, SARASOTA COUNTY, FLORIDA, DESCRIBED AS FOLLOWS:

: AT THE NORTHEAST CORNER OF SAID SECTION 3; THENCE ALONG .INE OF SAID SECTION, S.00°14'54"E, 1992.01 FEET TO THE POINT NG; THENCE LEAVING SAID EAST LINE, S.89°30'28"E., 258.81 FEET;)°17'54"E., 1848.62 FEET; THENCE S.89°33'45"E., 1064.89 FEET;)0°16'24"E., 160.53 FEET; THENCE S.89°40'23"E., 2435.49 FEET;)0°50'49"E., 1100.23 FEET; THENCE N.89°40'23"W., 3567.03 FEET; 00°26'15"W., 900.00 FEET; THENCE N.89°40'23"W., 300.00 FEET;)0°26'15"W., 708.76 FEET; THENCE S.89°33'45"E., 1262.00 FEET; 00°26'15"W., 300.00 FEET; THENCE S.33°24'33"W., 492.26 FEET; I"E., 295.00 FEET; THENCE S.89°42'06"W., 1267.00 FEET; THENCE I"W., 771.00 FEET; THENCE S.89°42'06"W., 3364.00 FEET; THENCE I"W., 517.00 FEET; THENCE N.89°42'06"E., 541.00 FEET; THENCE I"W., 1530.00 FEET; THENCE N.44°44'05"W., 591.33 FEET; THENCE I"W., 350.00 FEET; THENCE S.89°58'47"W., 253.00 FEET; THENCE 7"W., 1100.00 FEET; THENCE N.89°59'03"E., 247.57 FEET; THENCE I"W., 891.01 FEET; THENCE N.17°02'21"E., 205.12 FEET; THENCE 5"E., 1748.97 FEET; THENCE S.00°17'54"E., 73.97 FEET; THENCE 5"E., 1206.00 FEET; THENCE N.00°17'54"W., 61.58 FEET; THENCE I"E., 621.25 FEET TO THE POINT OF BEGINNING, CONTAINING 549.4 RE OR LESS.



CONCRETE MONUMENT	NORTHING	EASTING	ELEVATION
S.E. 1	1041954.613	534805.663	21.07'
S.E. 2	1041957.467	534305.651	21.20'
S.E. 3	1041960.321	533805.639	21.42'
S.E. 4	1041963.175	533305.628	21.42'
S.E. 5	1041966.029	532805.616	20.99'
S.E. 6	1041968.514	532370.216	
S.E. 7	1042129.044	532369.451	21.40'
S.E. 8	1042133.109	531836.976	20.63'
S.E. 9	1042137.175	531304.593	21.03'
S.E. 10	1042757.166	531301.365	21.38'
S.E. 11	1043377.156	531298.136	21.46'
S.E. 12	1043965.772	531294.967	21.32'
S.E. 13	1043987.352	531036.162	
S.E. 14	1043991.143	530414.928	21.41'
S.E. 15	1043929.567	530415.249	21.17'
S.E. 16	1043926.416	529810.257	20.91'
S.E. 17	1043923.265	529209.265	20.60'
S.E. 18	1043997.293	529208.890	
S.E. 19	1044000.208	528625.917	20.54'
S.E. 20	1044003.163	528042.925	20.32'
S.E. 21	1044006.118	527459.931	18.91'
S.E. 22	1043810.004	527399.837	18.97'
S.E. 23	1043364.010	527402.150	19.25'
S.E. 24	1042919.010	527404.468	19.97'
S.E. 25	1042918.942	527156.894	19.47'
S.E. 26	1042366.942	527157.046	19.97'
S.E. 27	1041819.942	527157.198	18.73'
S.E. 28	1041819.031	527410.198	
S.E. 29	1041469.036	527412.030	19.24'
S.E. 30	1041048.970	527828.214	19.16'
S.E. 31	1040538.977	527830.870	18.62'
S.E. 32	1040028.984	527833.526	18.73'
S.E. 33	1039518.991	527836.182	18.01'
S.E. 34	1039516.173	527295.190	
S.E. 35	1038998.180	527297.882	
S.E. 36	1039002.045	527847.875	17.59'
S.E. 37	1039004.649	528347.868	17.82'
S.E. 38	1039007.774	528947.860	17.71'
S.E. 39	1039010.378	529447.853	18.18'
S.E. 40	1039012.982	529947.847	17.91'
S.E. 41	1039016.701	530661.837	18.36'
S.E. 42	1038245.711	530665.853	18.18'
S.E. 43	1038249.011	531299.344	18.26'
S.E. 44	1038252.310	531932.835	
S.E. 45	1038547.306	531931.299	
S.E. 46	1038850.227	532202.945	
S.E. 47	1039256.218	532204.838	
S.E. 48	1039263.028	531573.895	
S.E. 49	1039267.855	530942.673	
S.E. 50	1039978.599	530948.065	
S.E. 51	1039978.108	531034.064	
S.E. 52	1040124.862	531249.336	
S.E. 53	1040874.862	531254.953	
S.E. 54	1040872.008	531754.945	
S.E. 55	1040869.154	532254.936	
S.E. 56	1040866.300	532754.928	
S.E. 57	1040863.446	533254.920	
S.E. 58	1040860.593	533754.912	
S.E. 59	1040857.739	534254.904	
S.E. 60	1040854.502	534821.925	
S.E. 61	1041404.522	534813.795	

NOTES

SURVEY WAS PREPARED WITHOUT BENEFIT OF A CURRENT TITLE IRT AND THEREFORE MAY NOT NECESSARILY INDICATE ALL ENCUM- ICES AFFECTING SUBJECT PROPERTY.

E ARE NO VISIBLE ENCROACHMENTS EXCEPT AS SHOWN.

LOCATION AND EXISTENCE OF UNDERGROUND UTILITIES AND ERGROUND FOUNDATIONS OR ANY OTHER PERTINENT ATTRIBUTES UBJECT PROPERTY (IF ANY) NOT ABSTRACTED AS PART OF THIS /EY, EXCEPT AS SHOWN.

IINGS BASED ON GRID NORTH OF THE STATE PLANE COORDINATE 'EM FOR THE FLORIDA WEST ZONE, NAD 1983, AS ESTABLISHED BY . SURVEY.

DISTANCES ARE MEASURED IN FEET AND TENTHS OF A FOOT.

IECT TO ALL RESTRICTIONS, RIGHTS-OF-WAY AND EASEMENTS OF)RD.

E OF DOCUMENTS: THIS DOCUMENT IS AN INSTRUMENT OF SERV- N RESPECT OF THE PROJECT TO WHICH IT APPLIES AND ANY REUSE IOUT WRITTEN VERIFICATION OR ADAPTATION BY BISHOP & ASSO- ES (B&A) FOR THE SPECIFIC PURPOSES INTENDED WILL BE AT THE I'S SOLE RISK AND WITHOUT LIABILITY OR LEGAL EXPOSURE TO B&A USER SHALL INDEMNIFY AND HOLD HARMLESS B&A FROM ALL :MS, DAMAGES, LOSSES AND EXPENSES INCLUDING ATTORNEYS . ARISING OUT OF OR RESULTING THEREFROM. ANY SUCH VERIFICA- I OR ADAPTATION WILL ENTITLE B&A TO FURTHER COMPENSATION ATES TO BE AGREED UPON BY USER AND B&A.

LIMITS OF THE O.U.C. AREAS WERE TAKEN FROM INFORMATION VIDED TO THIS FIRM BY CAMP, DRESSER AND MCKEE, INC.

SURVEY PERFORMED BY BISHOP & ASSOCIATES, 78 SARASOTA TER BOULEVARD, SARASOTA, FLORIDA 34240, PHONE NO. (813) 371- 2.

LEGEND

S.E. 58 — 4" x 4" CONCRETE MONUMENT SET (NUMBERED AS SHOWN)

(C) — DENOTES CALCULATED DISTANCE, ALL OTHER DISTANCES SHOWN ARE MEASURED

25 — DENOTES AERIAL TARGET

ERTIFY: THAT THIS PLAT OF LAND SURVEY REPRESENTS A FIELD SURVEY PERFORMED ON NOVEMBER 27,1991 UNDER MY DIRECTION AND THAT IT MEETS THE MINIMUM TECHNICAL STANDARDS FOR SURVEYING IN THE STATE OF FLORIDA AS SET FORTH BY THE FLORIDA BOARD OF LAND SURVEYORS, PURSUANT TO CHAPTER 472 OF THE FLORIDA STATUTES AND CHAPTER 21HH-6 OF THE FLORIDA ADMINISTRATIVE CODE.

ASSOCIATES
BUSINESS #128)
TA CENTER BLVD.
, FLORIDA 34240

DENNIS R. HOOVER
PROFESSIONAL LAND SURVEYOR
FLORIDA CERTIFICATE #4419

RVEY NOT VALID UNLESS IMPRINTED WITH AN EMBOSSED SEAL AND INK.

DISC NO. 232

REV. NO.	DATE	DRWN	CHKD	REMARKS

CIAL EXCEPTION
BOUNDARY

PROJECT NO. 9250-58

SHEET NO. G - 8

ATTACHMENT H-1

SARASOTA COUNTY
"Dedicated to Quality Service"

Geosyntec Consultants
14055 Riveredge Drive, Suite 300
Tampa, FL 32637

RE: Operation and Maintenance Acceptance
Flexible Leachate Storage Containers
Sarasota County Central County Solid Waste Disposal Complex

To Whom It May Concern:

This letter is to serve as an acceptance of the responsibility for the operation and maintenance of the two compartment Flexible Leachate Storage Container (FLSC) as well as all related piping and components.

The FLSC system is to provide backup temporary storage to our existing leachate management program. All manufacture procedures and conditions will be incorporated into our scheduled monitoring/maintenance program.

Sincerely,

P A Wingler

Paul A. Wingler, PE
Project Manager

C

APPENDIX A

PERMIT DRAWINGS

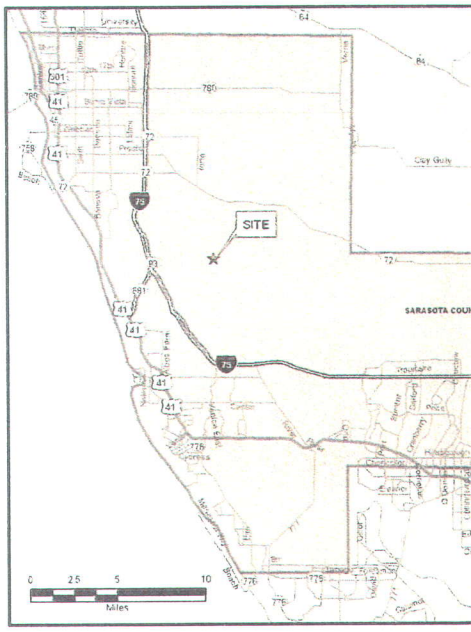
Sarasota

SOLID WASTE

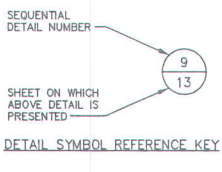
FLEXIBLE LEACHATE S
CENTRAL COUNTY SOLID W
SARASOTA CO

PERMIT DRAWINGS

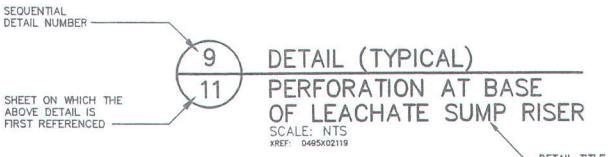
STATE MAP



COUNTY MAP

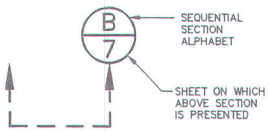


DETAIL SYMBOL REFERENCE KEY



DETAIL BAR REFERENCE KEY

EXAMPLE: DETAIL 9 WHICH IS PRESENTED ON SHEET 13 WAS FIRST REFERENCED ON SHEET 11.



SECTION SYMBOL REFERENCE KEY

SECTION BAR REFERENCE KEY

EXAMPLE: SECTION B WHICH IS PRESENTED ON SHEET 7 WAS FIRST REFERENCED ON SHEET 3.

NOTE: BLANK INDICATED FOR SHEET NUMBER IMPLIES THAT THE SECTION OR DETAIL IS NOT REFERENCED ON ANY SHEET.

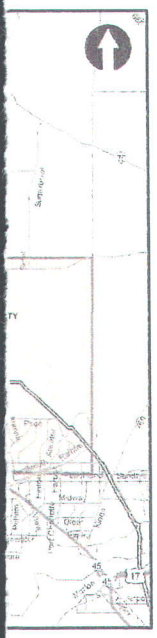
SHEET NO
1
2
3
4
5
6
7
8
9
10
11
12

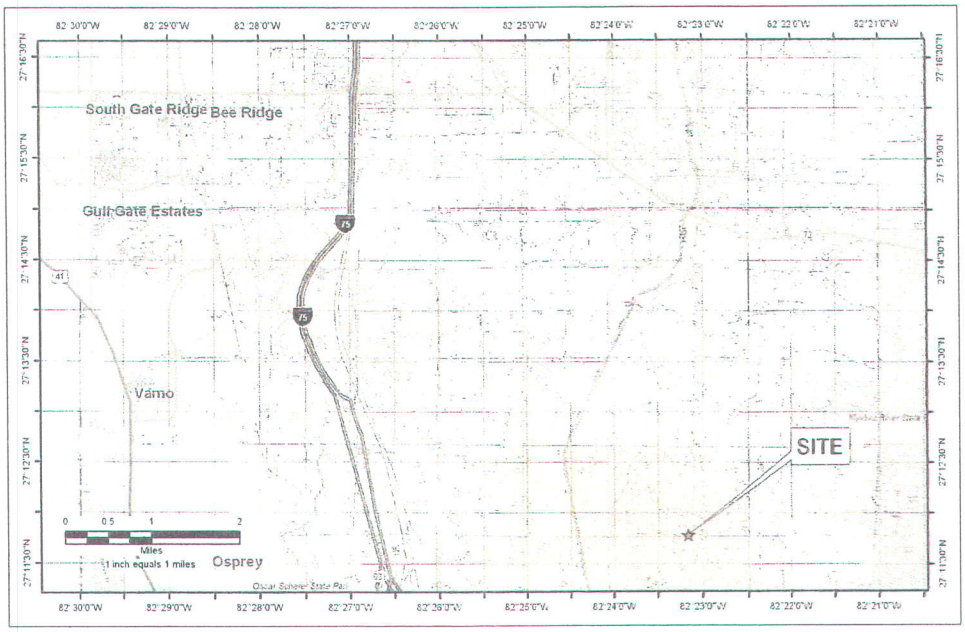
a County

: OPERATIONS

STORAGE CONTAINERS
VASTE DISPOSAL COMPLEX
UNTY, FLORIDA

S (NOVEMBER 2006)





VICINITY MAP

LIST OF SHEETS

SHEET TITLE	REVISION
Title Sheet	0
Site Development Plan	0
Base Grading Plan	0
Final Grading Plan	0
Typical Cross Sections	0
Liner System Details I	0
Liner System Details II	0
FLSC Piping Layout	0
Leachate Management System Process Diagram Legend Sheet	0
Leachate Management System Mechanical Flow Schematic	0
Leachate Management System Process and Instrumentation Schematic	0
Miscellaneous Details	0

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
TITLE SHEET

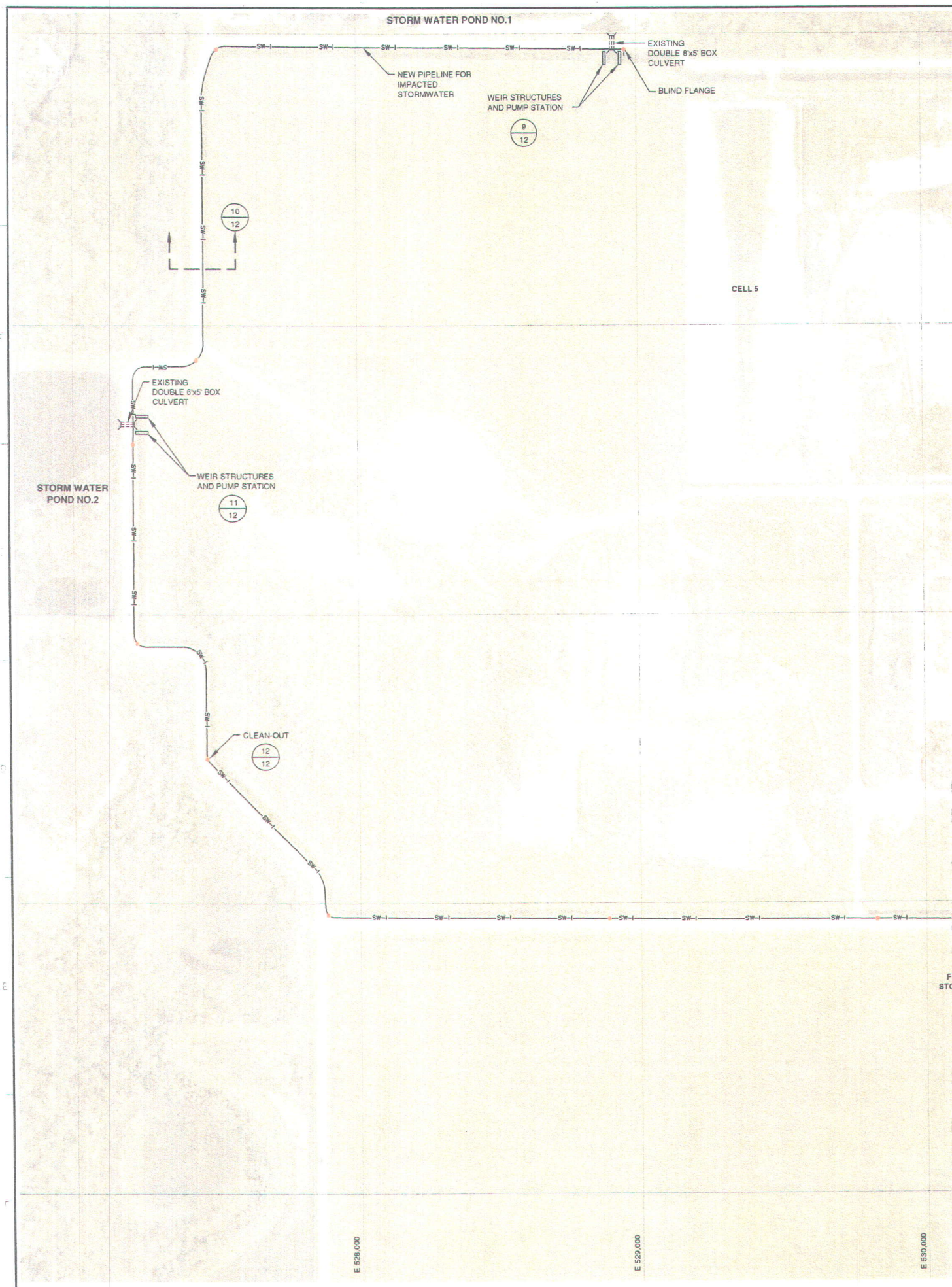
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

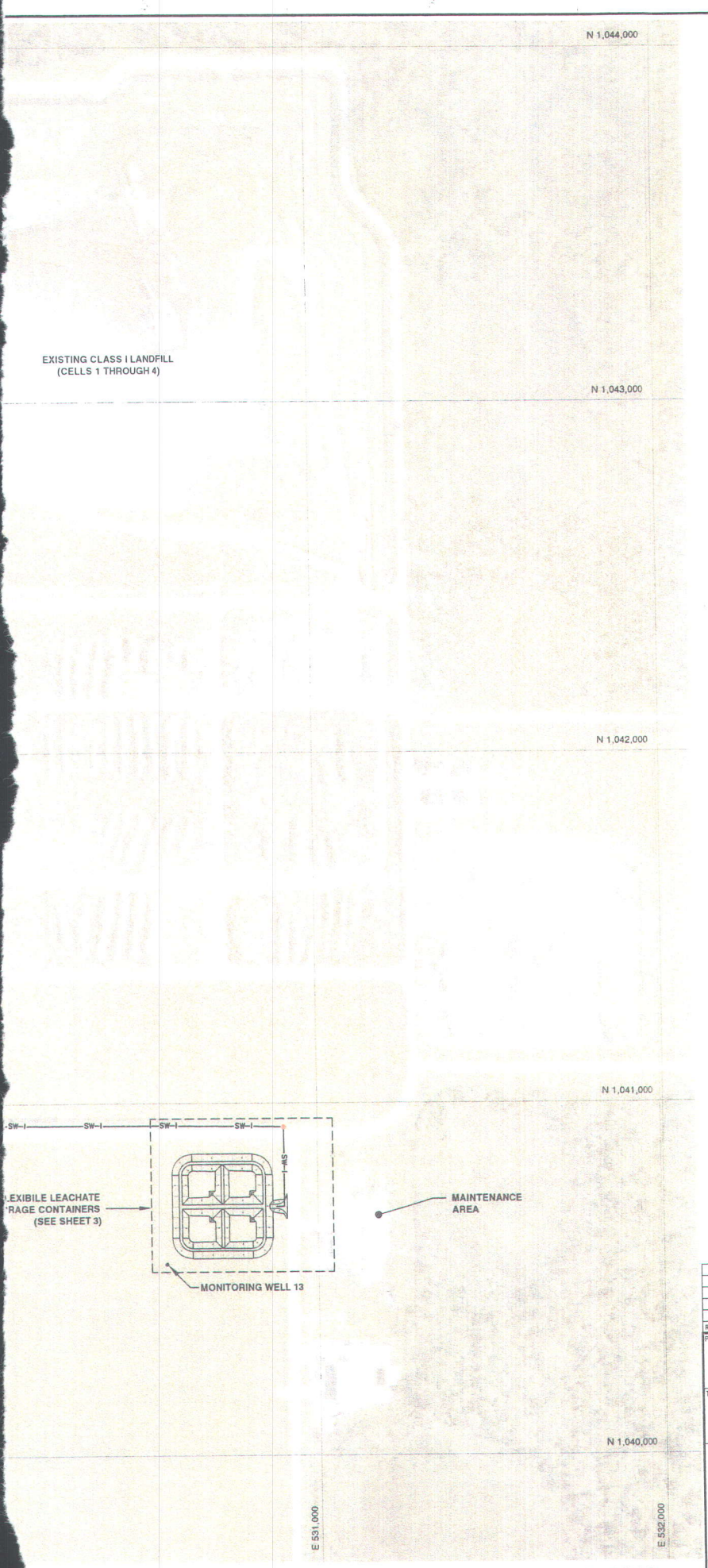
Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

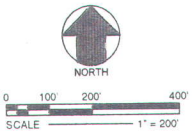
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: NA
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01001
PDF DOC NO.: GEAG-06-38
SHEET NO.: 1 OF 13

PERMIT DRAWING


STORM WATER POND NO.1
EXISTING DOUBLE 8'x5' BOX CULVERT
NEW PIPELINE FOR IMPACTED STORMWATER
WEIR STRUCTURES AND PUMP STATION
BLIND FLANGE
9
12
10
12
CELL 5
EXISTING DOUBLE 6'x5' BOX CULVERT
WEIR STRUCTURES AND PUMP STATION
STORM WATER POND NO.2
11
12
CLEAN-OUT
12
12
E 528,000
E 529,000
E 530,000





LEGEND

SW-I	IMPATECTED STORMWATER INFLOW PIPELINE
	CLEANOUT LOCATION

NOTES:

1. NORTHING AND EASTING COORDINATES SHOWN REFLECT FLORIDA STATE PLANE WEST ZONE NORTH AMERICAN DATUM OF 1983 (NAD83) AS ESTABLISHED BY OTHERS AND PROVIDED BY THE CLIENT
2. THE AERIAL PHOTOGRAPH SHOWN IS BASED ON AN AERIAL SURVEY PERFORMED ON 6 JULY 2005 BY LANDAIR MAPPING, INC., PEACHTREE CITY, GEORGIA.
3. MONITORING WELL 13 WAS INSTALLED BY NATIONAL ENVIRONMENTAL TECHNOLOGY, INC. (DOVER, FLORIDA) ON 1 AUGUST 2006

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
SITE DEVELOPMENT PLAN

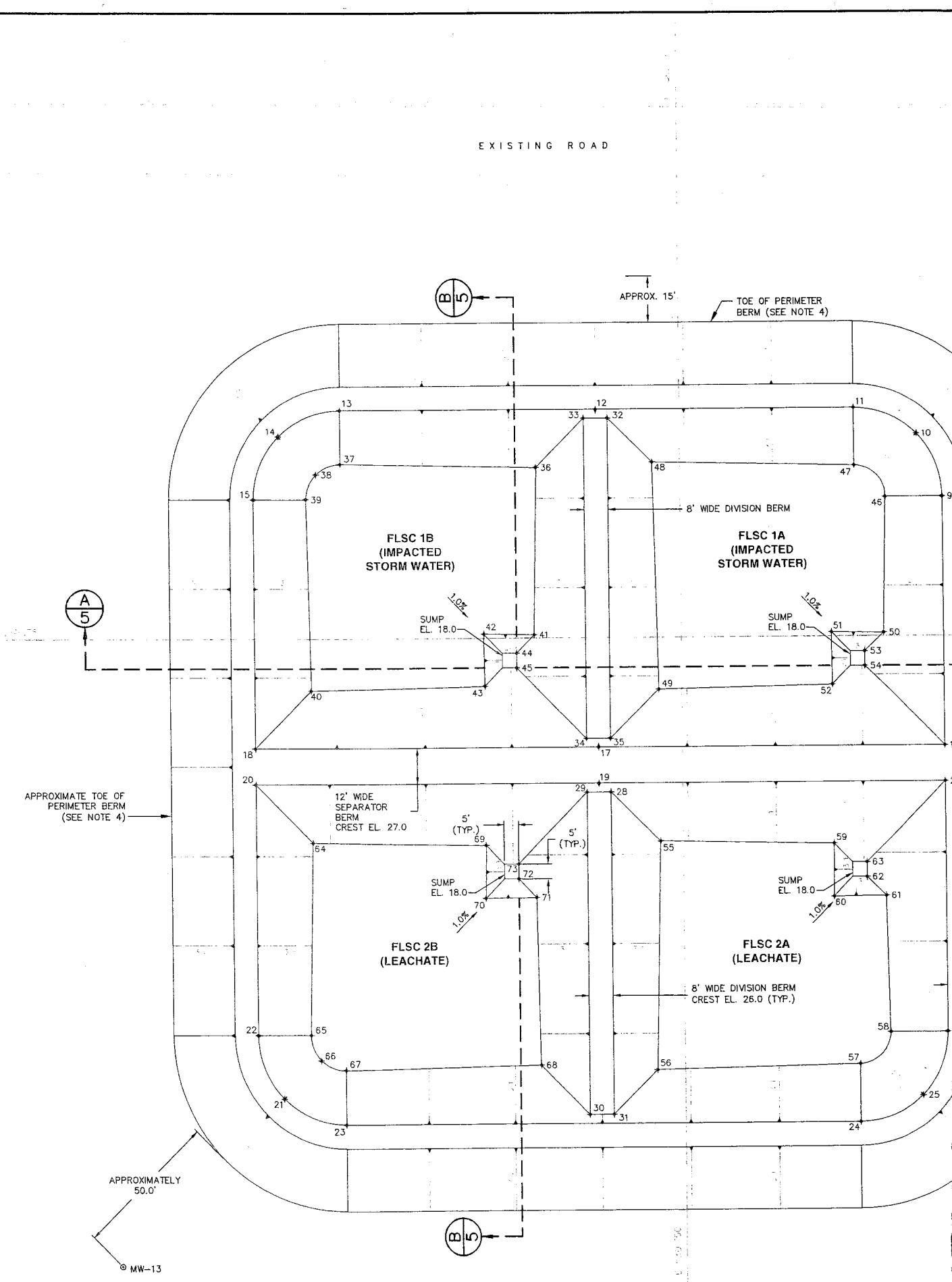
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

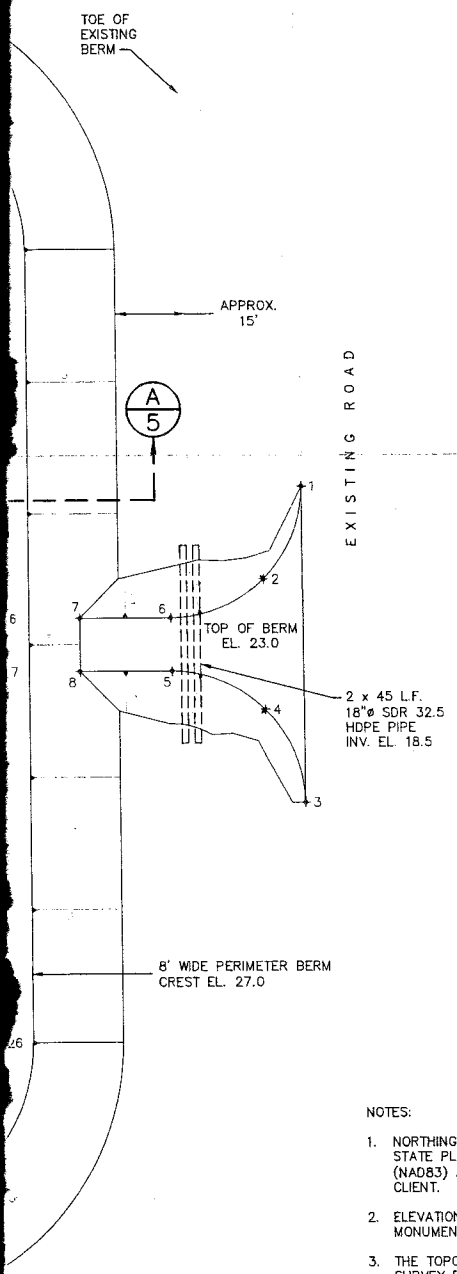
Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

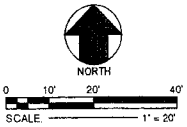
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: 1" = 200'
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01002
PDF DOC NO.: GEAG-06-38
SHEET NO.: 2 OF 13

PERMIT DRAWING


EXISTING ROAD
APPROX. 15'
TOE OF PERIMETER BERM (SEE NOTE 4)
B 5
A 5
FLSC 1B (IMPACTED STORM WATER)
FLSC 1A (IMPACTED STORM WATER)
8' WIDE DIVISION BERM
SUMP EL. 18.0
1.0%
12' WIDE SEPARATOR BERM CREST EL. 27.0
5' (TYP.)
FLSC 2B (LEACHATE)
FLSC 2A (LEACHATE)
8' WIDE DIVISION BERM CREST EL. 26.0 (TYP.)
APPROXIMATE TOE OF PERIMETER BERM (SEE NOTE 4)
APPROXIMATELY 50.0'
MW-13





LEGEND

+10 CONSTRUCTION CONTROL POINT

⊙ MONITORING WELL LOCATION

Construction Control Points

Point#	Northing	Easting	Elev.	Description
1	1,040,742.98	530,910.74	20.00	FLSC BASE
2	1,040,721.70	530,901.99	23.00	FLSC BASE
3	1,040,670.98	530,910.96	23.00	FLSC BASE
4	1,040,692.13	530,902.14	23.00	FLSC BASE
5	1,040,700.89	530,880.96	23.00	FLSC BASE
6	1,040,712.89	530,880.74	23.00	FLSC BASE
7	1,040,712.89	530,859.70	23.00	FLSC BASE
8	1,040,700.89	530,859.70	23.00	FLSC BASE
9	1,040,796.89	530,839.70	27.00	FLSC BASE
10	1,040,818.10	530,830.91	27.00	FLSC BASE
11	1,040,826.89	530,809.70	27.00	FLSC BASE
12	1,040,826.89	530,721.70	27.00	FLSC BASE
13	1,040,826.89	530,633.70	27.00	FLSC BASE
14	1,040,818.10	530,612.49	27.00	FLSC BASE
15	1,040,796.89	530,603.70	27.00	FLSC BASE
16	1,040,712.89	530,839.70	27.00	FLSC BASE
17	1,040,712.89	530,721.70	27.00	FLSC BASE
18	1,040,712.89	530,603.70	27.00	FLSC BASE
19	1,040,700.89	530,721.70	27.00	FLSC BASE
20	1,040,700.89	530,603.70	27.00	FLSC BASE
21	1,040,595.67	530,612.49	27.00	FLSC BASE
22	1,040,616.89	530,603.70	27.00	FLSC BASE
23	1,040,586.89	530,633.70	27.00	FLSC BASE
24	1,040,586.89	530,809.70	27.00	FLSC BASE
25	1,040,595.67	530,830.91	27.00	FLSC BASE
26	1,040,616.89	530,839.70	27.00	FLSC BASE
27	1,040,700.89	530,839.70	27.00	FLSC BASE
28	1,040,697.89	530,726.70	26.00	FLSC BASE
29	1,040,697.89	530,717.70	26.00	FLSC BASE
30	1,040,589.89	530,717.70	25.00	FLSC BASE
31	1,040,589.89	530,726.70	26.00	FLSC BASE
32	1,040,823.89	530,726.70	26.00	FLSC BASE
33	1,040,823.89	530,717.70	25.00	FLSC BASE
34	1,040,715.89	530,717.70	26.00	FLSC BASE
35	1,040,715.89	530,726.70	26.00	FLSC BASE
36	1,040,807.35	530,701.16	18.47	FLSC BASE
37	1,040,808.68	530,633.76	19.93	FLSC BASE
38	1,040,805.21	530,625.38	18.96	FLSC BASE
39	1,040,796.82	530,621.90	18.49	FLSC BASE
40	1,040,732.36	530,623.17	18.49	FLSC BASE
41	1,040,751.12	530,700.00	19.08	FLSC BASE
42	1,040,751.31	530,682.21	18.16	FLSC BASE
43	1,040,733.59	530,682.40	18.08	FLSC BASE
44	1,040,744.89	530,693.70	18.00	FLSC BASE
45	1,040,739.89	530,693.70	18.00	FLSC BASE
46	1,040,796.82	530,819.95	18.42	FLSC BASE
47	1,040,807.45	530,809.76	18.52	FLSC BASE
48	1,040,808.94	530,740.64	19.00	FLSC BASE
49	1,040,732.36	530,742.30	18.49	FLSC BASE
50	1,040,751.12	530,819.00	18.08	FLSC BASE
51	1,040,751.31	530,801.21	18.16	FLSC BASE
52	1,040,733.59	530,801.40	18.08	FLSC BASE
53	1,040,744.89	530,812.70	18.00	FLSC BASE
54	1,040,739.89	530,812.70	18.00	FLSC BASE
55	1,040,681.29	530,742.30	18.49	FLSC BASE
56	1,040,604.83	530,740.64	19.00	FLSC BASE
57	1,040,606.33	530,809.76	18.52	FLSC BASE
58	1,040,616.95	530,819.95	19.42	FLSC BASE
59	1,040,680.18	530,801.47	18.08	FLSC BASE
60	1,040,662.40	530,801.28	18.16	FLSC BASE
61	1,040,662.65	530,819.00	18.08	FLSC BASE
62	1,040,668.89	530,812.70	18.00	FLSC BASE
63	1,040,673.89	530,812.70	18.00	FLSC BASE
64	1,040,681.35	530,623.30	18.49	FLSC BASE
65	1,040,616.95	530,621.90	18.49	FLSC BASE
66	1,040,606.56	530,625.38	18.96	FLSC BASE
67	1,040,605.09	530,633.76	18.93	FLSC BASE
68	1,040,606.55	530,701.04	18.47	FLSC BASE
69	1,040,680.18	530,662.47	18.08	FLSC BASE
70	1,040,662.46	530,662.21	18.16	FLSC BASE
71	1,040,662.65	530,700.00	18.08	FLSC BASE
72	1,040,668.89	530,693.70	18.00	FLSC BASE
73	1,040,673.89	530,693.70	18.00	FLSC BASE

NOTES:

1. NORTHING AND EASTING COORDINATES SHOWN REFLECT FLORIDA STATE PLANE WEST ZONE NORTH AMERICAN DATUM OF 1983 (NAD83) AS ESTABLISHED BY OTHERS AND PROVIDED BY THE CLIENT.
2. ELEVATIONS ARE IN FEET AND BASED ON VERTICAL CONTROL MONUMENTS PROVIDED BY THE CLIENT.
3. THE TOPOGRAPHIC INFORMATION SHOWN IS BASED ON AN AERIAL SURVEY PERFORMED ON 6 JULY 2005 BY LANDAIR MAPPING, INC., PEACHTREE CITY, GEORGIA.
4. THE FLSC ELEVATIONS INDICATED ARE BASED ON AVAILABLE TOPOGRAPHIC INFORMATION. IF DIFFERENT FIELD CONDITIONS ARE ENCOUNTERED, THE ENGINEER SHALL MAKE NECESSARY ADJUSTMENTS IN THE FIELD.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDO	JWO	AG	JDO	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
BASE GRADING PLAN

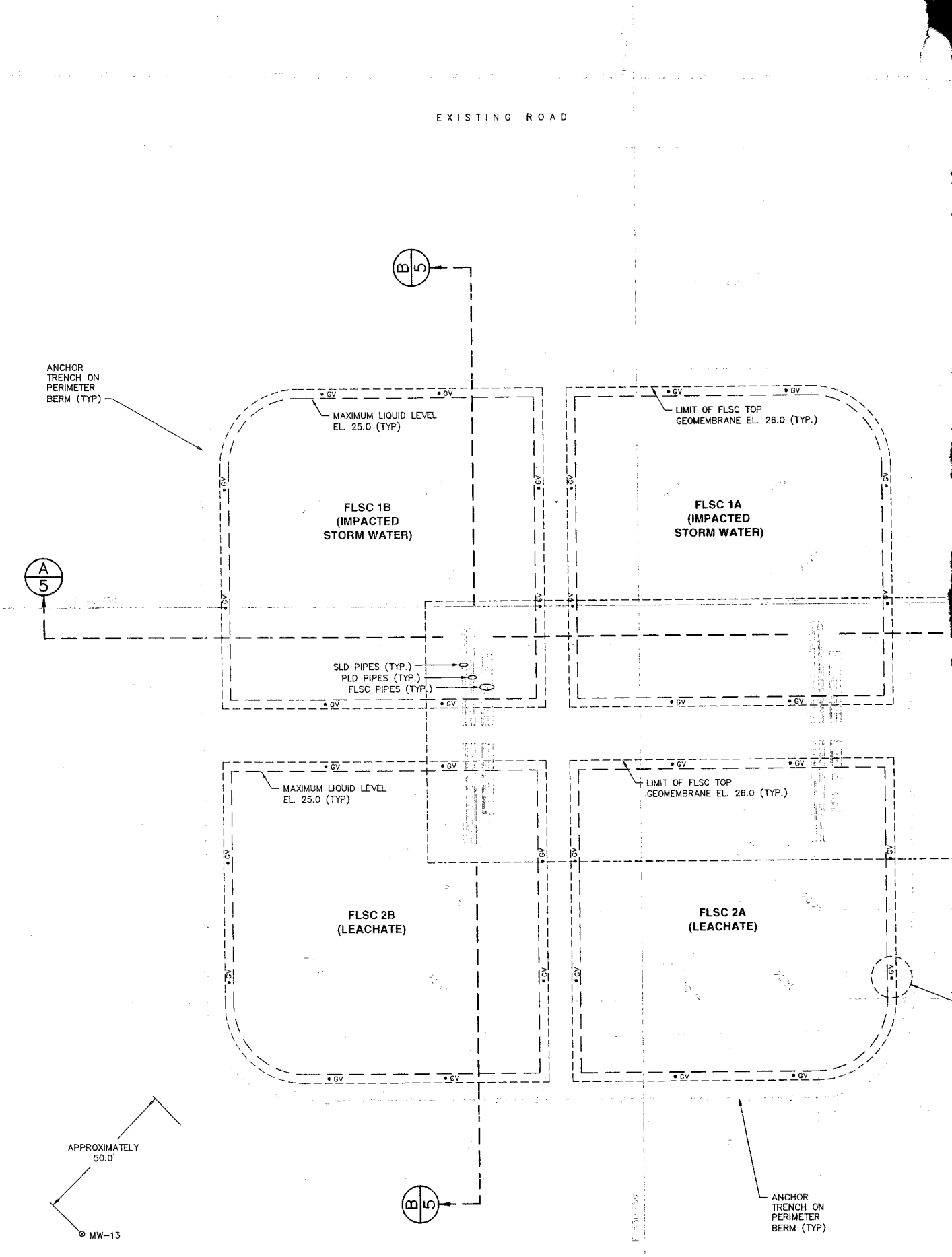
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

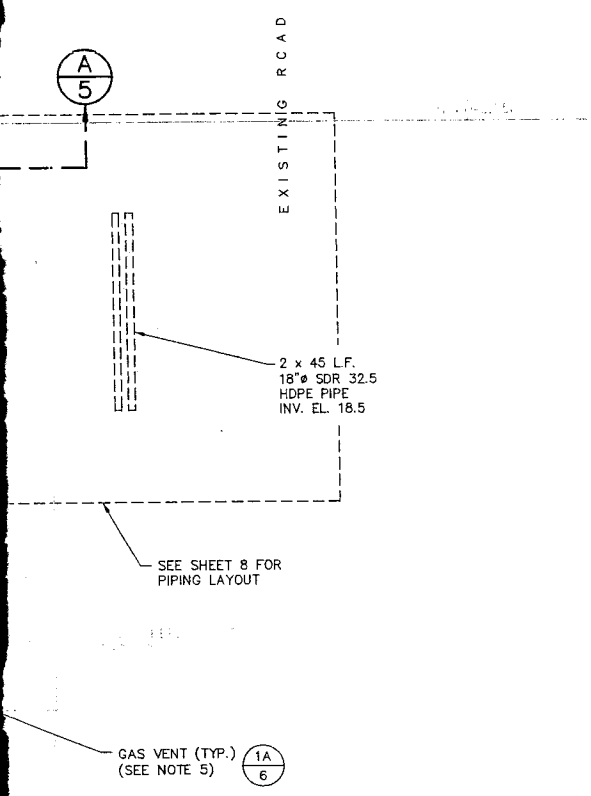
Consultant/Engineer:
GEOSYNTEC CONSULTANTS
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

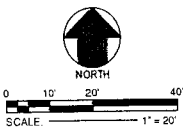
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: 1" = 20'
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01003
PDF DOC NO.: GEAG-06-38
SHEET NO.: 3 OF 13

PERMIT DRAWING


EXISTING ROAD
B 5
ANCHOR TRENCH ON PERIMETER BERM (TYP)
MAXIMUM LIQUID LEVEL EL. 25.0 (TYP)
LIMIT OF FLSC TOP GEOMEMBRANE EL. 26.0 (TYP.)
FLSC 1B (IMPACTED STORM WATER)
FLSC 1A (IMPACTED STORM WATER)
A 5
SLD PIPES (TYP.)
PLD PIPES (TYP.)
FLSC PIPES (TYP.)
GV
MAXIMUM LIQUID LEVEL EL. 25.0 (TYP)
LIMIT OF FLSC TOP GEOMEMBRANE EL. 26.0 (TYP.)
FLSC 2B (LEACHATE)
FLSC 2A (LEACHATE)
APPROXIMATELY 50.0'
MW-13
B 5
ANCHOR TRENCH ON PERIMETER BERM (TYP)





LEGEND

	ANCHOR TRENCH
• GV	GEOMEMBRANE GAS VENT (SEE NOTE 5)
⊙	MONITORING WELL LOCATION

NOTES:

1. NORTHING AND EASTING COORDINATES SHOWN REFLECT FLORIDA STATE PLANE WEST ZONE NORTH AMERICAN DATUM OF 1983 (NAD83) AS ESTABLISHED BY OTHERS AND PROVIDED BY THE CLIENT.
2. ELEVATIONS ARE IN FEET AND BASED ON VERTICAL CONTROL MONUMENTS PROVIDED BY THE CLIENT.
3. THE TOPOGRAPHIC INFORMATION SHOWN IS BASED ON AN AERIAL SURVEY PERFORMED ON 6 JULY 2005 BY LANDAIR MAPPING, INC., PEACHTREE CITY, GEORGIA.
4. THE FLSC ELEVATIONS INDICATED ARE BASED ON AVAILABLE TOPOGRAPHIC INFORMATION. IF DIFFERENT FIELD CONDITIONS ARE ENCOUNTERED, THE ENGINEER SHALL MAKE NECESSARY ADJUSTMENTS IN THE FIELD.
5. GEOMEMBRANE GAS VENTS SHALL BE INSTALLED ALONG THE PERIMETER OF THE FLSC TOP GEOMEMBRANE AT A FREQUENCY OF TWO PER SIDE. THE VENT SHALL BE CONSTRUCTED BY CUTTING A 6"ø HOLE IN THE FLSC TOP GEOMEMBRANE AND THEN WELDING THE TOP AND SIDES OF A 1' x 1' PATCH OVER THE HOLE, LEAVING THE BOTTOM EDGE OF THE GEOMEMBRANE PATCH UNWELDED TO ALLOW FOR GASES TO ESCAPE.
6. THIS SHEET ONLY SHOWS PARTIAL PIPING LAYOUT. SEE SHEET 8 FOR DETAILED PIPING LAYOUT.
7. FLSC = FLEXIBLE LEACHATE STORAGE CONTAINER, PLD = PRIMARY LEAK DETECTION, AND SLD = SECONDARY LEAK DETECTION.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
FINAL GRADING PLAN

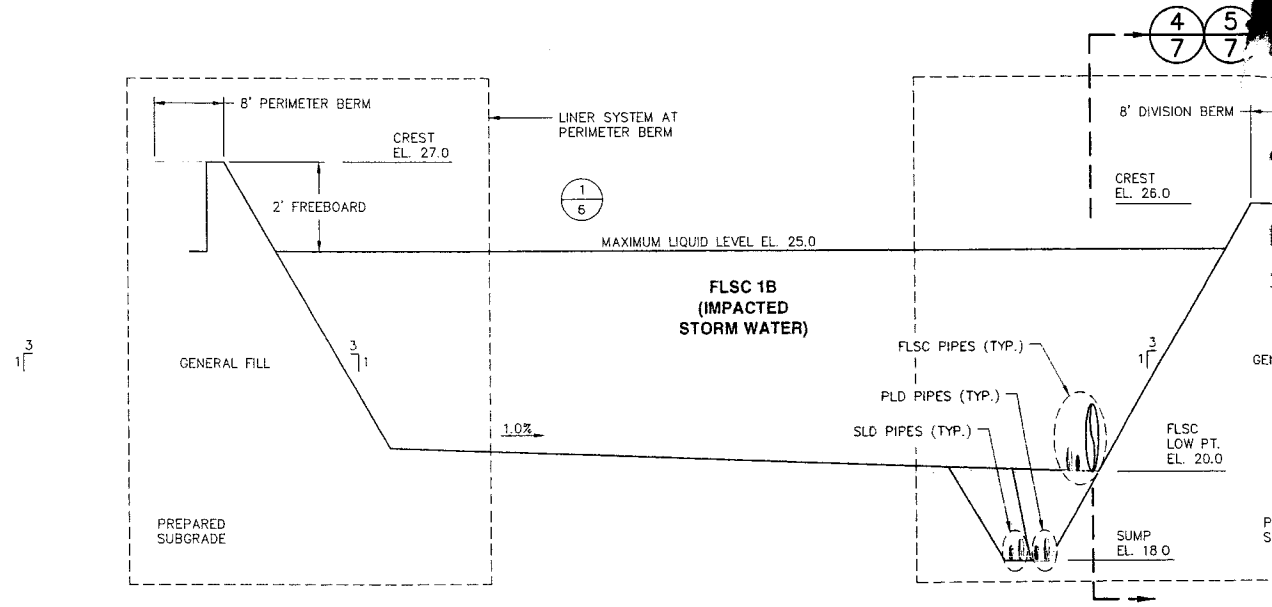
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

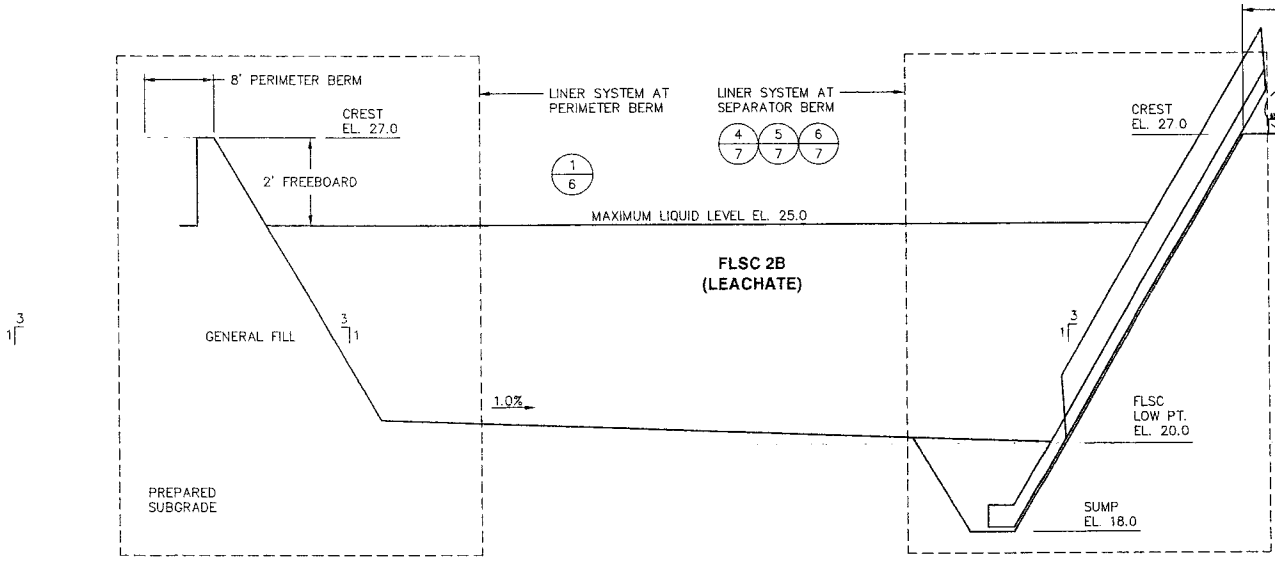
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE:	NOVEMBER 2006
SCALE:	1" = 20'
PROJECT NO.:	FL0819.02
FILE NO.:	FL0819-01004
PDF DOC NO.:	GEAG-06-38
SHEET NO.:	4 OF 13

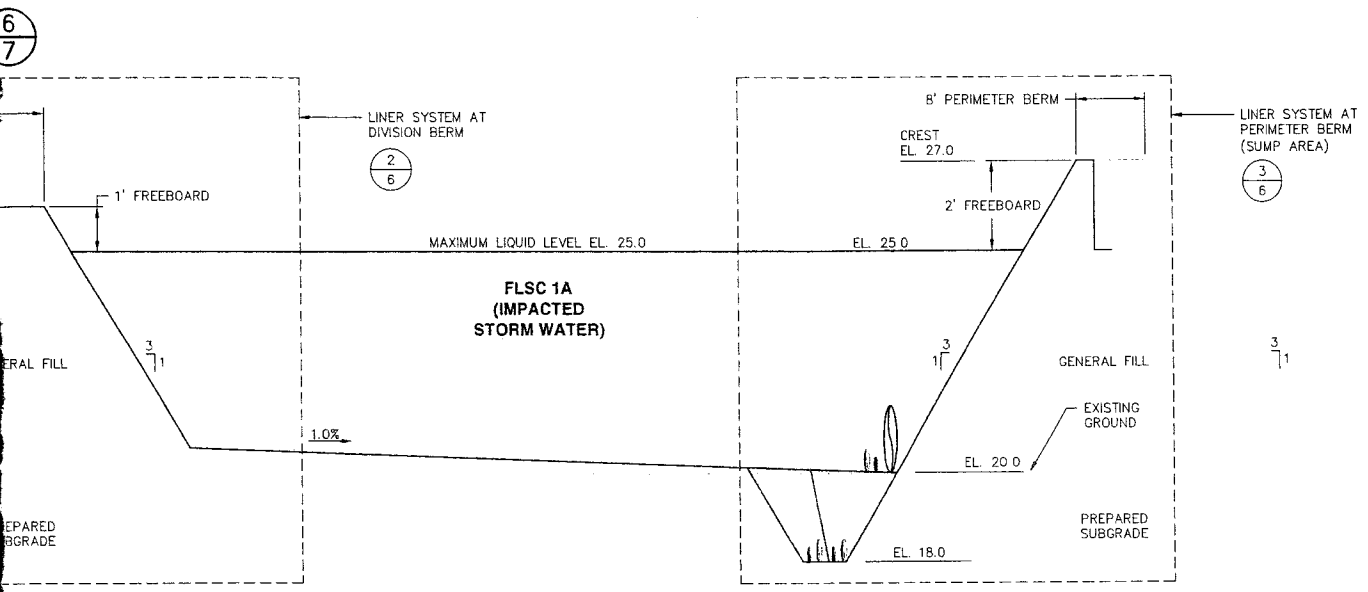
PERMIT DRAWING



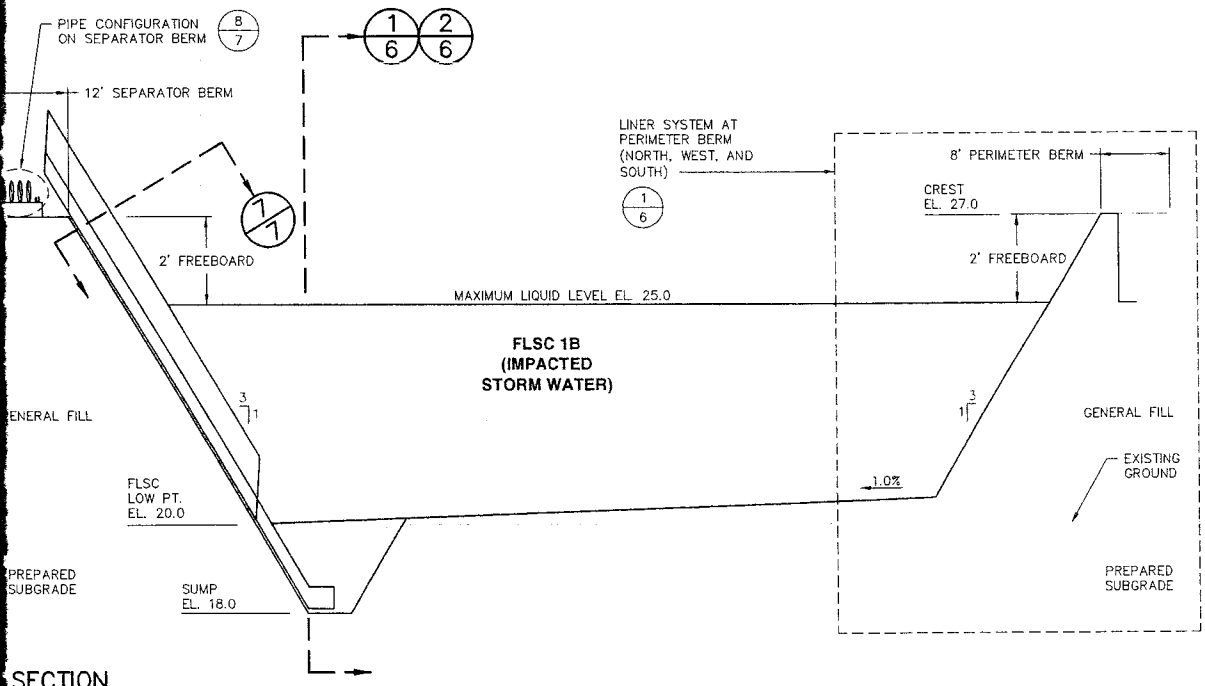
A/4 TYPICAL S
FLEXIBLE
SCALE: 1" = 10'



B/4 TYPICAL
FLEXIBL
SCALE: 1" =



ECTION
LEACHATE STORAGE CONTAINERS
HOR., 1" = 2' VER



SECTION
LEACHATE STORAGE CONTAINERS
0' HOR., 1" = 2' VER.

NOTES:

1. NORTHING AND EASTING COORDINATES SHOWN REFLECT FLORIDA STATE PLANE WEST ZONE NORTH AMERICAN DATUM OF 1983 (NAD83) AS ESTABLISHED BY OTHERS AND PROVIDED BY THE CLIENT.
2. ELEVATIONS ARE IN FEET AND BASED ON VERTICAL CONTROL MONUMENTS PROVIDED BY THE CLIENT.
3. THE TOPOGRAPHIC INFORMATION SHOWN IS BASED ON AN AERIAL SURVEY PERFORMED ON 6 JULY 2005 BY LANDAIR MAPPING, INC., PEACHTREE CITY, GEORGIA.
4. FLSC = FLEXIBLE LEACHATE STORAGE CONTAINER, PLD = PRIMARY LEAK DETECTION, AND SLD = SECONDARY LEAK DETECTION.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT: FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE: TYPICAL CROSS SECTIONS

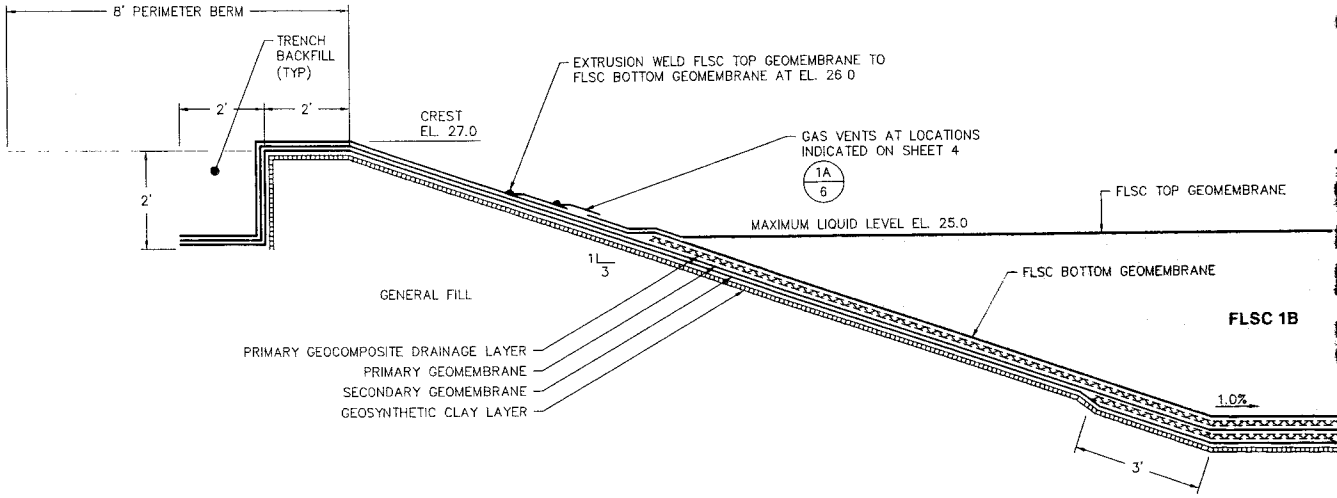
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

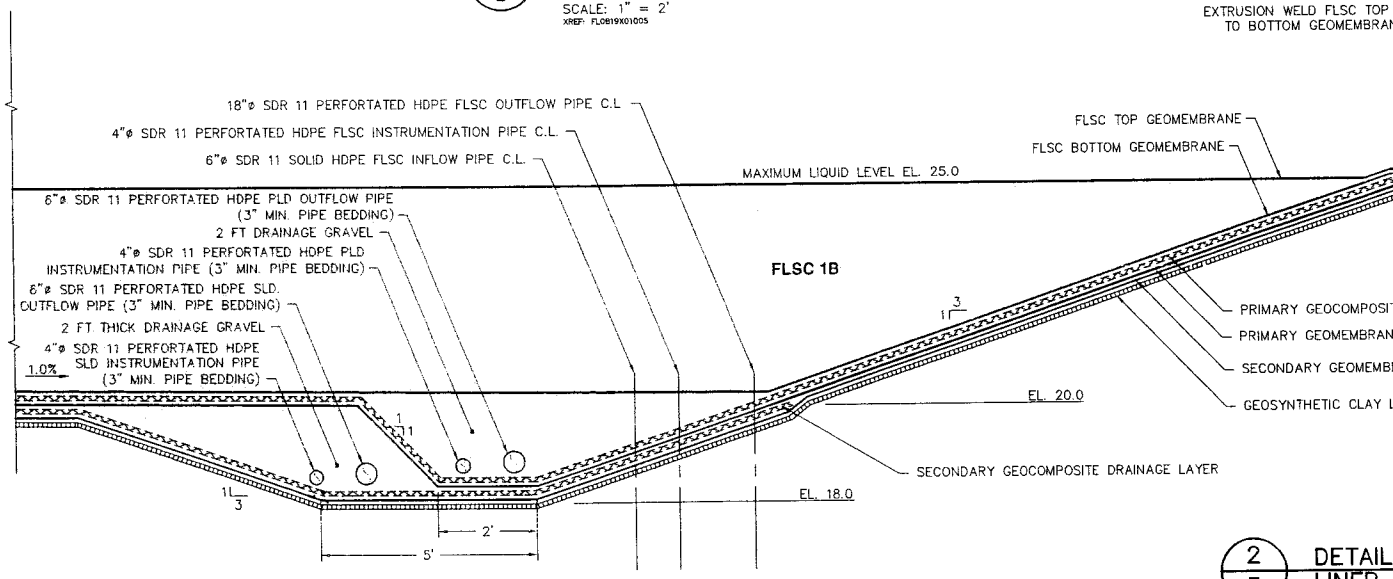
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: 1" = 10'
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01005
PDF DOC NO.: GEAG-06-38
SHEET NO.: 5 OF 13

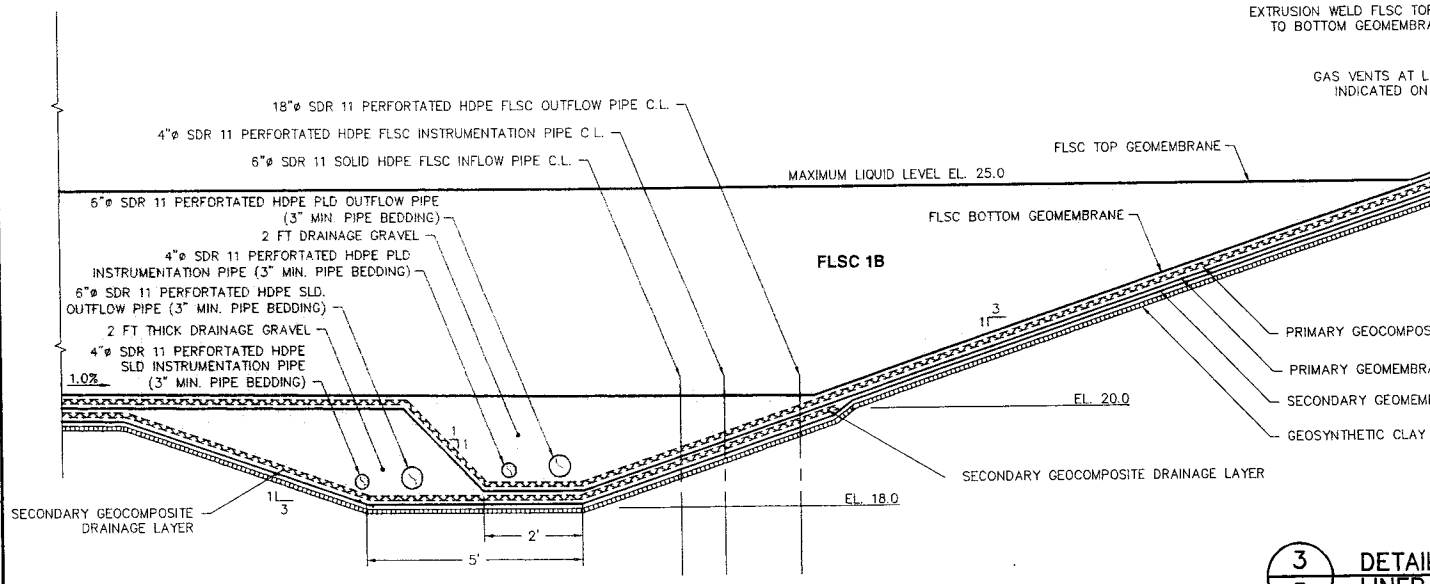
PERMIT DRAWING



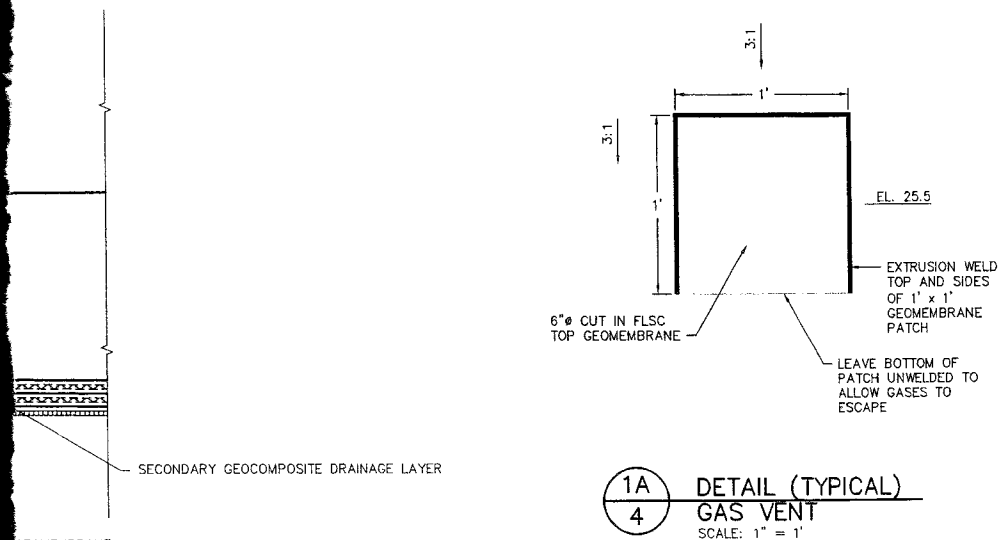
1/5 DETAIL (TYPICAL)
LINER SYSTEM AT PERIMETER BERM
SCALE: 1" = 2'
XREF: FL0819X01005



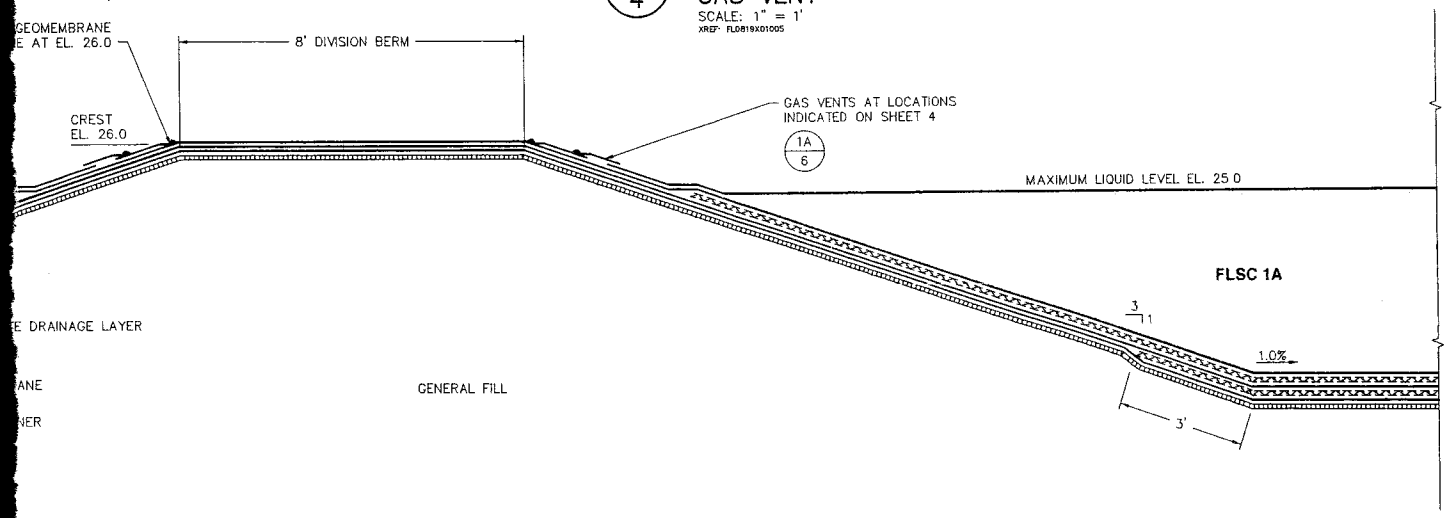
2/5 DETAIL
LINER
SCALE: 1"
XREF: FL0819X010



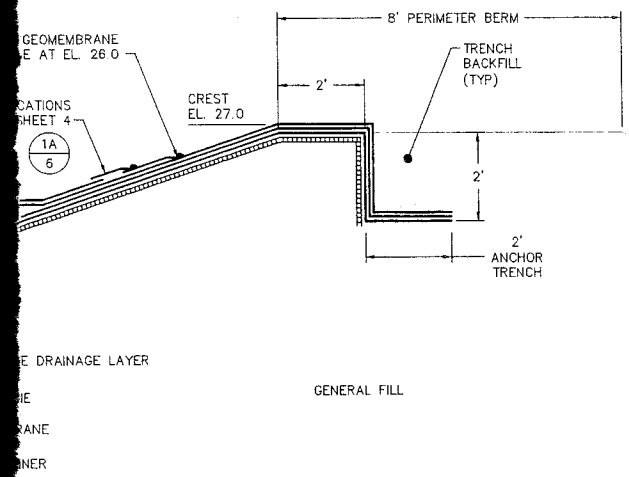
3/5 DETAI
LINER
SCALE: 1"
XREF: FL0819X0



1A / 4 DETAIL (TYPICAL)
GAS VENT
SCALE: 1" = 1'
XREF: FL0819X01005



(TYPICAL)
SYSTEM AT DIVISION BERM
2'



(TYPICAL)
SYSTEM AT PERIMETER BERM (SUMP AREA)
2'

NOTES:

1. ALL GEOMEMBRANES INDICATED ARE 60 mil HDPE TEXTURED GEOMEMBRANE.
2. FLSC = FLEXIBLE LEACHATE STORAGE CONTAINER, PLD = PRIMARY LEAK DETECTION, AND SLD = SECONDARY LEAK DETECTION.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
LINER SYSTEM DETAILS I

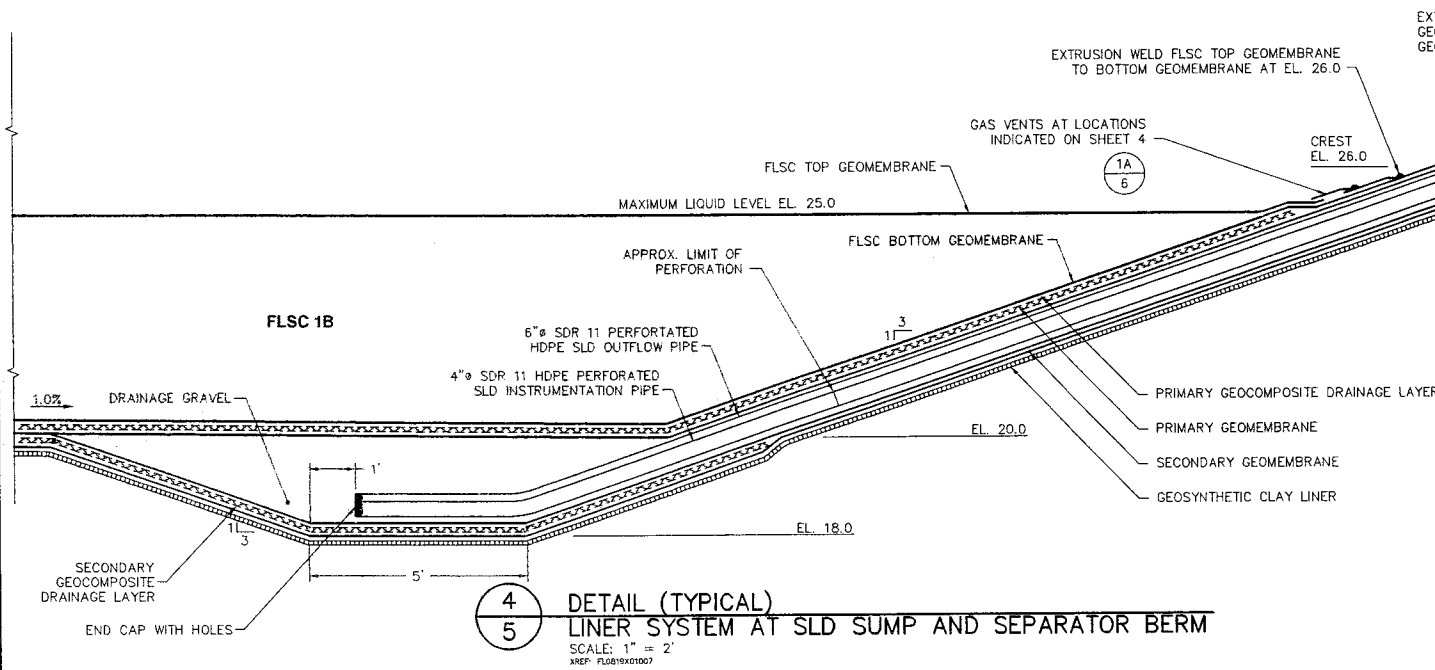
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GEOSYNTEC CONSULTANTS
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

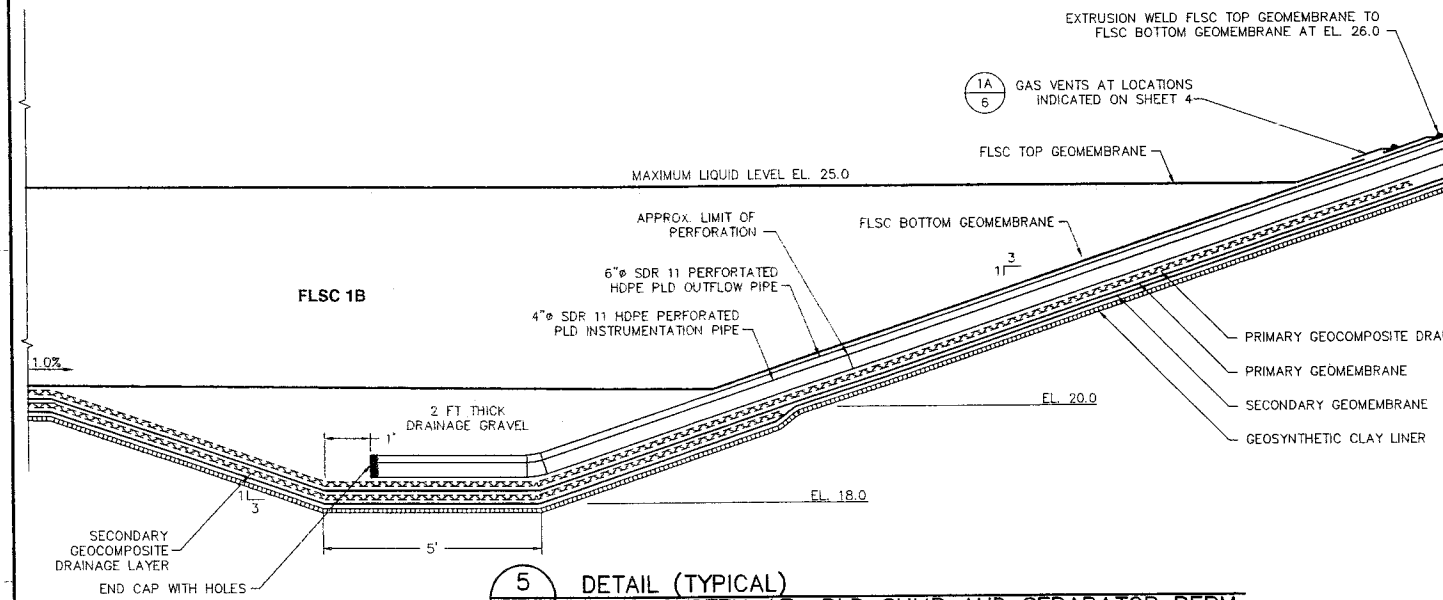
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: AS SHOWN
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01006
PDF DOC NO.: GEAG-06-38
SHEET NO.: 6 OF 13

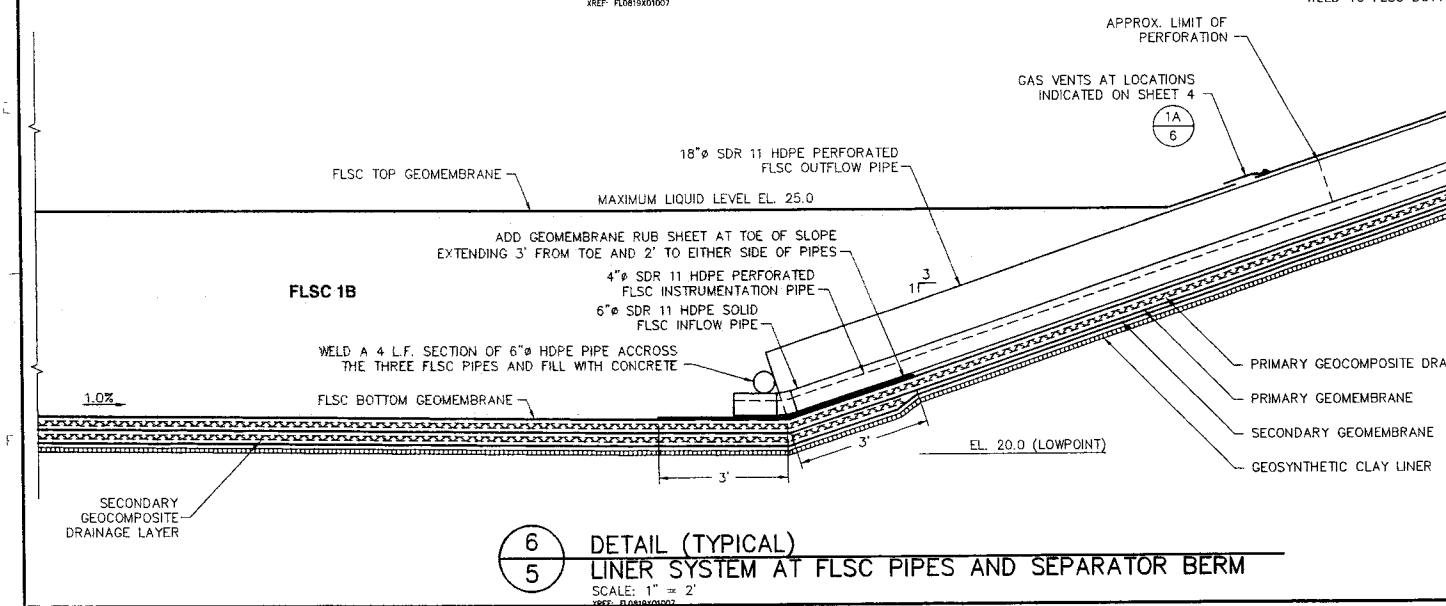
PERMIT DRAWING



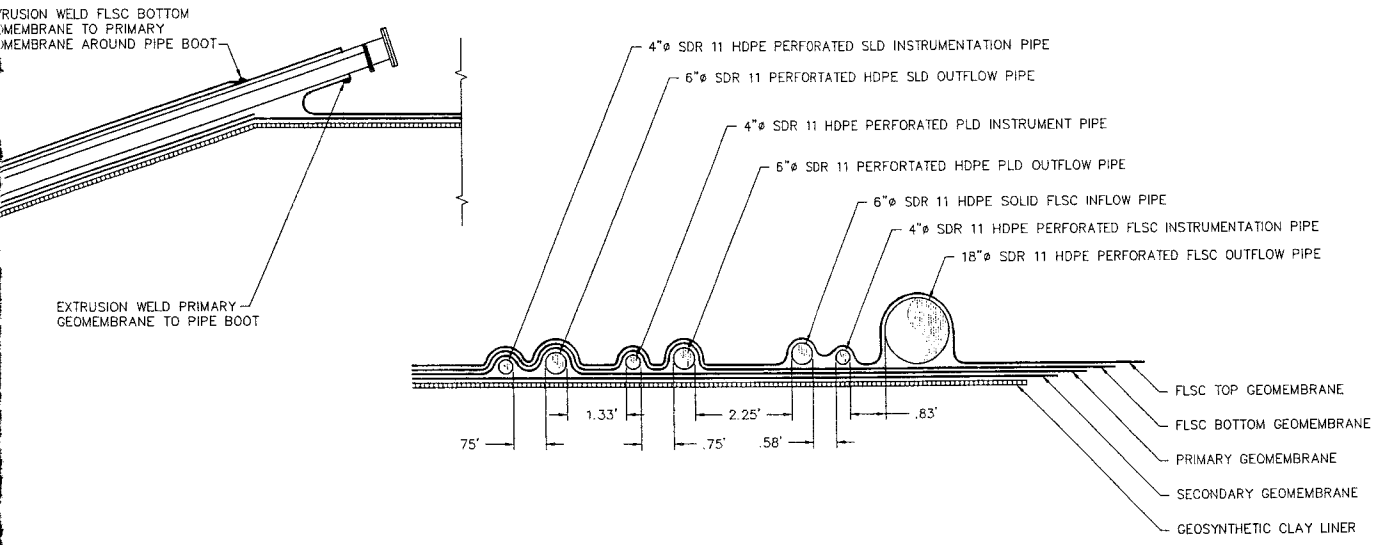
4/5 DETAIL (TYPICAL)
LINER SYSTEM AT SLD SUMP AND SEPARATOR BERM
SCALE: 1" = 2'
XREF: FL0819X01007



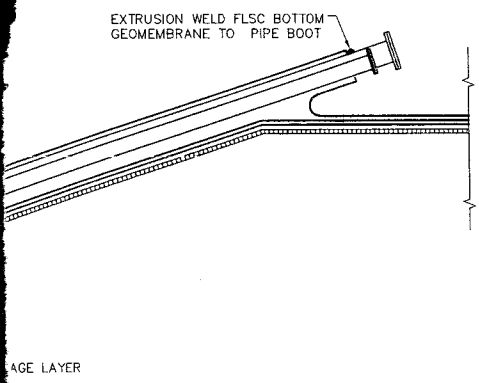
5/5 DETAIL (TYPICAL)
LINER SYSTEM AT PLD SUMP AND SEPARATOR BERM
SCALE: 1" = 2'
XREF: FL0819X01007

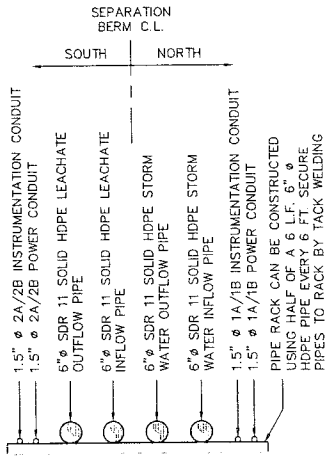


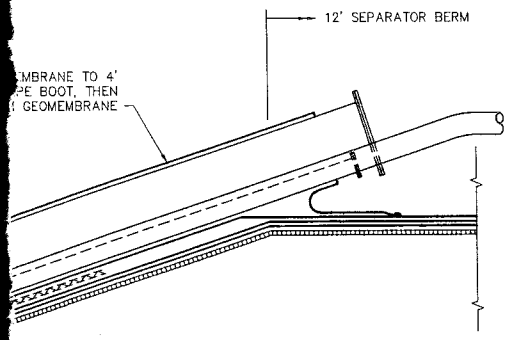
6/5 DETAIL (TYPICAL)
LINER SYSTEM AT FLSC PIPES AND SEPARATOR BERM
SCALE: 1" = 2'
XREF: FL0819X01007



7/5 SECTION (TYPICAL)
PIPES ON SIDESLOPE
SCALE: 1" = 2'
XREF: FL0819X01007







8/5 DETAIL (TYPICAL)
PIPE CONFIGURATION ON SEPARATOR BERM
SCALE: 1" = 2'
XREF: FL0819X01007

NOTES:

1. ALL GEOMEMBRANES INDICATED ARE 60-mil. HDPE TEXTURED GEOMEMBRANES.
2. FLSC = FLEXIBLE LEACHATE STORAGE CONTAINER, PLD = PRIMARY LEAK DETECTION, AND SLD = SECONDARY LEAK DETECTION.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
LINER SYSTEM DETAILS II

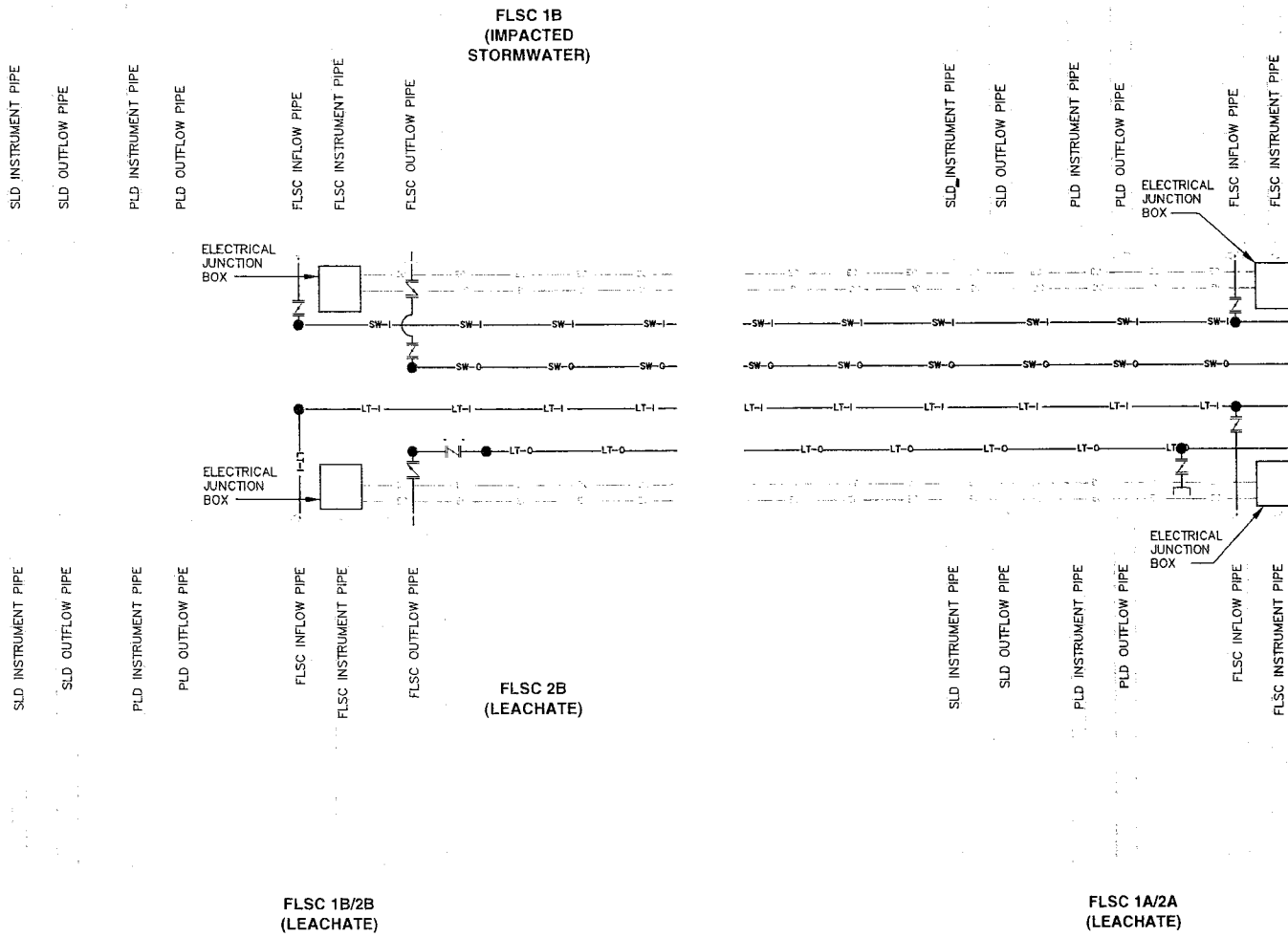
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL. 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

AYUSHMAN GUPTA
CERTIFICATE NO. 54023

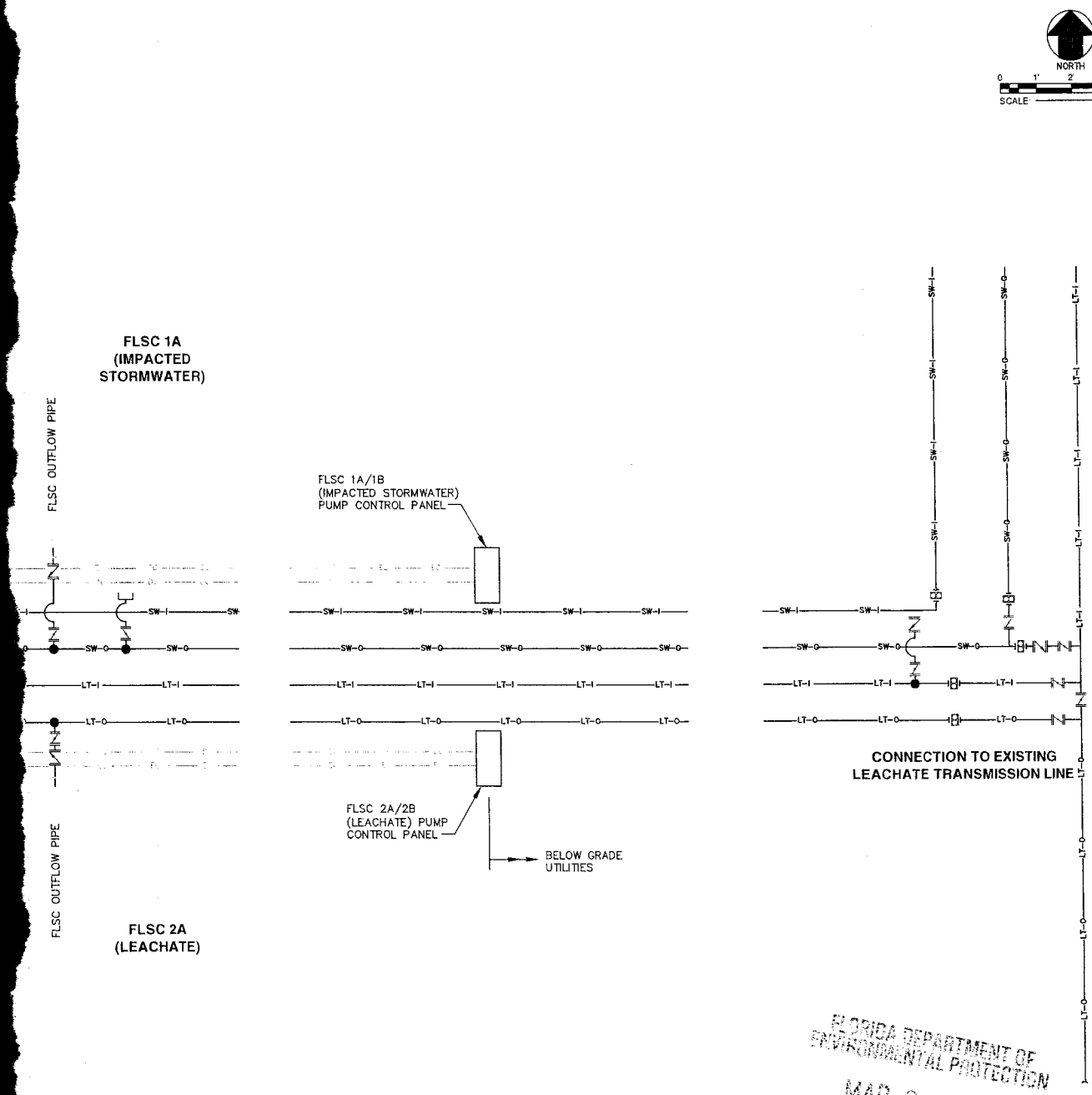
DATE: NOVEMBER 2006
SCALE: AS SHOWN
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01007
PDF DOC NO.: GEAG-06-38
SHEET NO.: 7 OF 13

PERMIT DRAWING



NOTES:

1. FLSC = FLEXIBLE LEACHATE STORAGE
LEAK DETECTION, AND SLD = SECOND



AINER, PLD = PRIMARY
EAK DETECTION.

LEGEND

SW-I	IMPACTED STORMWATER INFLOW PIPELINE
SW-O	IMPACTED STORMWATER OUTFLOW PIPELINE
LT-I	LEACHATE INFLOW PIPELINE
LT-O	LEACHATE OUTFLOW PIPELINE
	ELECTRIC POWER CONDUIT FOR LOAD-OUT PUMPS AND CONVENIENCE OUTLETS FOR SECONDARY AND PRIMARY PUMPS
	DATA CONDUIT

REV. NO.	DATE	DESCRIPTION	DES BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
FLSC PIPING LAYOUT

Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GEOSYNTEC CONSULTANTS
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: 1" = 2'
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01008
PDF DOC NO.: GEAG-06-38
SHEET NO.: 8 OF 13

PERMIT DRAWING

EQUIPMENT IDENTIFICATION

SUMP PUMP, CENTRIFUGAL (PC)

HORIZONTAL CENTRIFUGAL PUMP (PC)

SUBMERSIBLE POSITIVE DISPLACEMENT PUMP, PNEUMATIC (PS)

SUBMERSIBLE CENTRIFUGAL PUMP, MOTOR DRIVEN (PS)

FILTER OR STRAINER, BASKET TYPE (F)

PACKAGED EQUIPMENT BOUNDARY (PA)

INSTRUMENTS

INTERLOCK LOCAL MOUNTED

INTERLOCK, MAIN PANEL MOUNTED

INTERLOCK, REMOTE PANEL MOUNTED

INTERLOCK, NORMALLY INACCESSIBLE

DISCRETE INSTRUMENT–LOCAL MOUNTED

DISCRETE INSTRUMENT–MAIN PANEL MOUNTED

DISCRETE INSTRUMENT–REMOTE PANEL MOUNTED

DISCRETE INSTRUMENT–NORMALLY INACCESSIBLE

SHARED DISPLAY, SHARED CONTROL LOCAL MOUNTED

SHARED DISPLAY, SHARED CONTROL MAIN PANEL MOUNTED

SHARED DISPLAY, SHARED CONTROL REMOTE PANEL MOUNTED

COMPUTER FUNCTION–LOCAL MOUNTED

COMPUTER FUNCTION–MAIN PANEL MOUNTED

COMPUTER FUNCTION–REMOTE PANEL MOUNTED

COMPUTER FUNCTION–NORMALLY INACESSABLE

INSTRUMENTS SHARING COMMON HOUSING–LOCAL MOUNTED

INSTRUMENTS SHARING COMMON HOUSING–MAIN PANEL MOUNTED

TURBINE METER

THERMOWELL

PRESSURE REDUCING REGULATOR SELF CONTAINED (* = SET PRESSURE)

BACKPRESSURE REGULATOR SELF CONTAINED (* = SET PRESSURE)

DIAPHRAGM ACTUATOR

MOTOR OR PISTON ACTUATOR M = MOTOR, P = PISTON

SOLENOID

GEAR ACTUATOR

PRESSURE SAFETY VALVE

PRESSURE/VACUUM SAFETY VALVE

RUPTURE DISK OR SAFETY HEAD FOR PRESSURE RELIEF

RUPTURE DISK OR SAFETY HEAD FOR VACUUM RELIEF

PRESSURE/VACUUM RELIEF PLATE

FAILURE MODE

FAILURE POSITIONS

FC – FAIL CLOSED (ASSUMED)
FO – FAIL OPEN
FI – FAIL INDETERMINATE
FL – FAIL LOCKED

LINE SYMBOLS

MAJOR FLOW LINE

SECONDARY FLOW LINE

AIR DUCT

FLEXIBLE HOSE

PNEUMATIC INSTRUMENT SIGNAL

HYDRAULIC INSTRUMENT SIGNAL

CONDUIT

ELECTRICAL SIGNAL

CAPILLARY INSTRUMENT SIGNAL

OPTICAL SIGNAL

SOFTWARE LINK

LINE CONTINUATION

TIE POINT

SPECIFICATION BREAK, XX = PIPING MATERIAL AND CLASS

SHARED DISPLAY EXPLANATION

USED TO INDICATE A "SOFTWARE INTERNAL LINK" BETWEEN FUNCTIONS– NO EXTERNAL WIRING REQUIRED

FUNCTION PERFORMED WITHIN LOCAL PROCESSOR

1. DATA AVAILABLE ON P.C.L. (PROCESS COMMUNICATIONS LINK)
2. FUNCTION PERFORMED BY LOCAL PROCESSOR
3. DISPLAY ON CONSOLE C.R.T. SCREEN
4. DATA CAN BE TREND RECORDED ON C.R.T. SCREEN AND HARD COPY REPORT

COMPUTER INTERFACE SYMBOLS

AI ANALOG INPUT

AO ANALOG OUTPUT

DI DIGITAL INPUT

DO DIGITAL OUTPUT

PING SYMBOLS

FLANGED CONNECTION

CONCENTRIC REDUCER

ECCENTRIC REDUCER, F.O.T.

BLIND FLANGE

GATE VALVE

BALL VALVE

GLOBE VALVE

ANGLE VALVE

ANGLE GLOBE VALVE

BUTTERFLY VALVE

PLUG VALVE

CHECK VALVE

BLOCK AND BLEED VALVE

INSTRUMENT VALVE

NEEDLE VALVE

AIR ELIMINATOR

EXPANSION JOINT

CAMLOCK CONNECTION FEMALE

CAMLOCK CONNECTION MALE

SAMPLE POINT

PIPE CAP

EXTERNAL STRAINER

EPG PDISCONNECT ADAPTERS

PRESSURE GUAGE

FLOW METER

LEVEL TRANSDUCER

LEAK DETECTOR

DEFINITIONS

CAP – CAPACITY
C.S. – CARBON STEEL
C.R.T. – CATHODE RAY TUBE
D.I. – DUCTILE IRON
°F – DEGREES FARENHEIT
F.O.T. – FLAT ON TOP
FRP – FIBERGLASS REINFORCED PLASTIC
gal – GALLON
gpm – GALLONS PER MINUTE
HLL – HIGH LIQUID LEVEL
HP – HORSEPOWER
HDPE – HIGH DENSITY POLYETHYLENE
LLL – LOW LIQUID LEVEL
mA – MILLIAMPERE
MW – MANWAY
NLL – NORMAL LIQUID LEVEL
O.D. – OUTSIDE DIAMETER
PRESS. – PRESSURE
psig – POUNDS PER SQUARE INCH GAUGE
PVC – POLYVINYL CHLORIDE
RPM – REVOLUTIONS PER MINUTE
RTD – RESISTANCE TEMPERATURE DETECTOR
SCFM – STANDARD CUBIC FEET PER MINUTE
S.S. – STAINLESS STEEL
TDH – TOTAL DYNAMIC HEAD
TEMP. – TEMPERATURE
T.H. – TOTAL HIEGHT
t.t. – TANGENT TO TANGENT
V – VOLTS
VFD – VARIABLE FREQUENCY DRIVE
WC – WATER COLUMN

EQUIPMENT TAG NUMBERS

B – 2 6 01 01

- SEQUENCE
- STATION NUMBER
- CLASSIFICATION
- SYSTEM
- EQUIPMENT IDENTIFICATION

INSTRUMENT TAG NUMBERS

PI – 3 1 1 2 01

- SEQUENCE
- STATION NUMBER
- CLASSIFICATION
- SYSTEM
- INSTRUMENT IDENTIFICATION

SYSTEM

1 – STORMWATER COLLECTION
2 – STORMWATER TRANSMISSION AND STORAGE
3 – STORMWATER DISPOSAL
4 – LEACHATE TRANSMISSION AND STORAGE
5 – LEACHATE DISPOSAL

CLASSIFICATION

– STORM WATER
– LEACHATE 1

INSTRUMENT IDENTIFICATION

FIRST LETTERS	INITIATING OR MEASURED VARIABLE	CONTROLLERS				READOUT DEVICES		SWITCHES AND ALARM DEVICES (a)			TRANSMITTERS			SOLENOIDS, RELAYS, COMPUTING DEVICES	PRIMARY ELEMENTS	DATA INPUT	WELL OR PROBE	VIEWING DEVICE, GLASS	FINAL ELEMENT
		RECORDING	INDICATING	BLIND	SELF-ACTUATED CONTROL VALVES	RECORDING	INDICATING	HIGH**	LOW	COMB	RECORDING	INDICATING	BLIND						
A B C D E	ANALYSIS BURNER/COMBUSTION AIR/WATER RATIO USER'S CHOICE VOLTAGE	ARC BRC ERC	AIC BIC EIC	AC BC EC		AR BR ER	AI BI EI	ASH BSH ESH	ASL BSL ESL	ASHL BSHL ESHL	ART BRT ERT	AIT BIT EIT	AT BT ET	AY BY EY	AE BE EE	CX	AW BW	BG	AV BZ EZ
F FQ FF G H	FLOW RATE FLOW QUANTITY FLOW RATIO USER'S CHOICE HAND	FRC FQRC FFRC 	FIC FQIC FFIC HIC	FC FFC HC	FCV, FICV	FR FQR 	FI FQI 	FSH FQSH FFSH 	FSL FQSL FFSL 	FSHL HS	FRT 	FIT FQIT 	FT FQT 	FY FQY 	FE FQE FE 			FG	FV FQV FFV HV
I J KQ L M	CURRENT POWER TIME QUANTITY LEVEL LEAK	IRC JRC KQRC LRC 	IIC JIC KQIC LIC 	 KQC LC 	 KQCV LCV 	IR JR KQR LR 	II JI KQI LI MI	ISH JSH KQSH LSH MSH	ISL JSL KQSL LSL MSL	ISHL JSHL KQSHL LSHL 	IRT JRT KQRT LRT 	IIT JIT KQIT LIT 	IT JT KQT LT 	IY JY KQY LY 	IE JE KQE LE 		 LW 	 LG 	IV JV KQV LV
N O P PD Q	USER'S CHOICE USER'S CHOICE PRESSURE/ VACUUM PRESSURE, DIFFERENTIAL QUANTITY	 PRC PDRC QRC	 PIC PDIC QIC	 PC PDC 	 PCV PDCV 	 PR PDR QR	 PI PDI QI	 PSH PDSH QSH	 PSL PDSL QSL	 PSHL QSHL	 PRT PDRT QRT	 PIT PDIT QIT	 PT PDT QT	 PY PDY QY	 PE PE QE				 PV PDV QZ
R SP T TD U	RADIATION SET POINT TEMPERATURE TEMPERATURE, DIFFERENTIAL MULTIVARIABLE	RRC TRC TDRC 	RIC TIC TDIC 	RC TC TDC 	 TCV TDCV 	RR TR TDR UR	RI TI TDI UI	RSH SPH TSH TDSH 	RSL SPL TSL TDSL 	RSHL SPHH TSHL 	RRT TRT TDRT 	RIT TIT TDIT 	RT TT TDT 	RY TY TDY UY	RE TE TE 		RW TW TW 		RZ TV TDV UV
V W WD X Y	VIBRATION/MACHINERY ANALYSIS WEIGHT/FORCE WEIGHT/FORCE, DIFFERENTIAL UNCLASSIFIED EVENT/STATE/ PRESENCE	 WRC WDRC 	 WIC WDIC YIC	 WC WDC YC	 WCV WDCV 	VR WR WDR YR	VI WI WDI YI	VSH WSH WDSH YSH	VSL WSL WDSL YSL	VSHL WSHL 	VRT WRT WDRT 	VIT WIT WDIT 	VT WT WDT YT	VY WY WDY YY	VE WE WDE YE				VZ WZ WDZ YZ
Z ZD	POSITION/DIMENSION GAUGING, DEVIATION	ZRC ZDRC	ZIC ZDIC	ZC ZDC	ZCV ZDCV	ZR ZDR	ZI ZDI	ZSH ZDSH	ZSL ZDSL	ZSHL	ZRT ZDRT	ZIT ZDIT	ZT ZDT	ZY ZDY	ZE ZDE				ZV ZDV

(a) "A", ALARM, THE ANNUNCIATING DEVICE MAY BE USED IN THE SAME FASHION AS "S", SWITCH, THE ACTUATING DEVICE. THE LETTERS H AND L MAY BE OMITTED IN THE UNDEFINED CASE. HH OR LL MAY BE USED TO INDICATE HIGH-HIGH OR LOW-LOW DEVICES.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
LEACHATE MANAGEMENT SYSTEM
PROCESS DIAGRAM LEGEND SHEET

Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GEOSYNTEC CONSULTANTS
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL. 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: NA
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01009
PDF DOC NO.: GEAG-06-38
SHEET NO.: 9 OF 13

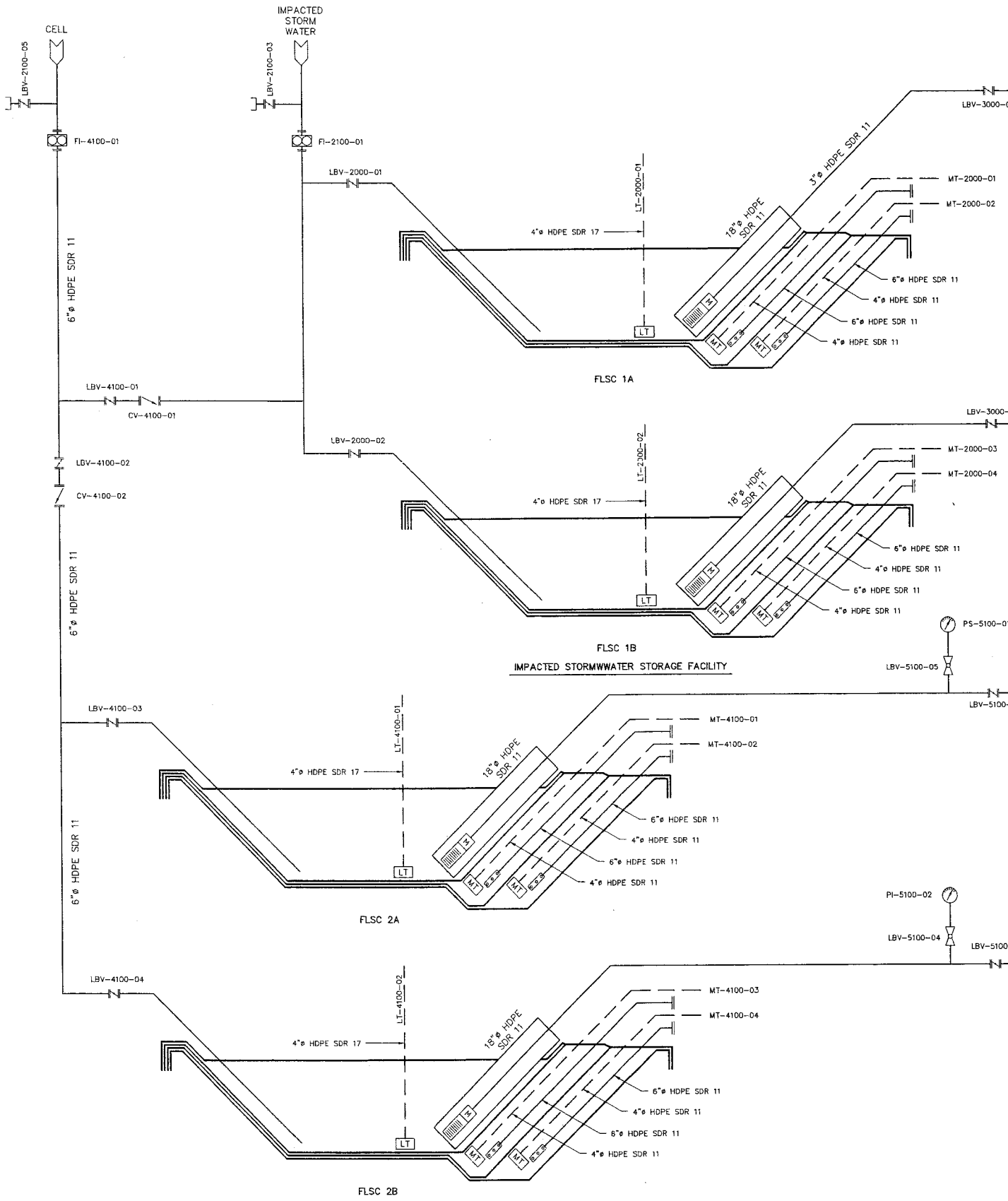
PERMIT DRAWING

PS-2000-01, PS-2000-02,
PS-4100-01, PS-4100-02
SUBMERSIBLE SUMP PUMP

MODEL: SLIGO SERIES 1-2-3 PSF
DRIVER: 2 HP, 460-3PH.
DESIGN CAPACITY: 50 GPM @ 46 FT HEAD

FI-2000-01 THROUGH FI-5100-
TOTALIZING FLOW METER

MODEL: TIGERMAG, FM656
MATERIALS: STAINLESS STEEL, FLANGED FACED

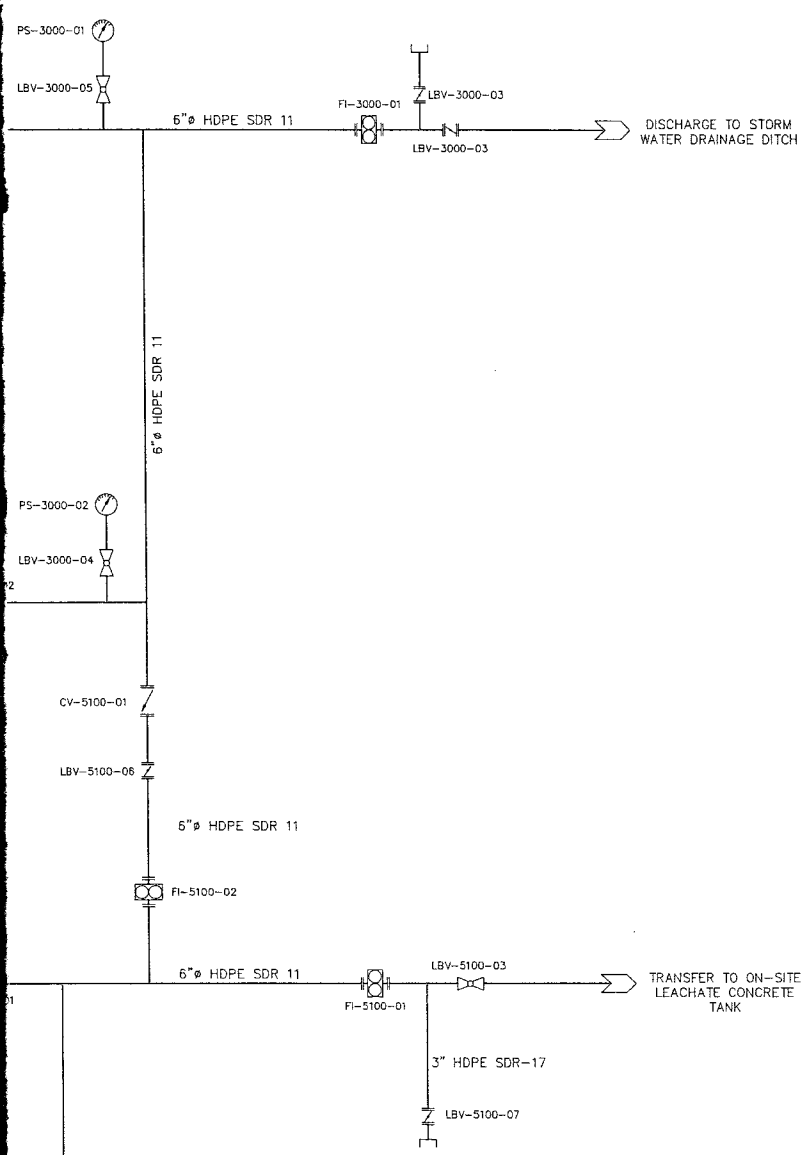


LT-2000-01, LT-2000-02, LT-4100-01, LT-4100-02
LEVEL TRANSDUCER

MODEL: SLIGO SYSTEMS SERIES PT-009-50 W/ BUILT-IN SURGE SUPPRESSION, PT15X0__P

MT-2000-01 THROUGH MT-2000-04, MT 4100-01 THROUGH MT-4100-04
LEAK DETECTION ELEMENT

MODEL: EPG LEAK DETECTION SENSOR, ASLD-YH80



OPERATIONAL NOTES:

1. Each pump shall be protected against run dry conditions by a level transducer, LT-2000-01, LT-2000-02, LT-4100-01, and LT-4100-02.
2. MT-2000-01, MT-2000-02, MT-4100-01, MT-4100-02 will activate an alarm light to indicate that liquid has been detected in the leak detection system

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT: FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE: LEACHATE MANAGEMENT SYSTEM
MECHANICAL FLOW SCHEMATIC

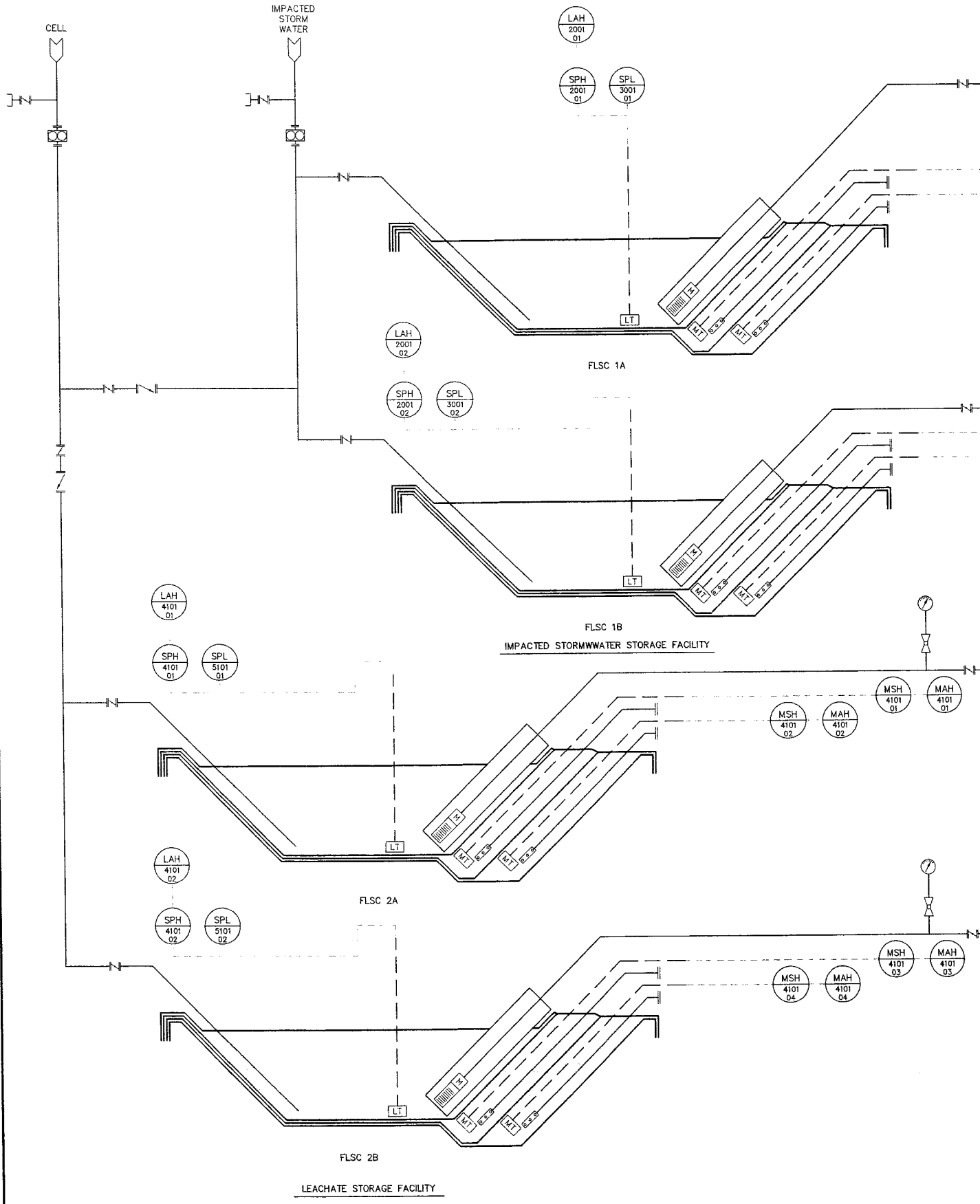
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 485-2620

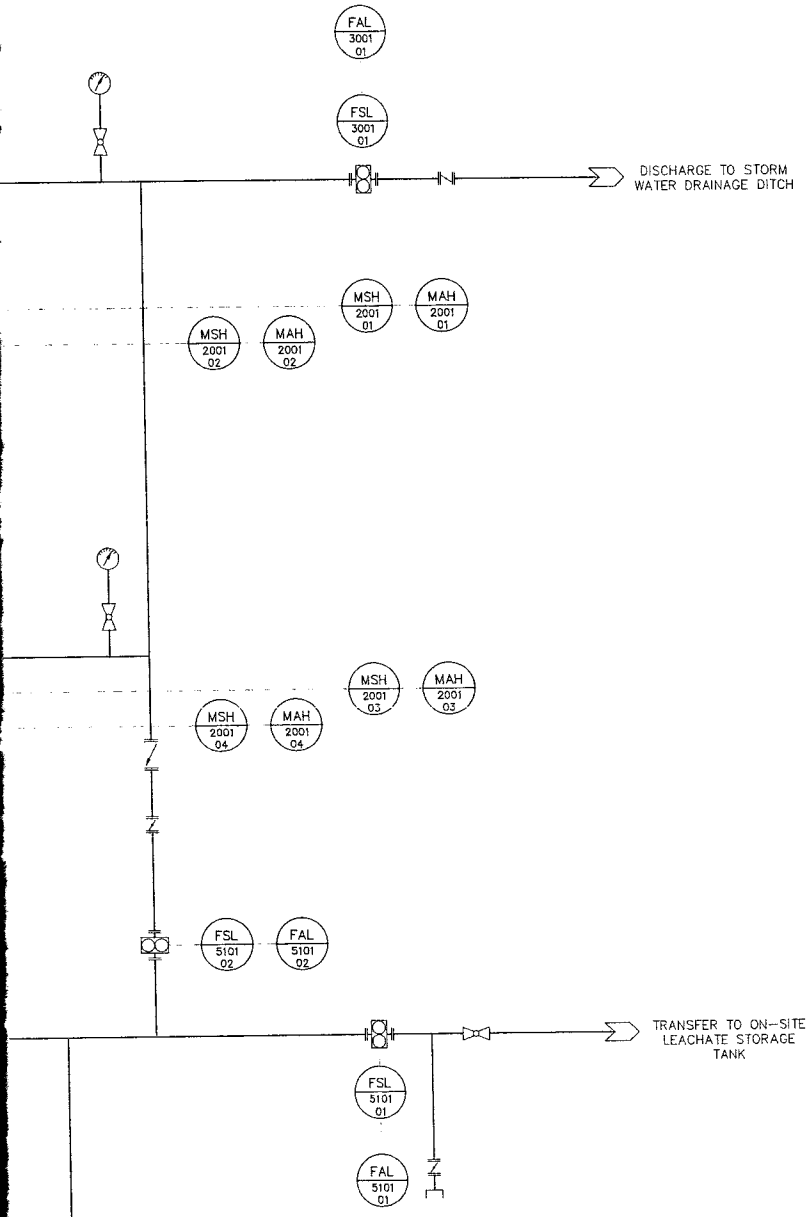
Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: NA
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01010
PDF DOC NO.: GEAG-06-38
SHEET NO.: 10 OF 13

PERMIT DRAWING


CELL
IMPACTED STORM WATER
LAH 2001 01
SPH 2001 01
SPL 3001 01
LT
M
MT
LAH 2001 02
SPH 2001 02
SPL 3001 02
FLSC 1A
FLSC 1B
IMPACTED STORMWWATER STORAGE FACILITY
LAH 4101 01
SPH 4101 01
SPL 5101 01
MSH 4101 01
MAH 4101 01
MSH 4101 02
MAH 4101 02
LAH 4101 02
SPH 4101 02
SPL 5101 02
FLSC 2A
MSH 4101 03
MAH 4101 03
MSH 4101 04
MAH 4101 04
FLSC 2B
LEACHATE STORAGE FACILITY



OPERATIONAL NOTES:

1. LOAD OUT PUMPS WILL BE OPERATED MANUALLY
2. SPL-3000-01, SPL-3000-02, SPL-5100-01, SPL 5100-02 SHALL SHUT OFF THEIR ASSOCIATED PUMP AT A MINIMUM DEPTH OF 6 INCHES OF FLUID IN THE STORAGE CONTAINER
3. SPH-2000-01, SPH-2000-02, SPH-4100-01, SPH-4100-02 SHALL ACTIVATE HIGH LEVEL ALARMS LAH-2000-01, LAH -2000-02, LAH -4100-01, LAH-4100-02 RESPECTIVELY TO INDICATE THAT THE ASSOCIATED STORAGE CONTAINER IS FULL TO ITS PERMITTED LEVEL.
4. MSHXX01-0X WILL ACTIVATE ALARM MAH-XX01-0X IF LIQUID IS DETECTED IN THE LEAK DETECTION SYSTEM.
5. FSLXX01-0X WILL ACTIVATE LOW FLOW ALARM FAL-XX01-0X IF A FLOW RATE OF LESS THAN FIVE GALLONS PER MINUTE IS DETECTED FOR ONE MINUTE.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT: FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE: LEACHATE MANAGEMENT SYSTEM
PROCESS AND INSTRUMENTATION SCHEMATIC

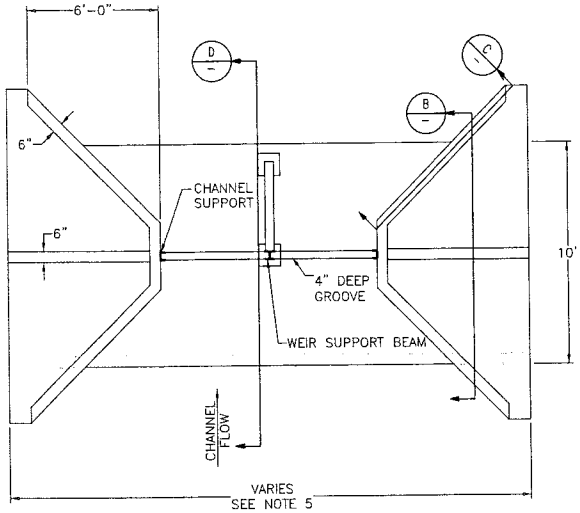
Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

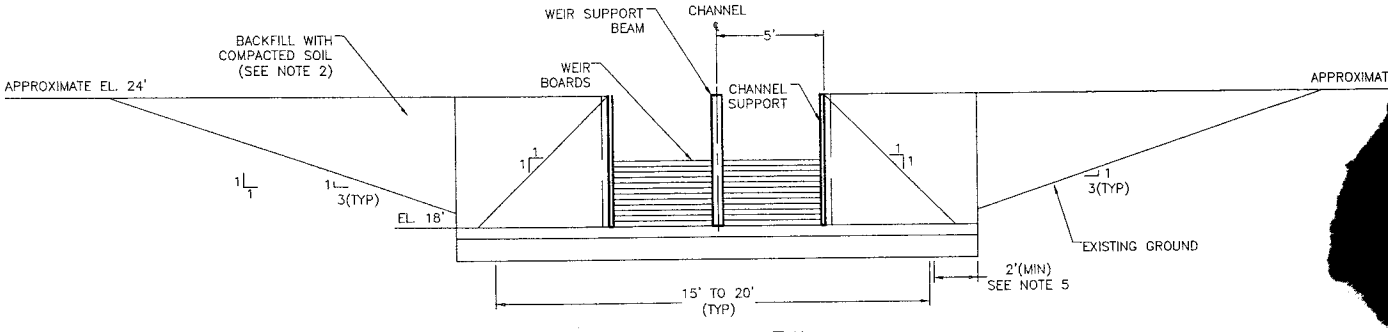
AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: NA
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01011
PDF DOC NO.: GEAG-06-38
SHEET NO.: 11 OF 13

PERMIT DRAWING



TOP VIEW

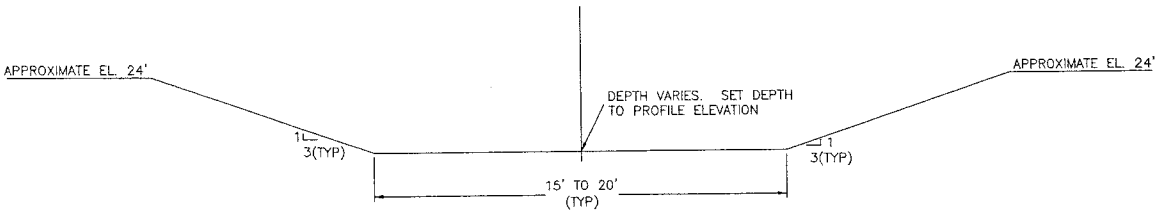


ELEVATION VIEW

9/2 WEIR DETAILS

SCALE: 1" = 4'

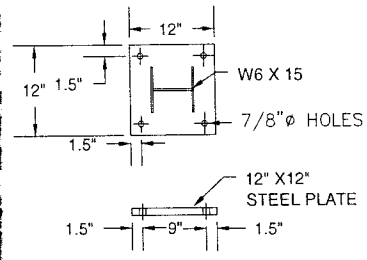
FL0819X0213



—/— PERIMETER DRAINAGE CHANNEL SECTION

SCALE: 1" = 4'

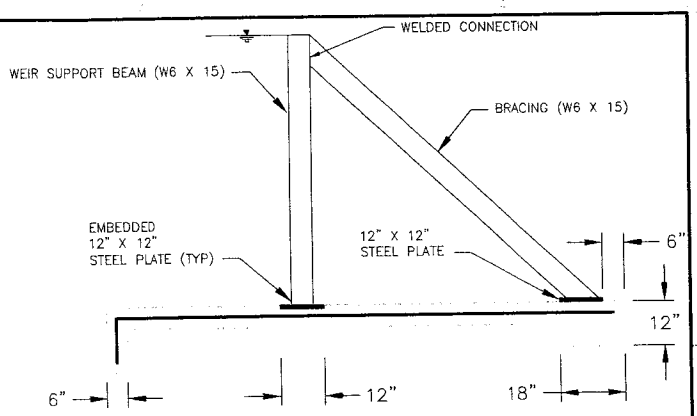
FL0819X0213



STEEL PLATE DETAIL

1" = 1'

FL0819X2014

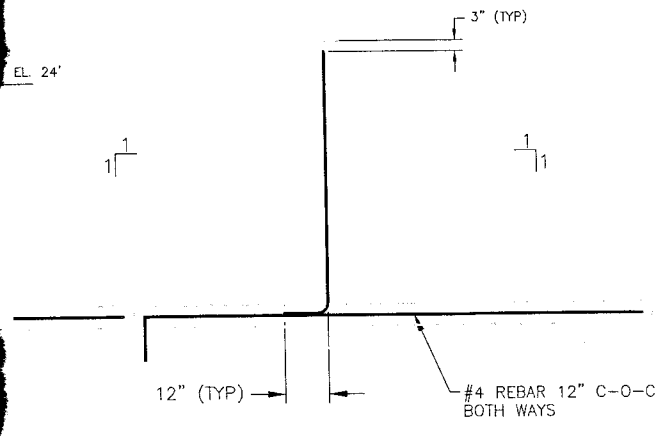


WIER BEAM SUPPORT DESIGN

B–B'

SCALE: 1" = 2'

FL0819X02012A

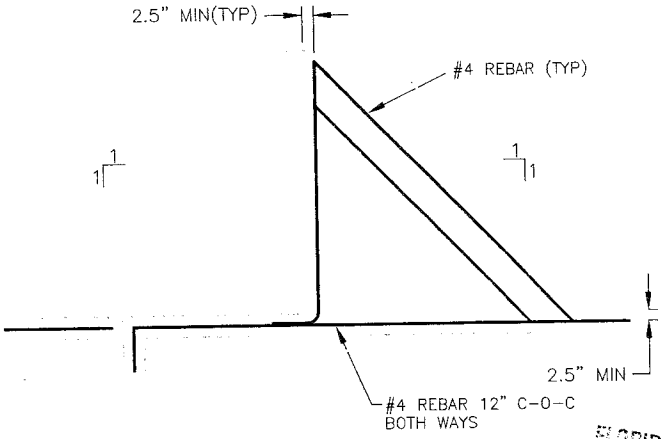


WIER CROSS SECTION

B–B'

SCALE: 1" = 2'

FL0819X02012A



WEIR CROSS SECTION

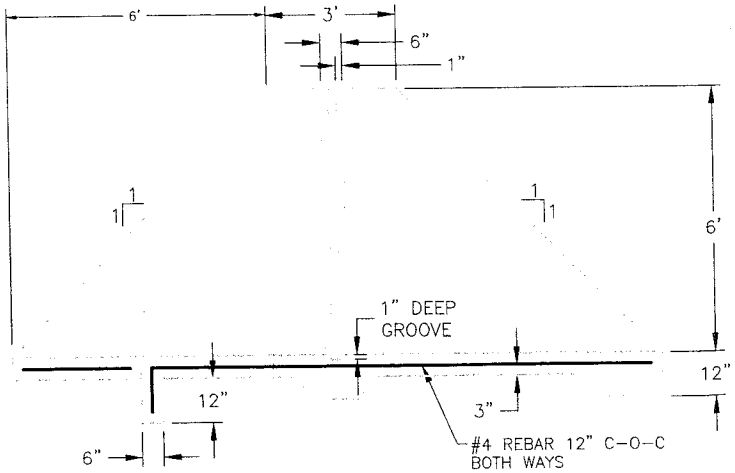
C–C'

SCALE: 1" = 2'

FL0819X02012A

NOTES:

1. WEIR BOARDS SHALL CONSIST OF 4"X6" LUMBER. BOARDS MAY BE FACED WITH MARINE GRADE PLYWOOD OR OTHER HARD SHEETING TO REDUCE LEAKAGE THROUGH BOARDS.
2. SOIL BACKFILL SHALL BE COMPACTED TO 95 PERCENT OF STANDARD PROCTOR DENSITY (ASTM D698), AT A MOISTURE CONTENT WITHIN 2 PERCENTAGE POINTS OF OPTIMUM MOISTURE CONTENT.
3. CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 3000 PSI AT 28 DAYS.
4. REINFORCING STEEL SHALL HAVE A MINIMUM TENSILE STRENGTH OF 60 KSI.
5. CONCRETE WEIR STRUCTURE SHALL EXTEND A MINIMUM OF 2FT INTO THE EXISTING DRAINAGE CHANNEL SIDE SLOPE.
6. CONCRETE ANCHOR BOLT HOLES SHALL BE 7" DEEP WITH A 1"ø.
7. THREADED ANCHOR BOLTS SHALL BE 7/8"ø, 304SS, RAWL CHEM–STUD TYPE BOLTS (OR EQUIVALENT.)



WEIR CROSS SECTION

D–D'

SCALE: 1" = 2'

FL0819X02012A

PERMIT DRAWING

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:

FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:

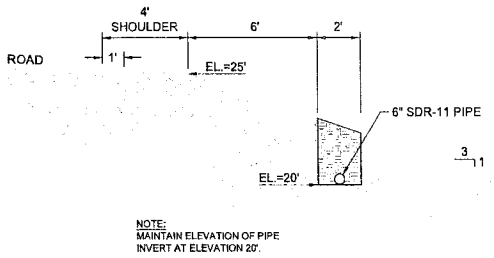
WEIR DETAILS

Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

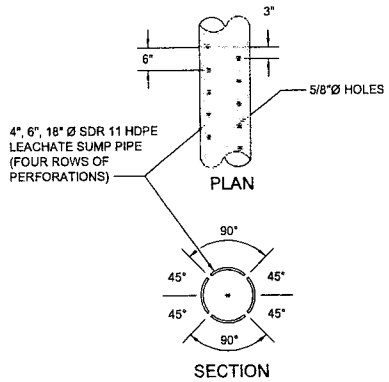
Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9728
AUTHORIZATION CERTIFICATE NO. 00004321

AYUSHMAN GUPTA
CERTIFICATE NO. 54023

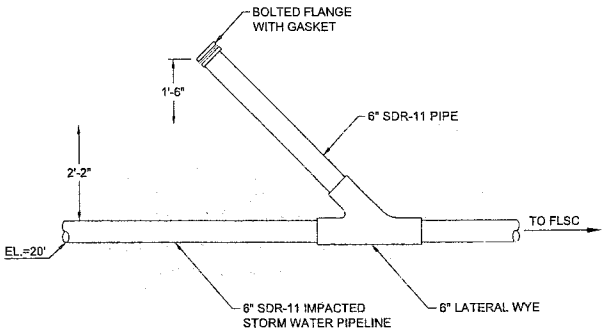
DATE: NOVEMBER 2006
SCALE: NA
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01012
PDF DOC NO.: GEAG-06-38
SHEET NO.: 12 OF 13



10/2 DETAIL
IMPACTED STORM WATER PIPELINE
1" = 4'
FL0819x1011



13/6 DETAIL – TYPICAL (NOTE 1)
PERFORATION FOR LEACHATE SUMP PIP
NTS
FL0819 X02012



12/2 DETAIL – TYPICAL
IMPACTED STORM WATER CLEAN-OUT
1" = 2'
FL0819x1012

S

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
MAR 29 2007
SOUTHWEST DISTRICT
TAMPA

NOTES:

1. PEFORATION DETAIL APPLIES TO ALL PIPES EXCEPT FOR THE FLSC INFLOW PIPES.

REV. NO.	DATE	DESCRIPTION	DES. BY	DR. BY	CHK. BY	RVW. BY	APP. BY
0	11/06	ISSUED FOR FDEP REVIEW	JDQ	JWO	AG	JDQ	AG

PROJECT:
FLEXIBLE LEACHATE STORAGE CONTAINERS
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
SARASOTA COUNTY, FLORIDA

TITLE:
MISCELLANEOUS DETAILS

Owner/Facility:
Sarasota County
SOLID WASTE OPERATIONS
4000 KNIGHTS TRAIL ROAD
NOKOMIS, FLORIDA 34275
TEL: (941) 861-1571 FAX: (941) 486-2620

Consultant/Engineer:
GeoSyntec Consultants
14055 RIVEREDGE DRIVE, SUITE 300,
TAMPA, FLORIDA 33637 USA
TEL: 813-558-0990 FAX: 813-558-9726
AUTHORIZATION CERTIFICATE NO. 00004321

AYUSHMAN GUPTA
CERTIFICATE NO. 54023

DATE: NOVEMBER 2006
SCALE: NA
PROJECT NO.: FL0819.02
FILE NO.: FL0819-01013
PDF DOC NO.: GEAG-06-38
SHEET NO.: 13 OF 13

PERMIT DRAWING

ATTACHMENT 6

FLSC Storage Capacity Calculations

A Gupta
3/29/07

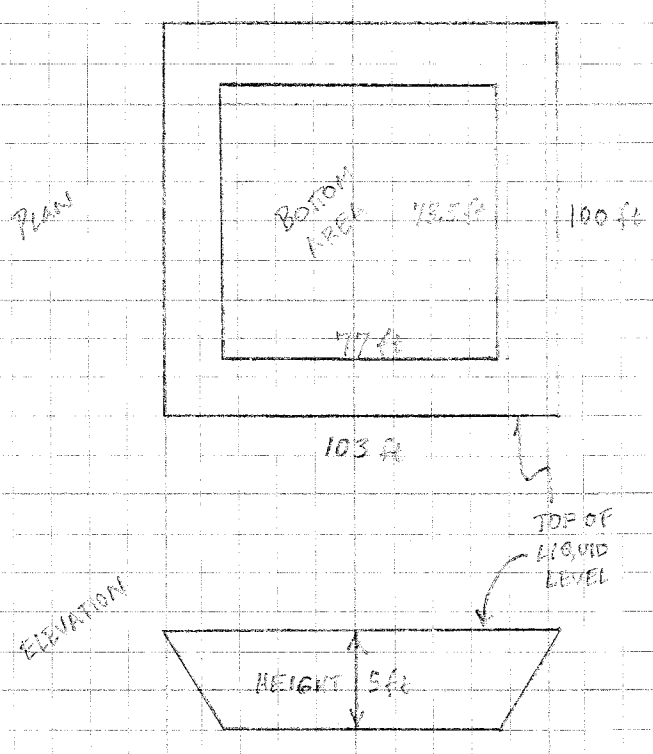
Geosyntec consultants

Written by: Juan Quiroz Date: 17/01/07 (DD MM YY) Reviewed by: _____ Date: ___/___/___ (DD MM YY)

Client: Sarasota Project: FLSCs Project/Proposal No. FL1109 Task No. _____

NOMINAL STORAGE CAPACITY OF FLSCs

ONE FLSC CELL CONFIG: (REFERENCE SH. 4 AND 5 OF PERMIT DWGS.)



Bottom Area = 77' x 73.5' = 5,659.5 ft²

Top Area = 103' x 100' = 10,300 ft²

AVG. END AREA = $\frac{5,659.5\,ft^2 + 10,300\,ft^2}{2}$

= 7,979.75 ft²

VOLUME = AVG. END AREA x HEIGHT

= 7,979.75 ft² x 5 ft

= 39,898.75 ft³

STORAGE CAPACITY PER FLSC CELL:

$39,898.75\,ft^3 \left(\frac{7.48\,gal}{1\,ft^3}\right) = 298,442.7\,gal$ say **300,000 gal**

ATTACHMENT 7

B. DISPOSAL FACILITY GENERAL INFORMATION

1. Provide brief description of disposal facility design and operations planned under this application:

This application is to construct a flexible leachate storage container (FLSC) facility at CCSWDC which consists of four lined cells. Each cell has a storage capacity of 300,000 gallons. Two of the cells will provide an additional leachate storage capacity and the remaining two cells will provide impacted storm water storage.

2. Facility site supervisor: **Frank Coggins**

Title: **Manager/Solid Waste Operations** Telephone: (941) **650-4160**

fcoggins@scgov.net
E-Mail address (if available)

3. Disposal area: Total **N/A** acres; Used **N/A** acres; Available **N/A** acres.

4. Weighing scales used: [ ] Yes [✓] No

5. Security to prevent unauthorized use: [✓] Yes [ ] No

6. Charge for waste received: **N/A** $/yds^3 **N/A** $/ton

7. Surrounding land use, zoning:

[ ] Residential
[ ] Agricultural
[ ] Commercial
[ ] Industrial
[✓] None
[ ] Other Describe: _____

8. Types of waste received: N/A

[ ] Residential
[ ] Commercial
[ ] Incinerator/WTE ash
[ ] Treated biomedical
[ ] Water treatment sludge
[ ] Air treatment sludge
[ ] Agricultural
[ ] Asbestos
[ ] C & D debris
[ ] Shredded/cut tires
[ ] Yard trash
[ ] Septic tank
[ ] Industrial
[ ] Industrial sludge
[ ] Domestic sludge
[ ] Other Describe: _____

9. Salvaging permitted: [ ] Yes [ ] No N/A

10. Attendant: [✓] Yes [ ] No Trained operator: [✓] Yes [ ] No

11. Spotters: Yes [ ] No [ ] Number of spotters used: **N/A**

12. Site located in: [ ] Floodplain [ ] Wetlands [ ] Other **N/A**

13. Property recorded as a Disposal Site in County Land Records: [ ] Yes [✓] No

14. Days of operation: Monday thru Sunday

15. Hours of operation: 24 hours a day

16. Days Working Face covered: N/A

17. Elevation of water table: 16 Ft. (NGVD 1929)

18. Number of monitoring wells: 1

19. Number of surface monitoring points: N/A

20. Gas controls used: [ ] Yes [✓] No Type controls: [ ] Active [ ] Passive N/A

Gas flaring: [ ] Yes [✓] No Gas recovery: [ ] Yes [✓] No

21. Landfill unit liner type: N/A

[ ] Natural soils
[ ] Single clay liner
[ ] Single geomembrane
[ ] Single composite
[ ] Slurry wall
[ ] Double geomembrane
[ ] Geomembrane & composite
[ ] Double composite
[ ] None
[ ] Other Describe: _____

22. Leachate collection method:

[✓] Collection pipes
[ ] Geonets
[ ] Well points
[ ] Perimeter ditch
[ ] Sand layer
[ ] Gravel layer
[ ] Interceptor trench
[ ] None
[ ] Other Describe: _____

23. Leachate storage method:

[ ] Tanks
[✓] Surface impoundments with flexble storage containers
[ ] Other Describe: _____

24. Leachate treatment method:

[ ] Oxidation
[ ] Secondary
[ ] Advanced
[✓] None
[ ] Other _____
[ ] Chemical treatment
[ ] Settling

D. **PROHIBITIONS** (62-701.300, FAC)

S	LOCATION	N/A	N/C		
__	3.5	__	__	1.	Provide documentation that each of the siting criteria will be satisfied for the facility; (62-701.300(2), FAC)
__		x	__	2.	If the facility qualifies for any of the exemptions contained in Rules 62-701.300(12) through (16), FAC, then document this qualification(s).
__		x	__	3.	Provide documentation that the facility will be in compliance with the burning restrictions; (62-701.300(3), FAC)
__		x	__	4.	Provide documentation that the facility will be in compliance with the hazardous waste restrictions; (62-701.300(4), FAC)
__		x	__	5.	Provide documentation that the facility will be in compliance with the PCB disposal restrictions; (62-701.300(5), FAC)
__		x	__	6.	Provide documentation that the facility will be in compliance with the biomedical waste restrictions; (62-701.300(6), FAC)
__		x	__	7.	Provide documentation that the facility will be in compliance with the Class I surface water restrictions; (62-701.300(7), FAC)
__		x	__	8.	Provide documentation that the facility will be in compliance with the special waste for landfills restrictions; (62-701.300(8), FAC)
__		x	__	9.	Provide documentation that the facility will be in compliance with the special waste for waste-to-energy facilities restrictions; (62-701.300(9), FAC)
__		x	__	10.	Provide documentation that the facility will be in compliance with the liquid restrictions; (62-701.300(10), FAC)
__		x	__	11.	Provide documentation that the facility will be in compliance with the used oil restrictions; (62-701.300(11), FAC)

M. WATER QUALITY AND LEACHATE MONITORING REQUIREMENTS (62-701.510, FAC):

S	LOCATION	N/A	N/C	
		x		1. Water quality and leachate monitoring plan shall be submitted describing the proposed ground water, surface water and leachate monitoring systems and shall meet at least the following requirements;
x	3.4			a. Based on the information obtained in the hydrogeological investigation and signed, dated and sealed by the PG or PE who prepared it; (62-701.510(2)(a),FAC)
		x		b. All sampling and analysis preformed in accordance with Chapter 62-160, FAC; (62-701.510(2)(b),FAC)
				c. Ground water monitoring requirements; (62-701.510(3),FAC)
		x		(1) Detection wells located downgradient from and within 50 feet of disposal units;
		x		(2) Downgradient compliance wells as required;
		x		(3) Background wells screened in all aquifers below the landfill that may be affected by the landfill;
		x		(4) Location information for each monitoring well;
		x		(5) Well spacing no greater than 500 feet apart for downgradient wells and no greater than 1500 feet apart for upgradient wells unless site specific conditions justify alternate well spacings;
x	3.4			(6) Well screen locations properly selected;
		x		(7) Procedures for properly abandoning monitoring wells;
		x		(8) Detailed description of detection sensors if proposed.

ATTACHMENT 8

Pipe Perforation Sizing Calculations

A Gupta
3/29/07

Geosyntec consultants

Written by: Juan Quiroz Date: 14/03/07 (DD MM YY) Reviewed by: _____ Date: __/__/__ (DD MM YY)

Client: Sarasota Co. Project: FL-SC3 Project/Proposal No. FL1109 Task No. _____

PIPE PERFORATION SIZING CALCULATIONS

REFERENCE: U.S. ENVIRONMENTAL PROTECTION AGENCY (USEPA). "LINING OF WASTE IMPOUNDMENT AND DISPOSAL FACILITIES," EPA Nº SW-870, USEPA, WASHINGTON, D.C., MARCH 1983.

GIVEN: No. 4 STONE (LEACHATE SUMP DRAINAGE GRAVEL)

D_{85} OF No. 4 STONE ≈ 1.26 INCH (SEE ATTACHED ASTM D 448 STANDARD FOR AGGREGATE SIZES)

$\frac{5}{8}$ INCH DIAMETER PIPE PERFORATIONS FOR LEACHATE SUMP PIPES

REQ'D: MAXIMUM PERFORATION DIAMETER TO PREVENT GRAVEL FROM PASSING THROUGH ($D_{H_{MAX}}$).

EQ'N:

$$D_{H_{MAX}} = \frac{D_{85}}{F} \qquad [\text{USEPA, 1983}]$$

WHERE D_{85}: PARTICLE SIZE OF GRAVEL FOR WHICH 85 % BY WEIGHT OF THE PARTICLES ARE FINER (INCHES)

F = FACTOR VARYING FROM 1.2 TO 2 (ASSUME 1.5)

$$D_{H_{MAX}} = \frac{1.26 \text{ INCH}}{1.5} = 0.840 \text{ INCHES}$$

$D_{H_{MAX}}$ = 0.840 INCHES > $\frac{5}{8}$ INCH (0.625 INCH) DIAMETER PIPE PERFORATIONS

∴ $\frac{5}{8}$ INCH DIAMETER PIPE PERFORATIONS FOR LEACHATE SUMP PIPES ARE ADEQUATE.

Designation: D 448 – 86

Standard Classification for Sizes of Aggregate for Road and Bridge Construction[1]

This standard is issued under the fixed designation D 448; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This classification defines aggregate size designations and ranges in mechanical analyses for standard sizes of coarse aggregate and screenings for use in the construction and maintenance of various types of highways and bridges.

1.2 With regard to sieve sizes and the size of aggregate as determined by the use of testing sieves, the values in inch-pound units are shown for the convenience of the user; however, the standard sieve designation shown in parentheses is the standard value as stated in Specification E 11.

2. Referenced Documents

2.1 *ASTM Standards:*

C 136 Method for Sieve Analysis of Fine and Coarse Aggregates[2]

D 75 Practice for Sampling Aggregates[3]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

3. Significance and Use

3.1 Contract documents may specify certain of these aggregate sizes for specific uses or may suggest one or more of these sizes as appropriate for the preparation of various end-product mixtures. In some cases, closer limits on variability of the aggregate grading may be required.

4. Manufacture

4.1 The standard sizes of aggregate described in this classification may be manufactured by means of any suitable process used to separate raw material into the desired size ranges. Standard sizes may also be produced by blending two or more different components.

5. Standard Sizes

5.1 Standard sizes of coarse aggregate shall comply with the sizes given in Table 1. All sizes shall be determined by means of laboratory sieves having square openings and conforming to Specification E 11.

6. Basis of Classification

6.1 Classification is based upon the size number and size ranges shown in Table 1 with the aggregate sampled in accordance with Practice D 75 and tested for grading by Method C 136.

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[1] This classification is under the jurisdiction of ASTM Committee D-4 on Road and Paving Materials and is the direct responsibility of Subcommittee D04.50 on Aggregate Specifications.

Current edition approved March 27, 1986. Published June 1986. Originally published as D 448 – 37T. Last previous edition D 448 – 80.

[2] *Annual Book of ASTM Standards*, Vols 04.02 and 04.03.

[3] *Annual Book of ASTM Standards*, Vol 04.03.

[4] *Annual Book of ASTM Standards*, Vols 04.01 and 14.02.

TABLE 1 Standard Sizes of Processed Aggregate

Size Number	Nominal Size, Square Openings	Amounts Finer than Each Laboratory Sieve (Square Openings), weight percent														
		4-in. (100-mm)	3½-in. (90-mm)	3-in. (75-mm)	2½-in. (63-mm)	2-in. (50-mm)	1½-in. (37.5-mm)	1-in. (25.0-mm)	¾-in. (19.0-mm)	½-in. (12.5-mm)	⅜-in. (9.5-mm)	No. 4 (4.75-mm)	No. 8 (2.36-mm)	No. 16 (1.18-mm)	No. 50 (300-µm)	No. 100 (150-µm)
1	3½ to 1½-in. (90 to 37.5-mm)	100	90 to 100	...	25 to 60	...	0 to 15	...	0 to 5	...	...	...	...	...	...	...
2	2½ to 1½-in. (63 to 37.5-mm)	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...	...
24	2½ to ¾-in. (63 to 19.0-mm)	...	...	100	90 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...	...	...	...	...
3	2 to 1-in. (50 to 25.0-mm)	...	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...
357	2-in. to No. 4 (50 to 4.75-mm)	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	...	0 to 5	...	...	...	...
4	1½ to ¾-in. (37.5 to 19.0-mm)	...	...	...	...	100	90 to 100	20 to 55	0 to 15	...	0 to 5	...	...	...	...	...
467	1½-in. to No. 4 (37.5 to 4.75-mm)	...	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	0 to 5	...	...	...	...
5	1 to ½-in. (25.0 to 12.5-mm)	...	...	...	...	...	100	90 to 100	20 to 55	0 to 10	0 to 5	...	...	...	...	...
56	1 to ⅜-in. (25.0 to 9.5-mm)	...	...	...	...	...	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	...	...	...	...
57	1-in. to No. 4 (25.0 to 4.75-mm)	...	...	...	...	...	100	95 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...	...
6	¾ to ⅜-in. (19.0 to 9.5-mm)	...	...	...	...	...	...	100	90 to 100	20 to 55	0 to 15	0 to 5	...	...	...	...
67	¾-in. to No. 4 (19.0 to 4.75-mm)	...	...	...	...	...	...	100	90 to 100	...	20 to 55	0 to 10	0 to 5	...	...	...
68	¾-in. to No. 8 (19.0 to 2.36-mm)	...	...	...	...	...	...	100	90 to 100	...	30 to 65	5 to 25	0 to 10	0 to 5	...	...
7	½-in. to No. 4 (12.5 to 4.75-mm)	...	...	...	...	...	...	...	100	90 to 100	40 to 70	0 to 15	0 to 5	...	...	...
78	½-in. to No. 8 (12.5 to 2.36-mm)	...	...	...	...	...	...	...	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	...	...
8	⅜-in. to No. 8 (9.5 to 2.36-mm)	...	...	...	...	...	...	...	...	100	85 to 100	10 to 30	0 to 10	0 to 5	...	...
89	⅜-in. to No. 16 (9.5 to 1.18-mm)	...	...	...	...	...	...	...	...	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	...
9	No. 4 to No. 16 (4.75 to 1.18-mm)	...	...	...	...	...	...	...	...	...	100	85 to 100	10 to 40	0 to 10	0 to 5	...
10	No. 4 to 0[A] (4.75-mm)	...	...	...	...	...	...	...	...	...	100	85 to 100	...	...	...	10 to 30

[A] Screenings.

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

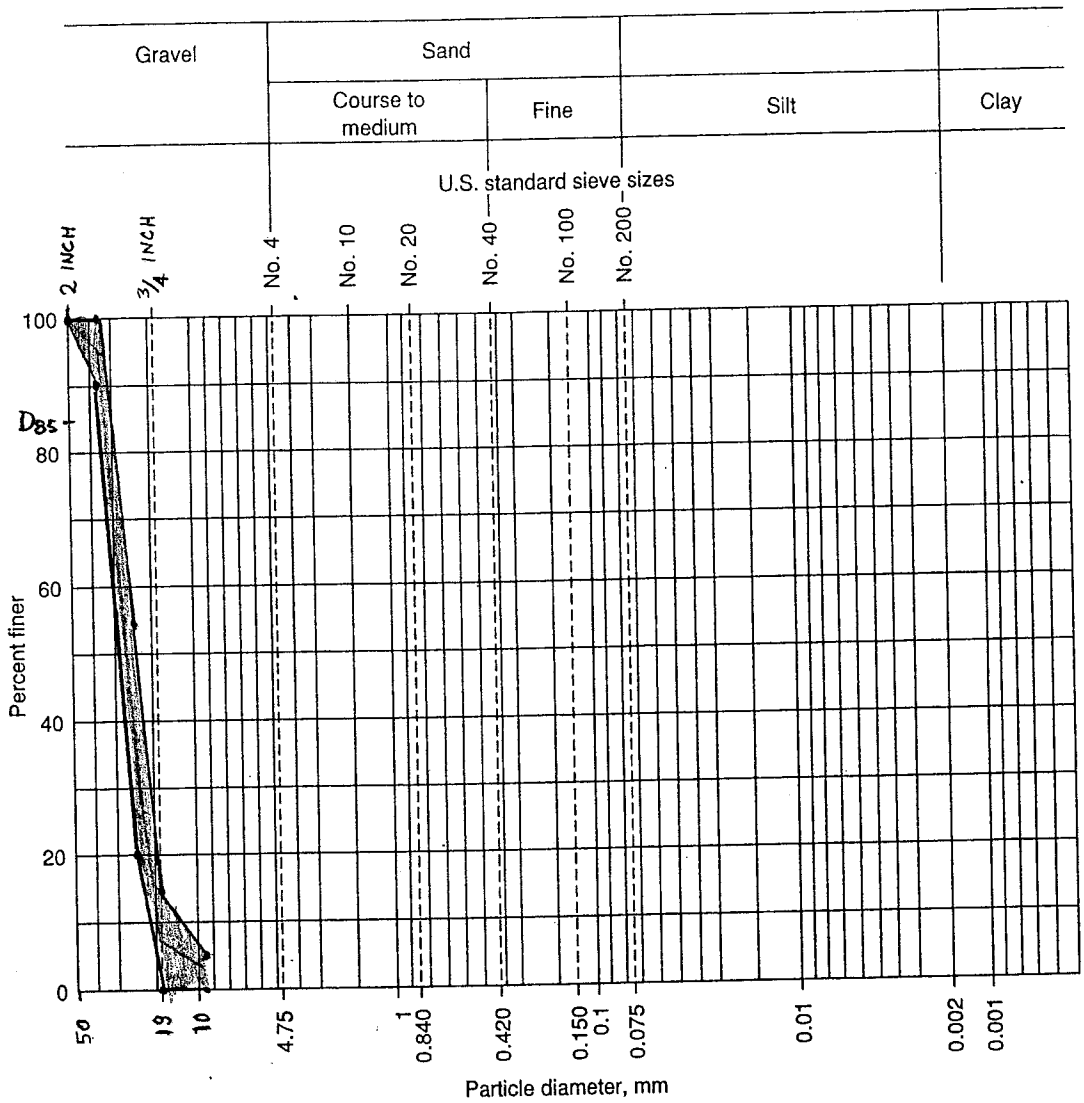
GRAIN SIZE DISTRIBUTION

Data Sheet 5*b*

Project No. 4 STONE PER ASTM D448 Job No. _____

Location of Project _____ Boring No. _____ Sample No. _____

Tested By _____ Date of Testing _____



Visual soil description _____

Soil classification _____

System _____

$D_{85} \approx 32\,mm = \underline{1.26\ IN}$

ATTACHMENT 9

Pipe Structural Stability Calculations

A Gupta
3/29/07

Geosyntec consultants

Written by: JUAN QUIROZ Date: 16/03/07 Reviewed by: _____ Date: __/__/__

Client: SARASOTA CO Project: FLSC Project/Proposal No. FL1109 Task No. _____

EFFECT OF PERFORATIONS ON PIPE STRUCTURAL STABILITY

REFERENCE: SHARMA, H.D. AND LEWIS, S.P. WASTE CONTAINMENT SYSTEMS, WASTE STABILIZATION AND LANDFILLS: DESIGN AND EVALUATION. JOHN WILEY AND SONS, INC, NEW YORK, 1994.

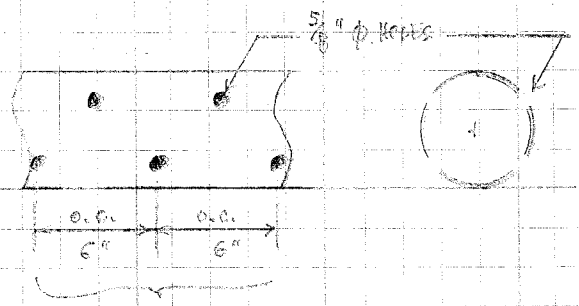
WASHINGTON STATE DEPARTMENT OF ECOLOGY. SOLID WASTE LANDFILL DESIGN MANUAL. PUBLICATION 87-13, 1987.

CONSERVATIVE APPROACH: INCREASE LOAD IN PROPORTION TO THE EFFECT OF THE PERFORATIONS (I.E., REDUCTION IN EFFECTIVE LENGTH OF PIPE).

$$\text{DESIGN LOAD} = \text{ACTUAL LOAD} \left(\frac{L}{L - L_p} \right)$$ [WASHINGTON STATE, 1987]

WHERE L = UNIT LENGTH OF PIPE

L_p = CUMMULATIVE LENGTH OF PERFORATIONS PER UNIT LENGTH OF PIPE



TYPICAL PERFORATION DETAIL
(SEE SHEET 13 OF PERMIT DWGS)

UNIT LENGTH = 1 FT = 12"
(at 2 HOLES/FT)

$L = 12''$

$L_p = \underbrace{2}_{\text{HOLES/FT}} \times \frac{5}{8}'' = \frac{10}{8}'' = 1.25''$

$$\left(\frac{L}{L - L_p} \right) = \frac{12''}{12'' - 1.25''} = \underline{\underline{1.12}}$$

∴ INCREASE ACTUAL LOAD BY 1.12, SEE ATTACHED CALCULATIONS.

PIPE STRUCTURAL STABILITY OF THE 4-INCH PERFORATED PIPES

This worksheet was prepared to show the pipe stability calculations for the perforated pipes within the FLSCs. The methodology of the stability calculations presented herein were presented in Appendix D of the engineering report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006.

PIPE DATA (Chevron Phillips Chemical Company, 2001)

Pipe Diameter in	SDR	OD in	Average ID in	Min. Wall Thickness in	Area with ID in^2
4	11	4.500	3.633	0.409	10.37

OD= outside diameter

ID= inside diameter

CALCULATION OF APPLIED STRESS WITHIN FLSCs (FOR PERFORATED PIPES)

The assumed overburden stress consists of approximately 2 ft of drainage gravel plus approximately 5 ft of liquid with an assumed density of 62.4 pcf.

$$\sigma_{ov} = \sum \gamma_{ov} D_{ov}$$ (ASCE, 1979)

γ_{ov}= unit weight of the overburden material (assume 135 pcf for gravel and 62.4 pcf for the liquid)

D_{ov}= thickness of the overburden materials

σ_{ov}= stress on the pipe due to FLSC loading

RESULTS

4-in Pipe

γ_{ov}=	135	pcf (gravel)
	62.4	pcf (liquid)
D_{ov}=	2	ft (gravel)
	5	ft (fluid)
L/(L-Lp)=	1.12	
σ_{ov}=	652	psf
	4.53	**psi**

Per Sharma and Lewis (1994) and Washington State (1987), see attached calculations.

WALL CRUSHING

$$\sigma_{crush} = \frac{2\sigma_y}{(SDR-1)}$$ (Phillips 66, 1991)

σ_{crush}= maximum applied stress which may be withstood by the pipe

σ_y= compressive strength of the pipe

SDR= standard dimension ratio of the pipe

RESULTS

4-in Pipe

σ_y=	1500	psi
SDR=	11	
σ_{crush}=	**300**	psi
FS=	**66.3**	

WALL BUCKLING

$$\sigma_{buckle} = 1.2\left[\frac{E'E}{SDR^3}\right]^{1/2}$$ (Phillips 66, 1991)

SDR= standard dimension ratio of the pipe

σ_{buckle}= critical buckling soil pressure at the top of the pipe

E'= modulus of soil reaction

E= modulus of elasticity of the pipe material (Plastic Pipe Institute, 1993)

$$S_A = \frac{(SDR - 1)\sigma_{vo}}{2}$$ (Phillips 66, 1991)

σ_{vo}= total external pressure on top of the pipe
SDR= standard dimension ratio of the pipe
S_A= tensile stress intensity

4-in Pipe

σ_{ov}=	5	psi
SDR=	11	
S_A=	23	psi
E =	37000	psi

$$E' = k * M_s$$ (Selig, 1990)

E'= modulus of soil reaction for the pipe bedding material
k= empirical factor assumed as 1.5 (this factor varies from 0.7 to 2.3) (Selig, 1990)
M_s= constrained modulus

$$M_s = \frac{E_s(1-\nu)}{(1+\nu)(1-2\nu)}$$ (Selig, 1990)

E_s= Young's modulus obtained from Selig (1990) for the general fill material (see properties for SM material at 90% standard Proctor)
v = Poisson's ratio (Selig, 1990)

4-in Pipe

E_s=	2400	psi
v=	0.36	
M_s=	4034	psi
k=	1.5	
E'=	6050	psi

RESULTS FOR WALL BUCKLING

4-in Pipe

E'=	6050	psi
E=	37000	psi
SDR=	11	psi
$\sigma_{buckling}$=	500	psi
FS=	110.5	

RING DEFLECTION

Modified Iowa Equation:

$$\Delta X = \frac{D_L K W_c}{\frac{EI}{r^3} + 0.061\ E'}$$ (Koerner 1998)

$$Ring\ \ Deflection = \frac{\Delta Y}{D_{od}}$$

ΔX= horizontal deflection, this is the horizontal increase in diameter
D_L= deflection lag factor which varies from 1 to 1.5 (Koerner, 1998).
K= bedding constant which varies from 0.083 to 0.11 (Wilson-Fahmy and Koerner, 1994).
W_c= Marston's prism load per unit length of pipe (overburden stress times the pipe outside diameter)
E= modulus of elasticity of the pipe material (obtained in calculations for wall buckling)
I= moment of inertia of the pipe wall per unit length = $t^3/12$
r= mean radius of the pipe = $(D_{od} - t)/2$
D_{od}= pipe outside diameter
t= wall thickness of the pipe
E'= modulus of soil reaction for the pipe bedding material (obtained in calculations for wall buckling)

RESULTS FOR RING DEFLECTION

4-in Pipe

D_L=	1.25	
K=	0.11	
D_{od}=	4.500	psi
W_c=	20	lb/in
E=	37000	psi
t=	0.409	in
I=	0.006	in^3
r=	2.05	in
E'=	6050	psi
ΔX=	0.007	in
Ring Deflection=	0.16%	< 3%

BENDING STRAIN

$$\varepsilon_b = f_d \frac{t * \Delta y}{D^2} * 100$$ (Mosher, 1990)

ε_b=	bending strain
f_d=	deformation shape factor equal to 6 for design
t	minimum wall thickness
Δ_y=	vertical deflection assumed approximately the same as ΔX obtained with the Modified Iowa Equation
D=	pipe inside diameter
allowable ε_b=	allowable bending strain of 4.2 % (Chevron Chemical Company, 1994)

4-in Pipe

f_d=	6	
t	0.409	in
Δ_y=	0.007	in
D=	3.633	in
ε_b=	0.13	%
allowable ε_b=	4.2	%

PIPE STRUCTURAL STABILITY OF THE 6-INCH PERFORATED PIPES

This worksheet was prepared to show the pipe stability calculations for the perforated pipes within the FLSCs. The methodology of the stability calculations presented herein were presented in Appendix D of the engineering report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006.

PIPE DATA (Chevron Phillips Chemical Company, 2001)

Pipe Diameter in	SDR	OD in	Average ID in	Min. Wall Thickness in	Area with ID in^2
6	11	6.625	5.349	0.602	22.47

OD= outside diameter

ID= inside diameter

CALCULATION OF APPLIED STRESS WITHIN FLSCs (FOR PERFORATED PIPES)

The assumed overburden stress consists of approximately 2 ft of drainage gravel plus approximately 5 ft of liquid with an assumed density of 62.4 pcf.

$$\sigma_{ov} = \sum \gamma_{ov} D_{ov}$$ (ASCE, 1979)

γ_{ov}= unit weight of the overburden material (assume 135 pcf for gravel and 62.4 pcf for the liquid)

D_{ov}= thickness of the overburden materials

σ_{ov}= stress on the pipe due to FLSC loading

RESULTS

6-in Pipe

γ_{ov}=	135	pcf (gravel)
	62.4	pcf (liquid)
D_{ov}=	2	ft (gravel)
	5	ft (fluid)
L/(L-Lp)=	1.12	
σ_{ov}=	652	psf
	4.53	**psi**

Per Sharma and Lewis (1994) and Washington State (1987), see attached calculations.

WALL CRUSHING

$$\sigma_{crush} = \frac{2\sigma_y}{(SDR-1)}$$ (Phillips 66, 1991)

σ_{crush}= maximum applied stress which may be withstood by the pipe

σ_y= compressive strength of the pipe

SDR= standard dimension ratio of the pipe

RESULTS

6-in Pipe

σ_y=	1500	psi
SDR=	11	
σ_{crush}=	**300**	psi
FS=	**66.3**	

WALL BUCKLING

$$\sigma_{buckle} = 1.2\left[\frac{E'E}{SDR^3}\right]^{1/2}$$ (Phillips 66, 1991)

SDR= standard dimension ratio of the pipe

σ_{buckle}= critical buckling soil pressure at the top of the pipe

E'= modulus of soil reaction

E= modulus of elasticity of the pipe material (Plastic Pipe Institute, 1993)

$$S_A = \frac{(SDR - 1)\sigma_{vo}}{2}$$ (Phillips 66, 1991)

σ_{vo}= total external pressure on top of the pipe
SDR= standard dimension ratio of the pipe
S_A= tensile stress intensity

6-in Pipe

σ_{ov}=	5	psi
SDR=	11	
S_A=	23	psi
E =	37000	psi

$$E' = k * M_s$$ (Selig, 1990)

E'= modulus of soil reaction for the pipe bedding material
k= empirical factor assumed as 1.5 (this factor varies from 0.7 to 2.3) (Selig, 1990)
M_s= constrained modulus

$$M_s = \frac{E_s(1-\nu)}{(1+\nu)(1-2\nu)}$$ (Selig, 1990)

E_s= Young's modulus obtained from Selig (1990) for the general fill material (see properties for SM material at 90% standard Proctor)
v = Poisson's ratio (Selig, 1990)

6-in Pipe

E_s=	2400	psi
v=	0.36	
M_s=	4034	psi
k=	1.5	
E'=	6050	psi

RESULTS FOR WALL BUCKLING

6-in Pipe

E'=	6050	psi
E=	37000	psi
SDR=	11	psi
$\sigma_{buckling}$=	500	psi
FS=	110.5	

RING DEFLECTION

Modified Iowa Equation:

$$\Delta X = \frac{D_L K W_c}{\frac{EI}{r^3} + 0.061\ E'}$$ (Koerner 1998)

$$Ring\ \ Deflection = \frac{\Delta Y}{D_{od}}$$

ΔX= horizontal deflection, this is the horizontal increase in diameter
D_L= deflection lag factor which varies from 1 to 1.5 (Koerner, 1998).
K= bedding constant which varies from 0.083 to 0.11 (Wilson-Fahmy and Koerner, 1994).
W_c= Marston's prism load per unit length of pipe (overburden stress times the pipe outside diameter)
E= modulus of elasticity of the pipe material (obtained in calculations for wall buckling)
I= moment of inertia of the pipe wall per unit length = $t^3/12$
r= mean radius of the pipe = $(D_{od} - t)/2$
D_{od}= pipe outside diameter
t= wall thickness of the pipe
E'= modulus of soil reaction for the pipe bedding material (obtained in calculations for wall buckling)

RESULTS FOR RING DEFLECTION

6-in Pipe

D_L=	1.25	
K=	0.11	
D_{od}=	6.625	psi
W_c=	30	lb/in
E=	37000	psi
t=	0.602	in
I=	0.018	in^3
r=	3.01	in
E'=	6050	psi
ΔX=	0.010	in
Ring Deflection=	0.16%	< 3%

BENDING STRAIN

$$\varepsilon_b = f_d \frac{t * \Delta y}{D^2} * 100$$ (Mosher, 1990)

ε_b=	bending strain
f_d=	deformation shape factor equal to 6 for design
t	minimum wall thickness
Δ_y=	vertical deflection assumed approximately the same as ΔX obtained with the Modified Iowa Equation
D=	pipe inside diameter
allowable ε_b=	allowable bending strain of 4.2 % (Chevron Chemical Company, 1994)

6-in Pipe

f_d=	6	
t	0.602	in
Δ_y=	0.010	in
D=	5.349	in
ε_b=	0.13	%
allowable ε_b=	4.2	%

PIPE STRUCTURAL STABILITY OF THE 18-INCH PERFORATED PIPES

This worksheet was prepared to show the pipe stability calculations for the perforated pipes within the FLSCs. The methodology of the stability calculations presented herein were presented in Appendix D of the engineering report titled "Application for a Permit to Construct Flexible Leachate Storage Containers at Central County Solid Waste Disposal Complex," dated November 2006.

PIPE DATA (Chevron Phillips Chemical Company, 2001)

Pipe Diameter in	SDR	OD in	Average ID in	Min. Wall Thickness in	Area with ID in^2
18	11	18.000	14.532	1.636	165.86

OD= outside diameter

ID= inside diameter

CALCULATION OF APPLIED STRESS WITHIN FLSCs (FOR PERFORATED PIPES)

The assumed overburden stress consists of approximately 2 ft of drainage gravel plus approximately 5 ft of liquid with an assumed density of 62.4 pcf.

$$\sigma_{ov} = \sum \gamma_{ov} D_{ov}$$ (ASCE, 1979)

γ_{ov}= unit weight of the overburden material (assume 135 pcf for gravel and 62.4 pcf for the liquid)

D_{ov}= thickness of the overburden materials

σ_{ov}= stress on the pipe due to FLSC loading

RESULTS

18-in Pipe

γ_{ov}=	135	pcf (gravel)
	62.4	pcf (liquid)
D_{ov}=	2	ft (gravel)
	5	ft (fluid)
L/(L-Lp)=	1.12	
σ_{ov}=	652	psf
	4.53	**psi**

Per Sharma and Lewis (1994) and Washington State (1987), see attached calculations.

WALL CRUSHING

$$\sigma_{crush} = \frac{2\sigma_y}{(SDR-1)}$$ (Phillips 66, 1991)

σ_{crush}= maximum applied stress which may be withstood by the pipe

σ_y= compressive strength of the pipe

SDR= standard dimension ratio of the pipe

RESULTS

18-in Pipe

σ_y=	1500	psi
SDR=	11	
σ_{crush}=	**300**	psi
FS=	**66.3**	

WALL BUCKLING

$$\sigma_{buckle} = 1.2\left[\frac{E'E}{SDR^3}\right]^{1/2}$$ (Phillips 66, 1991)

SDR= standard dimension ratio of the pipe

σ_{buckle}= critical buckling soil pressure at the top of the pipe

E'= modulus of soil reaction

E= modulus of elasticity of the pipe material (Plastic Pipe Institute, 1993)

$$S_A = \frac{(SDR - 1)\sigma_{vo}}{2}$$ (Phillips 66, 1991)

σ_{vo}= total external pressure on top of the pipe
SDR= standard dimension ratio of the pipe
S_A= tensile stress intensity

18-in Pipe

σ_{ov}=	5	psi
SDR=	11	
S_A=	23	psi
E =	37000	psi

$$E' = k * M_s$$ (Selig, 1990)

E'= modulus of soil reaction for the pipe bedding material
k= empirical factor assumed as 1.5 (this factor varies from 0.7 to 2.3) (Selig, 1990)
M_s= constrained modulus

$$M_s = \frac{E_s(1-\nu)}{(1+\nu)(1-2\nu)}$$ (Selig, 1990)

E_s= Young's modulus obtained from Selig (1990) for the general fill material (see properties for SM material at 90% standard Proctor)
v = Poisson's ratio (Selig, 1990)

18-in Pipe

E_s=	2400	psi
v=	0.36	
M_s=	4034	psi
k=	1.5	
E'=	6050	psi

RESULTS FOR WALL BUCKLING

18-in Pipe

E'=	6050	psi
E=	37000	psi
SDR=	11	psi
$\sigma_{buckling}$=	500	psi
FS=	110.5	

RING DEFLECTION

Modified Iowa Equation:

$$\Delta X = \frac{D_L K W_c}{\frac{EI}{r^3} + 0.061\ E'}$$ (Koerner 1998)

$$Ring\ \ Deflection = \frac{\Delta Y}{D_{od}}$$

ΔX= horizontal deflection, this is the horizontal increase in diameter
D_L= deflection lag factor which varies from 1 to 1.5 (Koerner, 1998).
K= bedding constant which varies from 0.083 to 0.11 (Wilson-Fahmy and Koerner, 1994).
W_c= Marston's prism load per unit length of pipe (overburden stress times the pipe outside diameter)
E= modulus of elasticity of the pipe material (obtained in calculations for wall buckling)
I= moment of inertia of the pipe wall per unit length = $t^3/12$
r= mean radius of the pipe = $(D_{od} - t)/2$
D_{od}= pipe outside diameter
t= wall thickness of the pipe
E'= modulus of soil reaction for the pipe bedding material (obtained in calculations for wall buckling)

RESULTS FOR RING DEFLECTION

18-in Pipe

D_L=	1.25	
K=	0.11	
D_{od}=	18.000	psi
W_c=	81	lb/in
E=	37000	psi
t=	1.636	in
I=	0.365	in^3
r=	8.18	in
E'=	6050	psi
ΔX=	0.028	in
Ring Deflection=	0.16%	< 3%

BENDING STRAIN

$$\varepsilon_b = f_d \frac{t * \Delta y}{D^2} * 100$$ (Mosher, 1990)

ε_b=	bending strain
f_d=	deformation shape factor equal to 6 for design
t	minimum wall thickness
Δ_y=	vertical deflection assumed approximately the same as ΔX obtained with the Modified Iowa Equation
D=	pipe inside diameter
allowable ε_b=	allowable bending strain of 4.2 % (Chevron Chemical Company, 1994)

18-in Pipe

f_d=	6	
t	1.636	in
Δ_y=	0.028	in
D=	14.532	in
ε_b=	0.13	%
allowable ε_b=	4.2	%

COPY OF APPROPRIATE SECTIONS FOR:

ASCE, "Design and Construction of Sanitary and Storm Sewers", Manual and Report on Engineering Practice No. 37, American Society of Civil Engineers, New York, NY, Printed 1969, Reprinted 1979.

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CHAPTER 9. STRUCTURAL REQUIREMENTS

A. INTRODUCTION

The structural design of a sewer requires that the supporting strength of the conduit as installed, divided by a suitable factor of safety, must equal or exceed the loads imposed on it by the weight of earth and any superimposed loads.

This chapter presents generally accepted criteria and methods for determining loads and supporting strength, as well as procedures for combining these elements with the application of a factor of safety to produce a safe and economical design.

Methods are presented for estimating probable maximum loads due to gravity earth forces and for both static and moving superimposed loads. Where so noted, the methods apply to rigid and flexible conduits in the three most common conditions of installation: in a trench in natural ground; in an embankment; and in a tunnel.

The supporting strength of buried conduits is a function of installation conditions as well as the strength of the pipe itself. This chapter presents procedures for determining the field or installed supporting strength of rigid sewer pipe based on its established relationship to the laboratory test strength. It also presents a brief discussion of the method of determining the safe supporting strength of flexible pipe based on a semi-empirical equation for deflection.

Since installation conditions have such an important effect on both load and supporting strength, a satisfactory sewer construction project requires attainment of design conditions in the field. Therefore, this chapter also includes a section on recommendations for construction and field observations to achieve this goal.

This chapter does not include information on reinforced concrete design or design of the conduit section. Reference should be made to standard textbooks and to ASTM specifications or industry handbooks for such design data.

B. LOADS ON SEWERS DUE TO GRAVITY EARTH FORCES

1. General Method

Marston (1) (2) developed methods for determining the vertical load on buried conduits due to gravity earth forces in all of the most commonly encountered construction conditions. His methods are based on both theory and experiment and have achieved acceptance as being the most useful and reliable. In general, the theory states that the load on a buried conduit is equal to the weight of the prism of earth directly over

it, called the interior prism, plus or minus the frictional shearing forces transferred to that prism by the adjacent prisms of earth—the magnitude and direction of these frictional forces being a function of the relative settlement between the interior and adjacent earth prisms. The theory makes the following assumptions:

(*a*) The calculated load is the load which will develop when ultimate settlement has taken place.
(*b*) The magnitude of the lateral pressures which induce the shearing forces between the interior and adjacent earth prisms is computed in accordance with Rankine's theory.
(*c*) Cohesion is negligible except for tunnel conditions.

The general form of Marston's equation is

$$W = CwB^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 1$$

in which W is the vertical load per unit length acting on the conduit due to gravity earth loads; w is the unit weight of earth per unit volume; B is the trench width or conduit width, depending on installation conditions; and C is a dimensionless coefficient that measures the effect of:

(*a*) Ratio of the height of fill to width of trench or conduit,
(*b*) Shearing forces between interior and adjacent earth prisms, and
(*c*) Direction and amount of relative settlement between interior and adjacent earth prisms for embankment conditions.

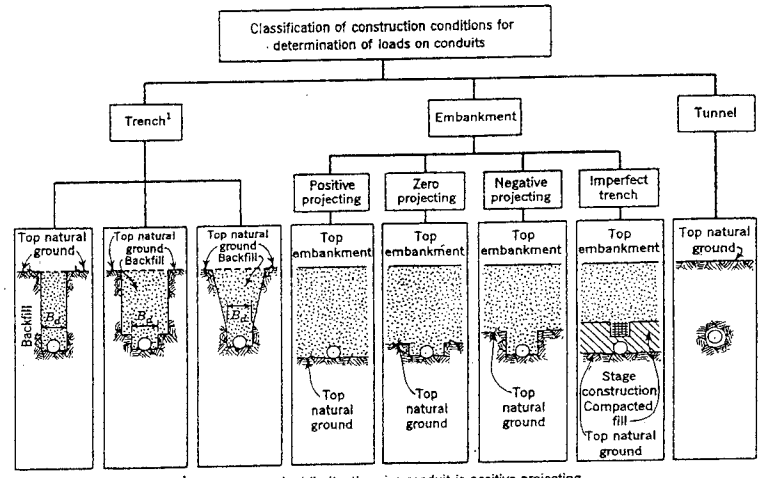
2. Types of Loading Conditions

Although the general form of Marston's equation includes all the factors necessary to analyze all types of installation conditions, it is convenient to classify these conditions, write a specialized form of the equation, and prepare separate graphs and tables of coefficients for each.

The accepted system of classification is shown diagrammatically in Figure 43 and is described briefly below:

Trench conditions are defined as those in which the conduit is installed in a relatively narrow trench cut in undisturbed ground and covered with earth backfill to the original ground surface.

Embankment conditions are defined as those in which the conduit is covered with fill above the original ground surface or when a trench in undisturbed ground is so wide that trench wall friction does not affect the load on the pipe. The embankment classification is further subdivided into two major subclassifications—positive projecting and negative projecting. Conduits are defined as positive projecting when the top of the conduit is above the adjacent original ground surface. Negative projecting conduits are those installed with the top of conduit below the adjacent original ground surface in a trench which is narrow with respect to the size of pipe and depth of cover (Figure 43) and when the native material is of sufficient strength that the trench shape can be maintained dependably during the placing of the embankment.



[1] B_d within required limit, otherwise conduit is positive projecting

FIGURE 43.—Classification of construction conditions.

A special case, called the imperfect trench condition, may be employed to minimize the load on a conduit under embankments of unusual height.

3. Loads for Trench Conditions

Sewers usually are constructed in ditches or trenches which are excavated in natural or undisturbed soil, and then covered by refilling the trench to the original ground line. This construction procedure often is referred to as "cut and cover," or "cut and fill."

(*a*) **Load-Producing Forces.**—The vertical load to which a sewer pipe is subjected, when so constructed, is the resultant of two major forces: the first is the weight of the prism of soil within the trench and above the top of the pipe; and the second is the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench.

The backfill soil has a tendency to settle in relation to the undisturbed soil in which the trench is excavated. This downward movement or tendency for movement induces upward shearing forces which support a part of the weight of the backfill. Thus, the resultant load on the horizontal plane at the top of the pipe within the trench is equal to the weight of the backfill minus these upward shearing forces, as indicated in Figure 44.

(*b*) **Marston's Formula.**—Marston's formula for loads on rigid conduits in trench conditions is

$$W_c = C_d w B_d^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2$$

in which W_c is the load on the pipe in lb/ft (kg/m); w is the unit weight of backfill soil in lb/cu ft (kg/cu m); B_d is the width of trench at the top of the pipe in ft (m); C_d is a dimensionless load coefficient which is a

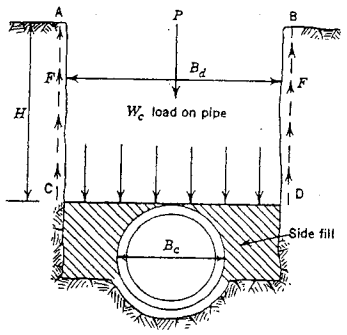


FIGURE 44.—Load-producing forces: P = weight of backfill $ABCD$; F = upward shearing forces on AC and BD; and $W_c = P - 2F$.

function of the ratio of height of fill to width of trench and of the friction coefficient between the backfill and the sides of the trench. The load coefficient, C_d, is computed as follows:

$$C_d = \frac{1 - e^{-2k\mu' H/B_d}}{2k\mu'} \quad \ldots\ldots 3$$

in which e is the base of natural logarithms; k = Rankine's ratio of lateral pressure to vertical pressure $= \frac{\sqrt{\mu^2+1}-\mu}{\sqrt{\mu^2+1}+\mu} = \frac{1-\sin\phi}{1+\sin\phi}$ 4

in which $\mu = \tan\phi$ = the coefficient of internal friction of backfill material; $\mu' = \tan\phi'$ = the coefficient of friction between backfill material and sides of trench (μ' may be equal to or less than μ, but never greater than μ); and H is the height of fill above top of pipe in ft (m). The value of C_d for various ratios of H/B_d and various types of soil backfill may be obtained from Figure 45.

The trench load formula, Equation 2, gives the total vertical load on a horizontal plane at the top of the pipe. If the pipe is rigid it will carry practically all this load. If the pipe is flexible and the soil at the sides is compacted to the extent that it will deform under vertical load the same amount as the pipe itself, the side fills may be expected to carry their proportional share of the total load. Under these circumstances the trench load formula may be modified to

$$W_c = C_d w B_c B_d \quad \ldots\ldots 5$$

in which B_c is the outside width of pipe in ft (m).

The term "side fill" refers to the soil backfill which is placed between the sides of a pipe and the sides of the trench. The character of this material and the manner of its placement have two important influences on the structural behavior of a pipe.

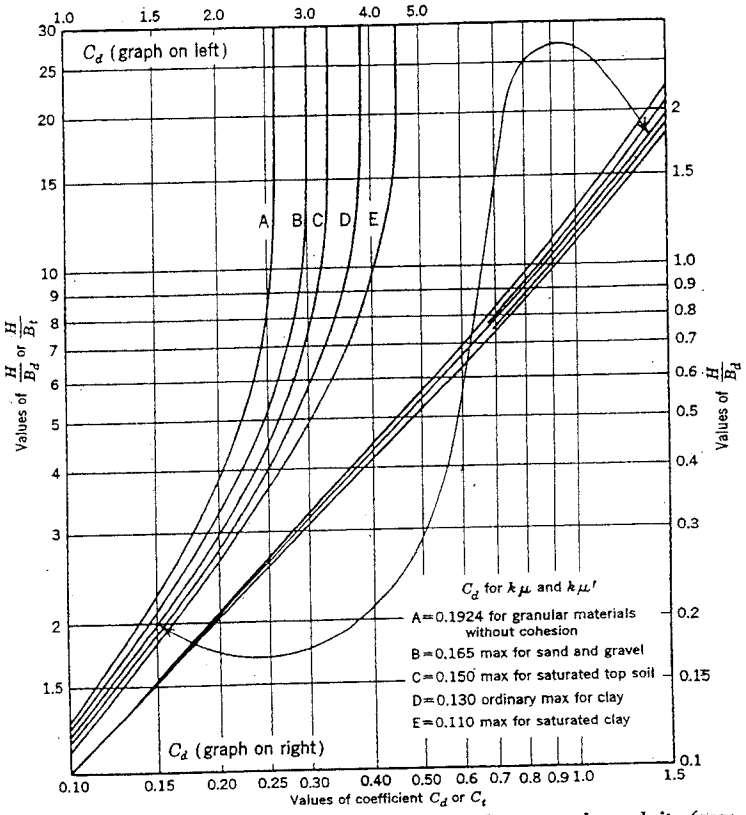


FIGURE 45.—Computation diagram for earth loads on trench conduits (completely buried in trenches).

First, the side fill may carry a part of the total vertical load on the horizontal plane at the elevation of the top of the pipe. If the side fill is relatively yielding as compared with the stiffness of the pipe, practically all the load will be carried by the pipe. If the side fill is of about the same stiffness as the pipe to the extent that it will deform under vertical load the same amount as the pipe itself (a condition which may exist when the pipe is flexible and the side fill is tamped), it may carry its proportional share of the total load on the plane.

Second, the side fill plays an important role in helping the pipe carry vertical load. Every pound of pressure which can be brought to bear against the sides of an elastic ring increases the ability of the ring to carry vertical load by nearly the same amount. This fact points up the desirability of tamping the side fills in the case of rigid pipe, and the absolute necessity in the case of flexible pipe. However, caution should

be exercised when tamping the soil at the sides of non-reinforced rigid pipe to prevent damage to the pipe.

(c) Influence of Width of Trench.—Examination of Equation 2 indicates the important influence width of the trench exerts on the load as long as the trench condition formula applies. This influence has been verified by extensive experimental evidence. These experiments also have indicated that the width of trench at the top of the pipe is the controlling factor.

Depending on height of fill above the pipe, the width of trench below the top of the pipe must not be permitted to exceed the safe limit for the strength of pipe and class of bedding used. The minimum width must be consistent with the provision of sufficient working space at the sides of the pipe to assemble joints properly, to insert and strip forms, and to compact backfill. The designer must allow reasonable tolerance in width for variations in field conditions and accepted construction practice.

The position of the lower wale usually will determine the proper width of trench from face-to-face of sheeting, where sheeting and bracing are required. A working-room allowance of 12 in. (30 cm) from each side of the pipe, pipe cradle, or monolithic conduit to the face of the sheeting is a workable minimum for small- and medium-sized pipe for trenches up to about 14 ft (4 m) deep.

At any given depth and for any given conduit size there is a certain limiting value to the width of trench beyond which no additional load is transmitted to the conduit. This limiting value is called the "transition width" (3). There are sufficient experimental data at hand to show that it is safe to calculate the imposed load by means of the trench-conduit formula (Equation 2) for all widths of trench less than that which gives a load equal to the load calculated by the projecting-conduit formula (Equation 6). In other words, as the width of the trench increases, other factors remaining constant, the load on a rigid conduit increases in accordance with the theory for a trench conduit until it equals the load determined by the theory for a projecting conduit. The width of trench at which this transition occurs may be determined from the diagram in Figure 46. [The term, $r_{sd}p$, is defined in Section B4b(2).]

It is advisable in the structural design of conduits to evaluate the effect of the transition width on both the design criteria and the construction latitude. A contractor, for instance, may wish to place well points for drainage in the trench. If this requires a wider trench than usual, a stronger pipe or higher class of bedding may be necessary.

It may be economical and proper to excavate the trench with sloping sides in undeveloped areas where no inconvenience to the public or danger to property, buildings, subsurface structures, pavements, etc., will result. A subtrench (Figure 47) may be employed in such cases to minimize the load on the pipe.

When sheeting of the trench at the pipe is necessary, it should extend about 1.5 ft (45 cm) above the top of the conduit. It is recommended that this sheeting and bracing be left in place. The load on the conduit should

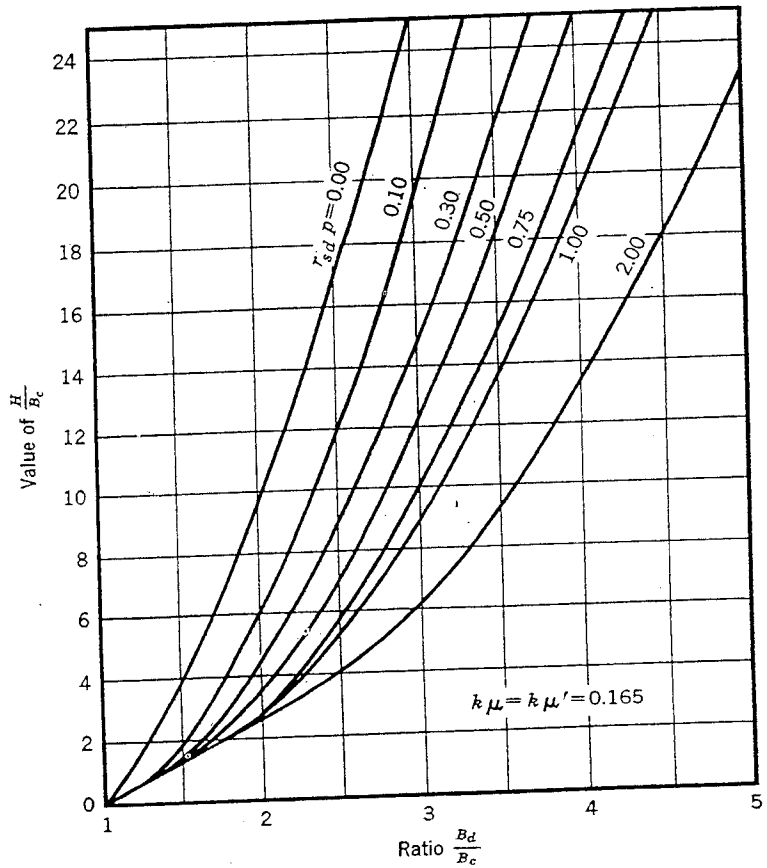


FIGURE 46.—Values of B_d/B_c at which the trench conduit and projecting conduit load formulas give equal loads (3).

be computed for a width of trench B_d equal to the distance to the outside of the sheeting if it is to be removed, or to the inside if left in place.

If a shield is employed in pipe-laying operations, the width of the shield controls the width of the trench at the top of the pipe. This width should be the width factor used in computing loads on the pipe.

Conduits which are to be constructed in sloping sided trenches with the slopes extending to the invert, or to any plane above the invert but below the top of the structure, should be designed for loads computed by using the actual width of the trench at the top of the pipe, or by the projecting-conduit formula (Equation 6), whichever gives the least load on the pipe.

If for any reason the trench becomes wider than that specified and for

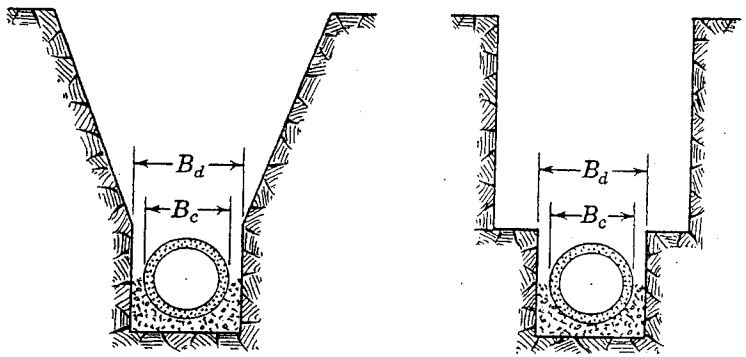


FIGURE 47.—Examples of subtrench.

which the pipe was designed, the load on the pipe should be checked and a stronger pipe furnished or higher class bedding used, if necessary.

Example 1. Determine the load on a 24-in. diam rigid pipe under 14 ft of cover in trench conditions.

Assume that the pipe wall thickness is 2 in.; $B_c=24+4=28$ in.$=2.33$ ft; $B_d=2.33+2.00=4.33$ ft; and $w=120$ lb/cu ft for saturated top soil. Then $H/B_d=14/4.33=3.24$; C_d (from Figure 45) $=2.1$; and $W_c=2.1\times120\times4.33^2=4{,}720$ lb/ft (7,030 kg/m).

Example 2. Determine the load on the same size conduit laid on a concrete cradle and with trench sheeting to be removed.

Assume that the wall thickness is 2 in.; the cradle projection outside of the pipe is 8 in. (4 in. on each side); and the maximum clearance between cradle and outside of sheeting is 14 in. Then $B_d=24+(2\times2$ in.$)+8+(2\times14)=64$ in.$=5.33$ ft.

As this seems to be an extremely wide trench, a check should be made on the transition width of the trench; $B_c=2.33$ ft; $H=14$ ft; $r_{sd}p=0.5$; and $H/B_c=14/2.33=6.0$.

From Figure 46, $B_d/B_c=2.39$ (the ratio of the width of the trench to the width of the conduit at which loads are equal by both ditch-conduit theory and projecting-conduit theory); $B_d=2.33\times2.39=5.80>5.33$; $H/B_d=14/5.33=2.61$; C_d (from Figure 45) $=1.85$; and $W_c=1.85\times120\times5.33^2=6{,}300$ lb/ft (9,350 kg/m).

Example 3. Determine the load on the same conduit if (rough) sheeting is left in place.

B_d becomes 4 in. less$=5.00$ ft; $H/B_d=14/5.00=2.8$; C_d (from Figure 45) $=1.92$; and $W_c=1.92\times120\times5.00^2=5{,}750$ lb/ft (8,570 kg/m).

Example 4. Determine the load on a 30-in. diam flexible conduit installed in a trench 4 ft 6 in. wide at a depth of 12 ft.

Assume the soil is clay weighing 120 lb/cu ft and that it will be well compacted at the sides of the pipe. Then $H=12$ ft; $B_d=4.5$ ft; $H/B_d=2.67$; $C_d=1.9$; and $W_c=1.9\times120\times4.5\times2.58=2{,}650$ lb/ft (3,950 kg/m).

(*d*) **Soil Characteristics—Trench Conditions.**—The load on a sewer pipe is influenced directly by the unit weight of the soil backfill. This value varies widely for different soils, from a minimum of about 100 lb/cu ft (1,600 kg/cu m) to a maximum of about 135 lb/cu ft (2,200 kg/cu m). The average maximum unit weight of the soil which will constitute the backfill over the pipe may be determined by density measurements in advance of the structural design of the pipe. A design value of not less than 120 or 125 lb/cu ft (1,900 or 2,000 kg/cu m) is recommended if such measurements are not made.

The load also is influenced by the coefficient of friction between the backfill and the sides of the trench and by the coefficient of internal friction of the backfill soil. Ordinarily these two values will be nearly the same and may be so considered for design purposes, as in Figure 45, but in special cases this may not be true. For example if the backfill is sharp sand and the sides of the trench are sheeted with finished lumber, μ may be substantially greater than μ'. Unless specific information to the contrary is available, values of the products $k\mu$ and $k\mu'$ may be assumed to be the same and equal to 0.130 (ordinary maximum for clay, Figure 45). If the backfill soil is a "slippery" clay and there is a possibility that it will become very wet shortly after being placed, $k\mu$ and $k\mu'$ equal to 0.110 (maximum for saturated clay, Figure 45) should be used.

4. Loads for Embankment Conditions

(*a*) **General.**—A sewer is described as a projecting conduit when it is installed in a wide trench or in such a manner that the top of the conduit is at or near the natural ground surface or the surface of thoroughly compacted soil and subsequently is covered with an embankment. If the top of the conduit projects some distance above the natural ground surface or if it is installed in a wide trench, it is a positive projecting conduit. There are, however, other methods of installing conduits under embankments which have the favorable effect of minimizing the load on the conduit. In these cases, the installation is classified as a negative projecting conduit or an imperfect trench conduit (Figure 43).

These variations of embankment conditions will be treated separately for convenience in computation.

(*b*) **Positive Projecting Conduits.**—The load on a positive projecting conduit is equal to the weight of the prism of soil directly above the structure, plus (or minus) vertical shearing forces which act on vertical planes extending upward into the embankment from the sides of the conduit. These vertical shearing forces ordinarily do not extend to the top

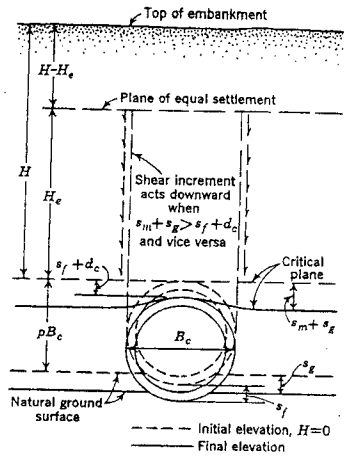


FIGURE 48.—Settlements that influence loads on positive projecting conduits: s_g = settlement of natural ground adjacent to conduit, s_m = compression of columns of soil of height pB_c, d_c = deflection of the conduit, and s_f = settlement of bottom of conduit.

of the embankment, but terminate in a horizontal plane at some elevation above the top of the conduit known as the "plane of equal settlement" as shown in Figure 48. The shear increment acts downward when $s_m+s_g>s_f+d_c$ and vice versa. In this expression s_m is the compression of the columns of soil of height pB_c; s_g is the settlement of the natural ground adjacent to the conduit; s_f is the settlement of the bottom of the conduit; and d_c is the deflection of the conduit.

1. Marston's Formula.—Marston's formula for loads on rigid and flexible positive projecting conduits is written

$$W_c=C_c w B^2_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 6$$

in which W_c is the load on the conduit in lb/ft (kg/m); w is the unit weight of the soil in lb/cu ft (kg/cu m); B_c is the outside width of the conduit in ft (m); and C_c is the load coefficient. Values of C_c may be obtained from Figure 49. In this diagram, H is the height of fill above the top of the conduit in ft (m); B_c is the outside width of conduit in ft (m); p is the projection ratio; and r_{sd} is the settlement ratio (the latter two terms are defined immediately below).

2. Influence of Environmental Factors.—The shear component of the total load on a sewer under an embankment depends on two factors associated with the conditions under which the conduit is installed. These are the projection ratio, p, and the settlement ratio, r_{sd}.

The projection ratio, p, is defined as the ratio of the distance that the

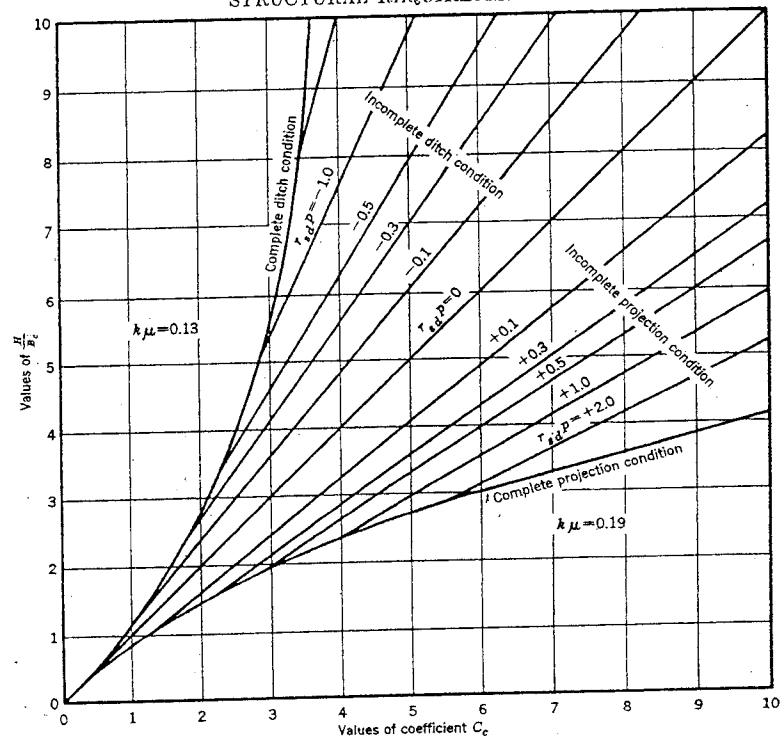


FIGURE 49.—Diagram for coefficient C_c for positive projecting conduits.
[From Figure 24-8 "Soils Engineering," by M. G. Spengler (see General References)]

top of the conduit projects above the adjacent natural ground surface, or the top of thoroughly compacted fill, or the bottom of a wide trench, to the vertical outside height of the conduit. It is a physical factor that can be determined in advanced stages of planning when the size of the conduit and its elevation have been established.

The settlement ratio, r_{sd}, indicates the direction and magnitude of the relative settlements of the prism of soil directly above the conduit and of the prisms of soil adjacent thereto. These relative settlements generate the shearing forces which combine algebraically with the weight of the central prism of soil to produce the resultant load on the conduit. The settlement ratio is the quotient obtained by taking the difference between the settlement of the horizontal plane in the adjacent soil which was originally level with the top of the conduit (the critical plane) and the settlement of the top of the conduit and dividing the difference by the compression of the columns of soil between the natural ground surface and the level of the top of the conduit. The formula for the settlement ratio is

$$r_{sd}=\frac{(s_m+s_g)-(s_f+d_c)}{s_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots 7$$

in which r_{sd} is the settlement ratio (positive projecting conduit); s_g is the settlement of the natural ground adjacent to the conduit; s_m is the compression of the columns of soil of height pB_c; (s_m+s_g) is the settlement of the critical plane; d_c is the deflection of the conduit, that is, the shortening of its vertical dimension; s_f is the settlement of the bottom of the conduit; and (s_f+d_c) is the settlement of the top of the conduit.

The elements of the settlement ratio are shown in Figure 48. When the settlement ratio is positive, the shearing forces induced along the sides of the central prism of soil are directed downward and the load on the conduit is greater than the weight of the central prism. When the settlement ratio is negative, the shearing forces act upward and the load is less than the weight of the central prism.

The numerical magnitude of the product of the projection ratio and the settlement ratio, $r_{sd}p$, is an indicator of the relative height of the plane of equal settlement and, therefore, of the magnitude of the shear component of the load. The plane of equal settlement is at the top of the conduit when this product is equal to zero. There are no induced shearing forces in this case and the load is equal to the weight of the central prism.

It is not practicable to predetermine a value of the settlement ratio by estimating the magnitude of its various elements except in very general terms. Rather, it should be treated as an empirical factor. Recommended design values of r_{sd}, based on measured settlements of a number of actual conduits, are:

Type of Conduit	Soil Conditions	Settlement Ratio, r_{sd}
Rigid	Rock or unyielding foundation	+1.0
Rigid	Ordinary foundation	+0.5 to +0.8
Rigid	Yielding foundation	0 to +0.5
Rigid	Negative projecting installations	−0.3 to −0.5
Flexible	Poorly-compacted side fills	−0.4 to 0
Flexible	Well-compacted side fills	0

3. *Embankment Soil Characteristics.*—The load on a projecting conduit is influenced directly by the unit weight of the embankment soil. If the soil is to be compacted to a specified dry density, the corresponding wet density under normal moisture conditions should be used in calculating the load. A design value of not less than 120 or 125 lb/cu ft (1,900 or 2,000 kg/cu m) is recommended if specific information relative to unit weight of soil is not available.

The load also is influenced by the coefficient of internal friction of the embankment soil. Recommended values of the product $k\mu$ are as follows (also see Figure 49):

For a positive settlement ratio, $k\mu=0.19$,

For a negative settlement ratio, $k\mu=0.13$.

Example 5. Determine the load on a 48-in. diam reinforced concrete pipe installed as a positive projecting conduit under a fill 32 ft high

above the top of the pipe. The wall thickness of the pipe is 5 in. and the fill weighs 125 lb/cu ft.

Assume the projection ratio is +0.5 and the settlement ratio is +0.6 and the projection ratio is +0.50. Then $H=32$ ft; $B_c=4.83$ ft; $H/B_c=6.63$; $r_{sd}p=0.6\times0.5=0.3$; C_c (from Figure 49) $=9.2$; and $W_c=9.2\times125\times4.83^2=26{,}800$ lb/ft (40,000 kg/m).

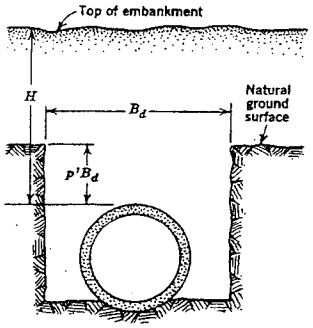
(c) Negative Projecting Conduits and Imperfect Trench Conduits. A negative projecting conduit (Figure 50) is one installed in a relatively shallow trench with its top at some elevation below the natural ground surface. The trench above the conduit is refilled with loose, compressible material, and the embankment is constructed to finished grade by ordinary methods.

Sometimes straw, hay, cornstalks, sawdust, or similar materials may be added to the trench backfill to augment the settlement of the interior prism. The greater the value of the negative projection ratio, p', and the more compressible the trench backfill over the conduit, the greater will be the settlement of the interior prism of soil in relation to the adjacent fill material. In using this technique, the plane of equal settlement must fall below the top of the finished embankment. This action generates upward shearing forces which relieve the load on the conduit.

An imperfect trench conduit (Figure 51) first is installed as a positive projecting conduit. The embankment then is built up to some height above the top and thoroughly compacted as it is placed. A trench of the same width as the structure next is excavated directly over the conduit down to or near its top. This trench is refilled with loose, compressible material, and the balance of the embankment is completed in a normal manner.

The formula for loads on negative projecting conduits is

$$W_c=C_nwB^2_d \qquad (8)$$



Negative projecting

FIGURE 50.—Negative projecting conduit.

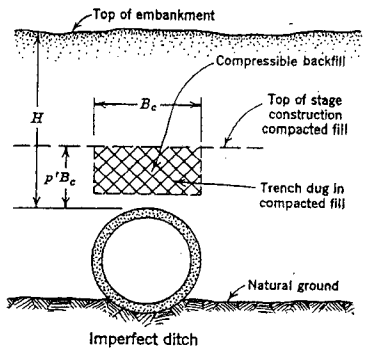


FIGURE 51.—Imperfect trench conduit.

in which W_c is the load on the conduit in lb/ft (kg/m); w is the unit weight of soil in lb/cu ft (kg/cu m); B_d is the width of the trench in ft (m); C_n is the load coefficient (Figure 52), a function of H/B_d or H/B_c, p', and r_{sd}; p' is the projection ratio; and r_{sd} is the settlement ratio as defined below.

In the case of imperfect trench conduits, B_c is substituted for B_d in Equation 8 in which B_c is the width of the pipe in ft (m), assuming the trench in fill is no wider than the pipe.

The projection ratio, p', is equal to the vertical distance from the firm ground surface down to the top of the conduit divided by the width of the trench, B_d, in the case of negative projecting conduits, or by the width of the conduit, B_c, in the case of imperfect trench conduits.

The settlement ratio, r_{sd}, for these cases is the quotient obtained by taking the difference between the settlement of the firm ground surface and the settlement of the plane in the trench backfill which was originally level with the ground surface (the critical plane) and dividing the difference by the compression of the column of soil in the trench. The formula for the settlement ratio is

$$r_{sd}=\frac{s_g-(s_d+s_f+d_c)}{s_d} \qquad (9)$$

in which r_{sd} is the settlement ratio for negative projecting or imperfect trench conduits; s_g is the settlement of the firm ground surface; s_d is the compression of trench backfill within the height $p'B_d$ or $p'B_c$; s_f is the settlement of the bottom of the conduit; d_c is the deflection of the conduit, that is, the shortening of its vertical dimension; and $(s_d+s_f+d_c)$ is the settlement of the critical plane. The elements of the settlement ratio are shown in Figure 53.

Present knowledge of the value of the settlement ratio which may develop in these special cases is very meager. A design value of -0.3 is recommended temporarily.

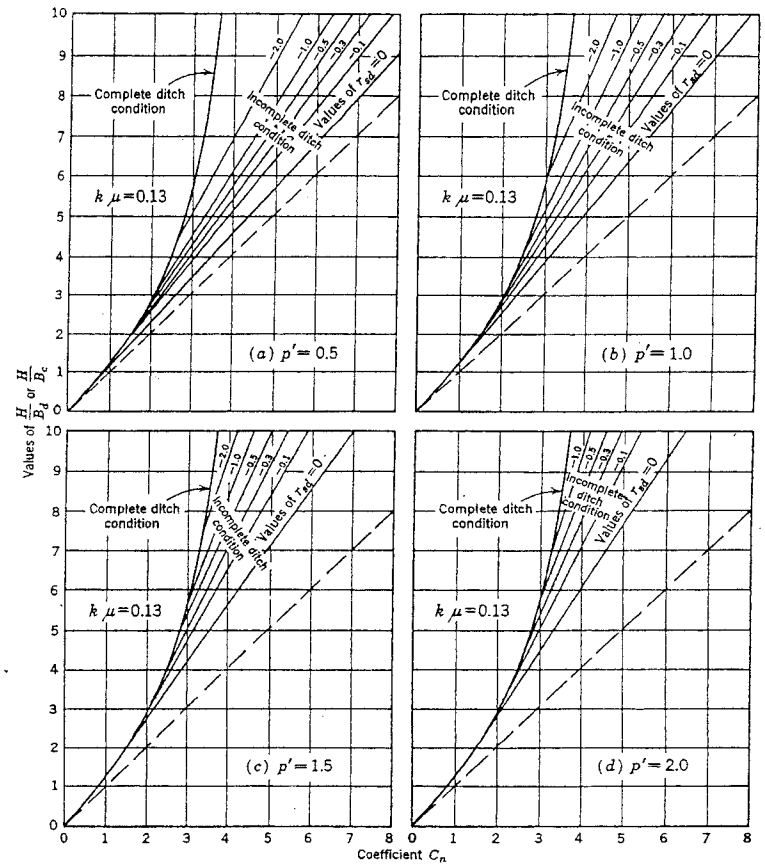


FIGURE 52.—Diagrams for coefficient C_n for negative projecting conduits and imperfect ditch conduits.

Example 6. Determine the load on the pipe of *Example 5* when installed as a negative projecting conduit in a trench whose depth is such that the top of the pipe is 7 ft below the surface of the natural ground in which the trench is dug. The width of the trench is 2 ft greater than the outside diameter of the pipe.

Assume the settlement ratio $=-0.3$. Then $H=32$ ft; $B_d=4.83+2=6.83$ ft; $H/B_d=4.69$; $p'\cong 1.0$; C_n (from Figure 52) $=3.0$; and $W_c=3.0\times 125\times 6.83^2=17{,}500$ lb/ft (26,100 kg/m).

Example 7. Determine the load on the pipe of *Example 5* when installed as an imperfect trench conduit with its top 2.5 ft below the elevation to

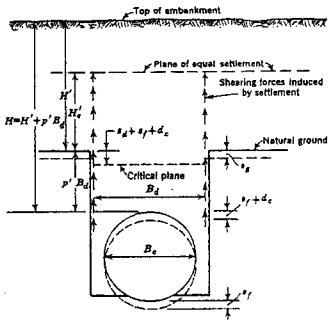


FIGURE 53.—Settlements that influence loads on negative projecting conduits.

which the soil is compacted thoroughly for a distance of 12 ft on each side of the pipe.

Assume the settlement ratio $=-0.3$. Then $H=32$ ft; $B_c=4.83$; $H/B_c=6.63$; $p'=$approximately 0.5; C_n (from Figure 52) $=4.8$; and $W_c=4.8\times125\times4.83^2=14{,}000$ lb/ft (20,900 kg/m).

5. Loads for Jacked Conduits and Certain Tunnel Conditions

(*a*) **General.**—When the sewer is more than 30 or 40 ft (9 or 12 m) deep, or when surface obstructions are such that it is difficult to construct the sewer by cut and cover, it may be more economical to place the sewer by means of jacking or tunneling. The theories set forth herein usually will be appropriate for materials where jacking of the pipe is possible and for tunnels in homogeneous soils of low plasticity. Where a tunnel is to be constructed through materials subject to unusual internal pressures and stresses, such as some types of clays or shales which tend to squeeze or swell, or through blocky and seamy rock, the loads on the conduit cannot be determined from the factors discussed here. Reference should be made to Section B6.

The methods of constructing sewers by tunneling and jacking are described in Chapter 11. Tunnel supports carry the earth load until the conduit is constructed and the voids between the conduit and tunnel supports are filled. Jacked pipe (4) (5) is assumed to carry the earth load as it is pushed into place.

(*b*) **Load-Producing Forces.**—In the materials considered herein, the vertical load acting on the jacked pipe or tunnel supports, and eventually the pipe in the tunnel, is the resultant of two major forces. First is the weight of the overhead prism of soil within the width of the jacked pipe or tunnel excavation. Second is the shearing forces generated between the interior prisms and the adjacent material due to friction and cohesion of the soil.

During excavation of a tunnel, and varying somewhat with construction methods, the soil directly above the face of the tunnel tends to settle slightly in relation to the soil adjacent to the tunnel because of the lack of support during the period immediately after excavation and prior to the placing of the tunnel support. Also, the tunnel supports and the sewer pipe must deflect and settle slightly when the vertical load comes on them. This downward movement or tendency for movement induces upward shearing forces which support a part of the weight of the prism of earth above the tunnel. In addition, the cohesion of the material provides further support for the weight of the prism of earth above the tunnel. The resultant load on the horizontal plane on the top of the tunnel and within the width of the tunnel excavation is equal to the weight of the prism of earth above the tunnel minus the upward friction forces and minus the cohesion of the soil along the limits of the prism of soil over the tunnel.

Hence, the forces involved with gravity earth loads on jacked pipe or tunnels in such soils are similar to those discussed for loads on pipe in trenches except for the cohesion of the material. Cohesion also exists in the case of the loads in trenches and embankments but is neglected because the cohesion of the disturbed soil is of minor consequence and may be absent altogether if the soil is saturated. However, in the case of jacked pipe or in tunnels, where the soil is undisturbed, cohesion can be an appreciable factor in the loads and may be considered safely if reasonable coefficients are assumed. Jacking stresses must be investigated in pipe which is to be jacked into place.

(*c*) **Marston's Formula.**—When modified to include cohesion, Marston's formula may be used to determine the gravity earth loads on jacked pipe or pipe in tunnels through undisturbed soil (Figure 54). It takes the form:

$$W_t=C_tB_t(wB_t-2c) \qquad (10)$$

in which W_t is the load on the pipe or tunnel support in lb/ft (kg/m); w is the unit weight of the soil above the tunnel in lb/cu ft (kg/cu m); B_t is the maximum width of the tunnel excavation in ft (m) (B_c in the case of jacked pipe); c is the cohesion coefficient in psf (kg/sq m); and C_t is a load coefficient which is a function of the ratio of the distance from the ground surface to the top of the tunnel to the width of the tunnel excava-

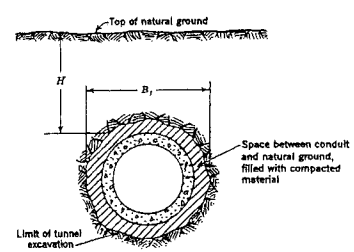


FIGURE 54.—Conduit in tunnel.

tion and of the coefficient of internal friction of the material of the tunnel.

The formula for C_t is identical to that for C_d (Equation 3), except that H is the distance from the ground surface to the top of the tunnel and B_t is substituted for B_d. The values of the coefficient for C_t for various ratios of H/B_t and various types of materials may be obtained from Figure 55.

An analysis of the formula for computing C_t indicates that for very

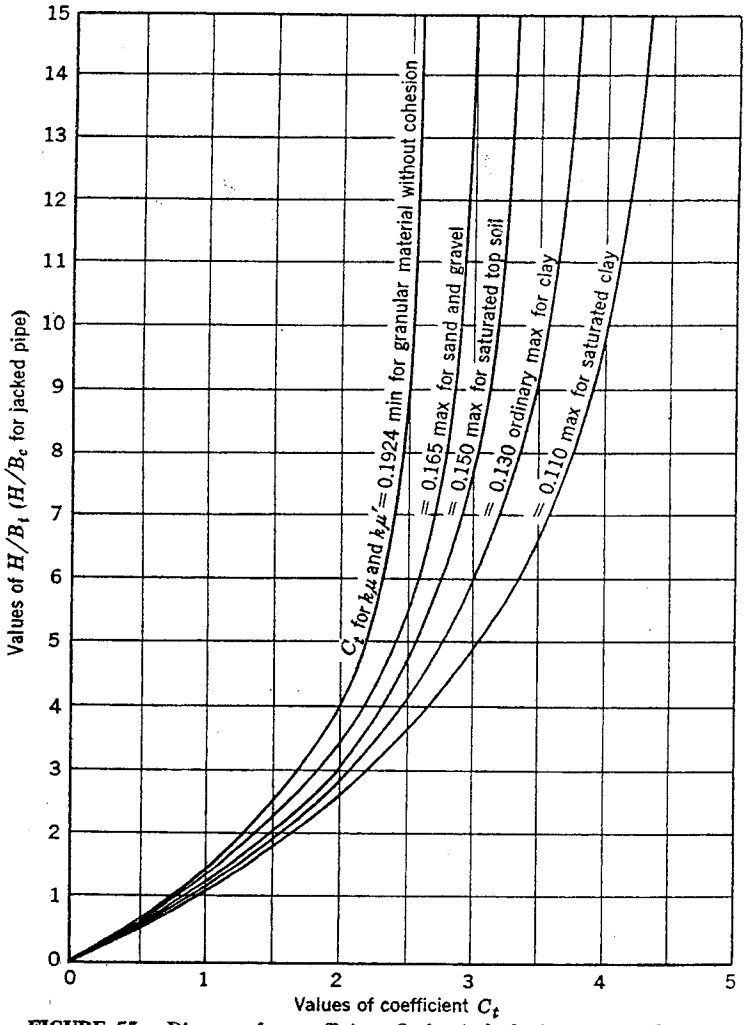


FIGURE 55.—Diagram for coefficient C_t for jacked pipe or tunnels in undisturbed soil.

high values of H/B_t, the coefficient C_t approaches the limiting value of $1/(2k\mu')$. Hence, where the tunnel is very deep, the load on the tunnel can be calculated readily by using the limiting value of C_t.

(*d*) ***Tunnel Soil Characteristics.***—The discussion regarding unit weight and coefficient of friction for sewers in trenches applies equally to the determination of earth loads on jacked pipe or pipe in tunnels through undisturbed soil.

The one additional factor that enters into the determination of loads on tunnels is c, the coefficient of cohesion. An examination of Equation 10 shows that the proper selection of the coefficient c is very important; unfortunately it can vary widely even for similar types of soils.

It may be possible in some instances to obtain undisturbed samples of the material and to determine the value of c by appropriate laboratory tests; and this should be done whenever possible. It is suggested that conservative values of c be employed in order to allow for a saturated condition of the soil or for other unknown factors. Design values should probably be about 33 percent of the laboratory test value to allow for uncertainties.

Recommended safe values of cohesion for various soils (if it is not practicable to determine c from laboratory tests) are:

Material	Values of c (psf)
Clay, very soft	40
Clay, medium	250
Clay, hard	1,000
Sand, loose dry	0
Sand, silty	100
Sand, dense	300
Top soil, saturated	100

Values of $k\mu$ and $k\mu'$ are the same as those noted in Figure 45.

(*e*) **Effect of Excessive Excavation.**—Where the tunnel is constructed by a method that results in excessive excavation and where these voids above the pipe or tunnel lining are not backfilled carefully or packed with grout or other suitable backfill materials, saturation of the soil or vibration may eventually destroy the cohesion of the undisturbed material above the conduit and result in loads in excess of those calculated using Equation 10. If this situation is anticipated, it is suggested that Equation 10 be modified by eliminating the cohesion term. The calculated loads then will be the same as those obtained from Equation 2.

Example 8. Determine the loads on a 60-in. diam pipe in a tunnel 40 ft deep.

Assume the width of excavation, $B_t = 78$ in. $= 6.5$ ft; type of soil is silty sand ($k\mu' = 0.150$, $c = 100$ psf, and $w = 110$ lb/cu ft); and the depth of tunnel, $H = 40$ ft. Then $H/B_t = 40/6.5 = 6.15$; C_t (from Figure 55) $= 2.83$.

Employing Equation 10, $W_t = 2.83 \times 6.5(110 \times 6.5 - 2 \times 100)$; or $W_t = 9{,}500$ lb (14,200 kg).

If the tunnel were very deep, $C_t = 1/(2k\mu') = 3.33$; and $W_t = 11{,}200$ lb (16,700 kg).

6. Loads for Tunnels

(*a*) **General.**—When the sewer is to be constructed in a tunnel through homogeneous soils of low plasticity, design should be based on theories set forth in Section B5 above. The design of tunnels through other types of materials is discussed in this section. The usual procedure in tunnel construction is to complete the excavation first and then place either a monolithic concrete liner or install pipe, grouting, or concreting it in place. The strength of such a section often is obtained by means of pressure grouting to strengthen the surrounding material instead of relying on the conduit itself. Tunnel loads therefore usually are determined for purposes of selecting supports to be used during excavation, and the pipe or cast-in-place liner designed primarily to withstand loads from pressure grouting.

A complete discussion of tunnels is not within the scope of this manual and the designer's attention is called to references listed at the end of this chapter (6) (7) (8) (9).

(*b*) **Load-Producing Forces.**—When the tunnel is to be constructed through soils which tend to squeeze or swell such as some types of clay or shale, or through blocky or seamy rock, the vertical load cannot be determined from a consideration of the factors discussed previously and Equation 10 is not applicable.

The determination of the rock pressures exerted against the tunnel lining is largely an estimate based on previous experience of the performance of linings in similar rock formations, although attempts at numerical analysis of stress conditions around a tunnel shaft have been made.

In case of plastic clay, the full weight of the overburden is likely to come to rest on the tunnel lining some time after construction. The extent of lateral pressures to be expected has not as yet been determined fully, especially the passive resistance which will be maintained permanently by a plastic clay in the case of flexible ring-shaped tunnel lining.

On the other hand when tunneling through sand, only part of the weight of the overburden will come to rest on the tunnel lining at any time if adequate precautions are taken. The relief will be due to the transfer of the soil weight immediately above the tunnel to the adjoining soil mass by shearing stresses along the vertical planes. In this case Marston's formula may be used for estimating the total load which the tunnel lining may have to carry.

Great care must be taken to prevent any escape of sand into the tunnel during its construction. Moist sand will usually arch over small openings and not cause trouble in this respect, but entirely dry sand, which is sometimes encountered, is liable to trickle into the tunnel through the smallest gap in the temporary lining. Sand movements of this kind destroy most

if not all of the arching around the tunnel, with a resulting strong increase of both vertical and horizontal pressures on the supports of the lining. Such cases have been recorded and have caused considerable difficulty.

All soil parameters required for design should be obtained from laboratory testing.

C. LOADS ON SEWERS DUE TO SUPERIMPOSED LOADS

1. General Method

Two types of superimposed loads are encountered commonly in the structural design of sewers and culverts. These two types are (*a*) concentrated load, and (*b*) distributed load. Loads on conduits due to both types of loading can be determined by application of Boussinesq's solution for stresses in a semi-infinite elastic medium through the convenience of an integration developed by Holl for concentrated loads and tables of influence coefficients developed by Newmark for distributed loads.

2. Concentrated Loads

The formula for load due to superimposed concentrated load, such as a truck wheel (Figure 56), is given the following form by Holl's integration of Boussinesq's formula

$$W_{sc} = C_s \frac{PF}{L} \qquad (11)$$

in which W_{sc} is the load on the conduit in lb/unit length (kg/unit length); P is the concentrated load in lb (kg); F is the impact factor; C_s is the load coefficient (Table XXVI), a function of $B_c/(2H)$ and $L/(2H)$; H is the height of fill from the top of conduit to ground surface in ft (m); B_c is the width of conduit in ft (m); and L is the effective length of conduit in ft

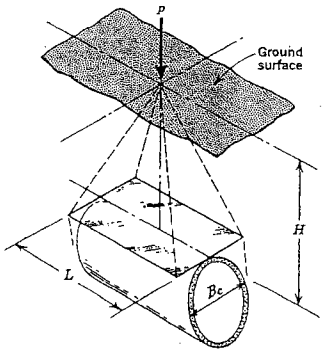


FIGURE 56.—Concentrated superimposed load vertically centered over conduit.

TABLE XXVI.—Values of Load Coefficients, C_s, for Concentrated and Distributed Superimposed Loads Vertically Centered Over Conduit *

$\frac{D}{2H}$ or $\frac{B_c}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$														
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.124	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.244	0.248
0.3	0.053	0.103	0.149	0.190	0.224	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.284	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.481	0.505	0.525	0.540	0.548
0.6	0.089	0.174	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.546	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.584	0.615	0.629	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.894	0.916
2.0	0.124	0.244	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.849	0.894	0.930	0.956

* Influence coefficients for solution of Holl's and Newmark's integration of the Boussinesq equation for vertical stress.

(m). The effective length of a conduit is defined as the length over which the average load due to surface traffic units produces the same stress in the conduit wall as does the actual load which varies in intensity from point to point. Little research information is available on this subject. Tentative recommendations are to use an effective length equal to 3 ft (0.9 m) for conduits greater than 3 ft long. Use the actual length for conduits shorter than 3 ft.

If the concentrated load is displaced laterally and longitudinally from a vertically centered location over the section of pipe under construction, the load on the conduit can be computed by adding algebraically the effect of the concentrated load on various rectangles each with a corner centered under the concentrated load. Values of C_s in Table XXVI divided by 4 equal the load coefficient for a rectangle whose corner is vertically centered under the concentrated load.

(a) Impact Factor

The impact factor, F, reflects the influence of dynamic loads caused by traffic. Suggested values for various kinds of traffic are:

Traffic Type	F
Highway	1.50
Railway	1.75
Airfield:	
Runways	1.00
Taxiways, aprons, hard stands	1.50

Example 9. Determine the load on a 24-in. diam pipe under 3 ft of cover caused by a 10,000-lb truck wheel applied directly above the center of the pipe.

Assume the pipe section is 2.5 ft long; the wall thickness is 2 in.; and the impact factor is 1.5. Then $B_c=24+4=28$ in.$=2.33$ ft; $L=2.5$ ft; and $H=3.0$ ft. Finally $B_c/2H=2.33/6=0.39$; and $L/2H=2.5/6=0.41$; the load coefficient is 0.240 from Table XXVI. Substituting in Equation 11,

$$W_{sc}=0.240\times\frac{10{,}000\times1.5}{2.5}=1{,}440 \text{ lb/ft } (2{,}140 \text{ kg/m}).$$

3. Distributed Loads

For the case of a superimposed load distributed over an area of considerable extent, as shown in Figure 57, the formula for load on the conduit is

$$W_{sd}=C_s pFB_c \quad \ldots\ldots\ldots\ldots 12$$

in which W_{sd} is the load on the conduit in lb/unit length (kg/unit length); p is the intensity of distributed load in psf (kg/sq m); F is the impact factor; B_c is the width of the conduit in ft (m); C_s is the load coefficient, a function of $D/(2H)$ and $M/(2H)$ from Table XXVI; H is the height from the top of the conduit to the ground surface in ft (m); and D and M

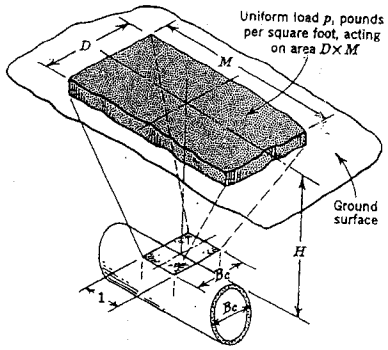


FIGURE 57.—Distributed superimposed load vertically centered over conduit.

are the width and length, respectively, of the area over which the distributed load acts, in ft (m).

Values of C_s can be read directly from Table XXVI if the area of the distributed superimposed load is centered vertically over the center of the conduit under consideration.

The load on the conduit can be computed by adding algebraically the effect of various rectangles of loaded area if the area of the distributed superimposed load is not centered over the conduit but is displaced laterally and longitudinally. It is more convenient to work in terms of load under one corner of a rectangular loaded area rather than at the center. Dividing the tabular values of C_s by 4 will give the effect for this condition.

4. Conduits Under Railway Tracks

The live load may be considered as a uniformly distributed load equal to the weight of locomotive driver axles divided by an area equal to the length occupied by the drivers multiplied by the length of ties when sewers are constructed under railroad tracks. In addition, 200 lb/ft (300 kg/m) should be allowed for weight of the track structure.

Example 10. Determine the load on a 48-in. diam concrete pipe under 6 ft of cover (bottom of ties to top of pipe) resulting from the Cooper E-70 railroad loading.

Assume the pipe wall thickness is 4 in., the locomotive load consists of four 70,000-lb axles spaced 5 ft center-to-center, the impact factor is 1.75, and the weight of track structure is 200 lb/ft. Then $B_c=48+8=56$ in. or 4.67 ft; $H=6$ ft; $D=8$ ft; and $M=20$ ft.

The unit load plus impact at the base of the ties is $\frac{4\times 70{,}000\times 1.75}{8\times 20}+\frac{200}{8}=3{,}085$ psf; $\frac{D}{2H}=\frac{8}{12}=0.67$; and $\frac{M}{2H}=\frac{20}{12}=1.67$; the influence coefficient is 0.641 (from Table XXV).

By Equation 12, $W_{sd}=0.641\times 3.085\times 4.67=9{,}240$ lb/ft (13,800 kg/m).

5. Conduits Under Rigid Pavement

A method of computing the load transmitted to conduits under rigid pavement is given elsewhere (10).

D. SUPPORTING STRENGTH OF RIGID CONDUITS

The ability of a conduit to resist safely the calculated earth load depends not only on its inherent strength but also on the distribution of the vertical load and bedding reaction and on the lateral pressure acting against the sides of the conduit.

The inherent strength of a rigid conduit usually is specified by its resistance in the three-edge bearing test. This test is both convenient and severe but it does not reproduce the actual field load conditions. Thus, to select the most economical combination of bedding and pipe strength, a relationship must be established between calculated load, laboratory strength, and field strength for various installation conditions.

Field or supporting strength, moreover, depends on the distribution of the vertical load and the reaction against the bottom of the pipe. It also depends on the magnitude and distribution of the lateral pressure acting on the sides of the pipe. These factors, therefore, make it necessary to qualify the term "supporting strength" with a description of conditions of installation in a particular case as they affect the distribution of the load, the reaction, and the magnitude and distribution of lateral pressure.

As in the case of computing loads on the conduit, it is convenient when determining supporting strength to classify installation conditions as either "trench" or "embankment."

1. Laboratory Test Strength

Rigid pipe may be tested for strength in the laboratory by the three-edge bearing test. Methods of testing are described in detail in ASTM Specifications C301, C497, and C500 and USASI * Specification A60.2. ASTM and USASI specifications for pipe contain the minimum required strengths for three-edge bearing tests.

Laboratory strength, in the case of reinforced concrete pipe, may be expressed as the load per foot of length which causes the pipe to develop a 0.01-in. (0.025-cm) crack, or as the 0.01-in. crack load and the ultimate load which the pipe will withstand. The cracking load and the ultimate load, in the case of non-reinforced pipe, are essentially the same, and the cracking load is considered to be the ultimate strength of the pipe.

The strength of the pipe, in lb/ft (kg/m) at either 0.01-in. crack or ultimate, divided by the nominal internal diameter of the pipe, in ft (m), is called the D-load strength. Thus, if a 48-in. (1.22-m) diam reinforced concrete pipe has a three-edge bearing test strength at a 0.01-in. crack of

* American Society for Testing and Materials, Philadelphia, Pa.; United States of America Standards Institute, New York, N.Y.

8,000 lb/ft (11,900 kg/m) and an ultimate strength of 12,000 lb/ft (17,900 kg/m), the 0.01-in. crack strength is 2,000 D (9,800 D) and the ultimate strength is 3,000 D (14,700 D).

2. Pipe Bedding

The contact between a pipe and the foundation on which it rests is the pipe bedding. This has an important influence on the distribution of the reaction against the bottom of the pipe and therefore influences the supporting strength of the pipe as installed. In the case of bell and spigot pipe, a suitable excavation should be made to receive the pipe bells. It should be of sufficient width and depth to insure that the bottom reaction will act only on the pipe barrel and not on the bell.

Concrete cradle bedding for large diameter reinforced concrete pipe has been used advantageously in some areas. The concrete cradle provides positive uniform distribution of the reaction at the bottom of the pipe.

If the full benefit of the bedding method is to be achieved, the bottom of the trench or embankment must be stable. Ways of achieving this condition are discussed in Chapter 11.

3. Backfill

The soil at the sides of a pipe and above it is the backfill. It influences the supporting strength of the pipe by exerting lateral pressure against the sides.

4. Field Supporting Strength

The field supporting strength of a rigid pipe conduit is the maximum load in lb/ft (kg/m) which the pipe will support while retaining complete serviceability when installed under specified conditions of bedding and backfilling.

The "field supporting strength" should not be confused with the "safe supporting strength" or "working strength" which contains a factor of safety.

The field supporting strength, in addition to the inherent strength of pipe, is influenced by the distribution of the vertical load on the top, the distribution of the vertical reaction on the bottom, and the amount and distribution of effective lateral pressure against the sides of the pipe. It is greater than the three-edge bearing test strength because of the more favorable distribution of the load and reaction in a field installation and because of the complete absence of side pressure in the laboratory test.

5. Load Factor

The ratio of the strength of a pipe under any stated condition of loading and bedding to its strength measured by the three-edge bearing test is called the load factor. The relationship between field supporting strength, laboratory strength, and load factor is expressed as follows:

Field supporting strength = load factor × three-edge bearing strength

The load factor does not contain a factor of safety. Load factors have been determined experimentally and analytically for the commonly used construction conditions for both trench and embankment conduits.

6. Supporting Strength in Trench Conditions

(*a*) **Classes of Bedding.**—Four classes of beddings most often used for pipes in trenches are described as follows and illustrated in Figure 58.

1. Class A—Concrete Cradle or Concrete Arch Bedding.—This class of bedding may take either of two forms:

a. Concrete Cradle. The pipe shall be bedded in a monolithic cradle of plain or reinforced concrete having a minimum thickness of one-fourth the inside pipe diameter or a minimum of 4 in. (10 cm) under the barrel and extending up the sides for a height equal to one-fourth the outside diameter. The cradle shall have a width at least equal to the outside diameter of the pipe barrel plus 8 in. (20 cm).

Backfill above the cradle and extending to 12 in. (30 cm) above the crown of the pipe shall be compacted carefully.

b. Concrete Arch. The pipe shall be embedded in carefully compacted granular material having a minimum thickness of one-fourth the outside diameter between barrel and bottom of trench excavation and extending halfway up the sides of the pipe. The top half of the pipe shall be covered with a monolithic plain or reinforced concrete arch having a minimum thickness of one-fourth the inside diameter at the crown and having a minimum width equal to the outside pipe diameter plus 8 in. (20 cm).

The load factor for Class A concrete cradle bedding is 2.2 for plain concrete with lightly tamped backfill; 2.8 for plain concrete with carefully tamped backfill; and up to 3.4 for reinforced concrete with $p = 0.4$ percent, in which p is the ratio of the area of steel to the area of concrete at the invert.

The load factor for Class A concrete arch type bedding is 2.8 for plain concrete; up to 3.4 for reinforced concrete with $p = 0.4$ percent; and up to 4.8 for reinforced concrete with $p = 1.0$ percent, in which p is the ratio of the area of steel to the area of concrete at the crown.

2. Class B—First-Class Bedding.—Class B bedding may be achieved by either of two construction methods:

a. Shaped Bottom with Tamped Backfill. The bottom of the trench excavation shall be shaped to conform to a cylindrical surface with a radius at least 2 in. (5 cm) greater than the radius to the outside of the pipe and with a width sufficient to allow six-tenths of the width of the pipe barrel to be bedded in fine granular fill placed in the shaped excavation. Carefully compacted backfill shall be placed at the sides of the pipe to a thickness of at least 12 in. (30 cm) above the top of the pipe. Shaped

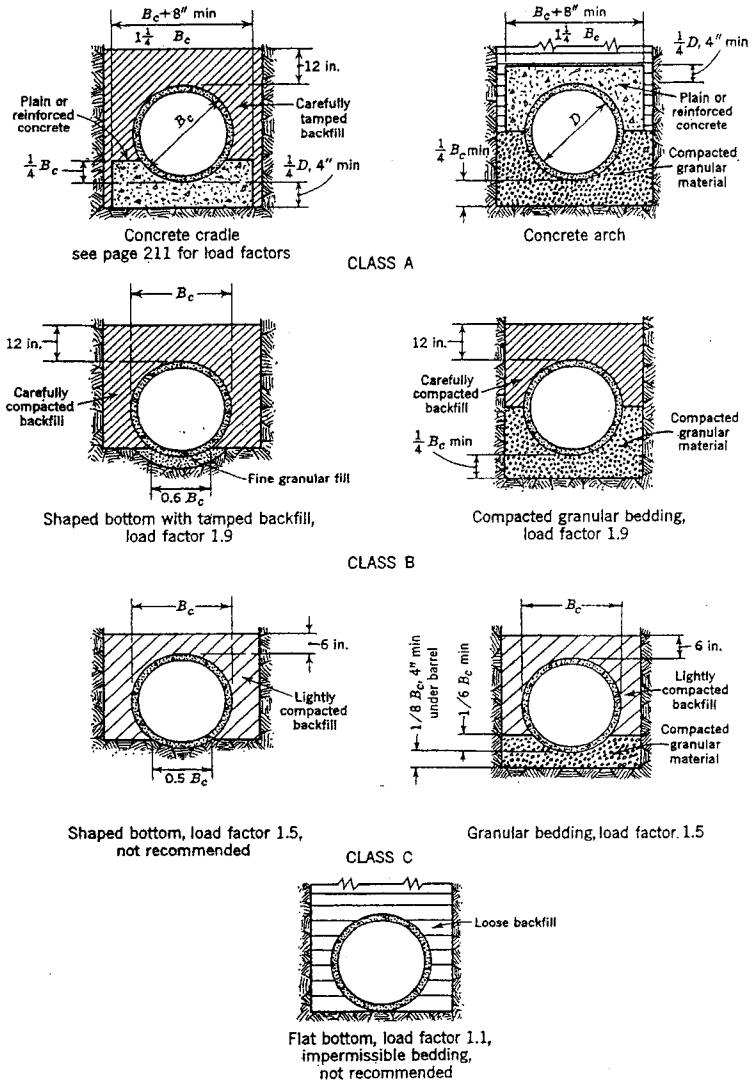


FIGURE 58.—Classes of bedding for conduits in trench. Note: In rock trench, excavate at least 6 in. (15 cm) below the bell of the pipe except where concrete cradle is used.

trench bottoms are difficult to achieve under current construction conditions.

b. Compacted Granular Bedding with Tamped Backfill. The pipe shall be bedded in compacted granular material placed on a flat trench bottom. The granular bedding shall have a minimum thickness of one-fourth the outside pipe diameter and shall extend halfway up the pipe barrel at the sides. The remainder of the side fills and a minimum depth of 12 in. (30 cm) over the top of the pipe shall be filled with carefully compacted material.

The load factor for either construction method is 1.9.

3. Class C—Ordinary Bedding.—Class C ordinary bedding may be achieved by either of two construction methods:

a. Shaped Bottom. The pipe shall be bedded with "ordinary" care in an earth foundation formed in the trench bottom by a shaped excavation which will fit the pipe barrel with reasonable closeness for a width of at least 50 percent of the outside pipe diameter. The side fills and area over the pipe to a minimum depth of 6 in. (15 cm) above the top of the pipe shall be filled with lightly compacted fill. The shaped bottom bedding is not recommended for pipeline construction because it is impractical and costly.

b. Compacted Granular Bedding with a Tamped Backfill. The pipe shall be bedded in compacted granular material placed on a flat trench bottom. The granular bedding shall have a minimum thickness of 4 in. (10 cm) under the barrel and shall extend one-tenth to one-sixth of the outside diameter up the pipe barrel at the sides. The remainder of the side fills and to a minimum depth of 6 in. (15 cm) over the top of the pipe shall be filled with lightly compacted backfill.

The load factor for Class C bedding is 1.5.

4. Class D—Flat Bottom Trench, Impermissible Bedding.—In this class of bedding the bottom of the trench is left flat, and no care is taken to secure compaction of backfill at the sides and immediately over the pipe. The load factor for Class D bedding is 1.1.

Class D bedding is not recommended for pipeline construction. Under present construction conditions, Class B or C bedding with a compacted granular bedding is generally a more practical and economical method of installation.

(*b*) Granular Material.—Granular material is used commonly to bed sewer pipes in lieu of shaping the trench bottom to fit the contour of the pipe. The granular bedding material, in addition to providing firm uniform support for the pipe, frequently also must stabilize the trench bottom. The pipe bedding material must remain firm and not permit displacement of the pipe either during pipe laying and backfilling or following completion of construction. Furthermore, the material should not have a tendency to flow when flooded nor when an excavation is opened through it.

The two general gradation classifications used for granular bedding materials are uniformly graded and well graded. Uniformly-graded materials, such as pea gravel, are one size materials with a low percentage of over- and under-size particles. Well-graded materials contain several sizes of particles, in stated proportions, ranging from a maximum to a minimum size.

Coarse sand, pea gravel, crushed gravel, gravel, crushed stone screenings, and crushed stone have been used for pipe bedding. Pea gravel and uniformly graded (one size) crushed stone of comparable size have been used commonly.

Fine materials, coarse sand, or screenings are not satisfactory for stabilizing trench bottoms and are difficult to compact in a uniform manner to provide proper pipe bedding.

Well-graded material is most effective for stabilizing trench bottoms and has less tendency to flow than a uniformly-graded material. However, uniformly-graded material is easier to place and compact around sewer pipes. Rounded granular material has a greater tendency to flow and allow pipe movement after laying than angular material.

Recent research (11) has shown that a well-graded crushed stone is most suitable for pipe bedding and that it is better suited for bedding than well-graded gravel. Both materials, however, are better suited for bedding than a uniformly-graded pea gravel.

It is considered that well-graded crushed stone or crushed gravel meeting the requirements of ASTM Designation C 33, Gradation 67 (¾-in. to No. 4) (1.9 to 0.48 cm) generally will provide the most satisfactory pipe bedding. A well-graded gravel meeting these same requirements also can be used.

Granular pipe bedding material when referred to in this manual generally means one of the foregoing well-graded materials. Any material used for pipe bedding should be compacted thoroughly as it is placed to provide uniform support for the pipe barrel and to fill completely all voids under and around the pipe.

(c) Rock or Other Incompressible Foundations.—Where ledge rock, compact rocky or gravelly soil, or other unyielding foundation material is encountered, the pipes should be bedded in accordance with the requirements of one of the foregoing classes of bedding, but with the following additions: The hard unyielding material should be excavated to the elevation of the bottom of the concrete cradle (Class A bedding) or below the bottom of the pipe and pipe bell (Class B, C, or D bedding) to a depth of at least 6 in. (15 cm). The width of the excavation should be at least five-fourths the outside diameter of the pipe and it should be refilled with granular material.

(d) Encased Pipe.—Total encasement of non-reinforced rigid pipe in concrete may be necessary where the required safe supporting strength cannot be obtained by other bedding methods.

A typical concrete encasement detail is shown in Figure 59 as used by the Department of Public Works, City of Los Angeles. The load factor

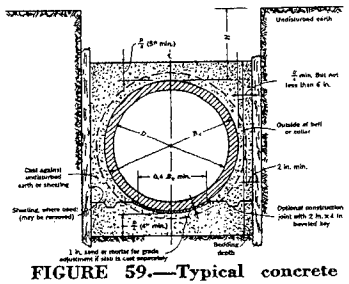


FIGURE 59.—Typical concrete encasement details.

for concrete encasement varies with the thickness of concrete and the use of reinforcing and may be greater than that used for concrete cradle or arch. The load factor for the encasement shown in Figure 59 is 4.5.

Concrete encasement also may be required for pipelines built in deep trenches in order to assure uniform support or for pipe lines built on comparatively steep grades where there is the possibility that earth beddings may be eroded by currents of water under and around the pipe.

7. Supporting Strength in Embankments

It is possible for the active soil pressure against the sides of a pipe placed in an embankment to be a significant factor in the resistance of the structure to vertical load. This factor is important enough to justify a separate examination of the supporting strength of embankment conduits.

(a) Positive Projecting Conduits.—The load factor for rigid pipes installed as projecting conduits under embankments or in wide trenches depends on the class of bedding in which the pipe is laid, the magnitude of the active lateral soil pressure against the sides of the pipe, and the area of the pipe over which the active lateral pressure is effective.

For projecting conduits of circular and elliptical cross section, the load factor is

$$L_f = \frac{A}{N - xq} \qquad (13)$$

in which L_f is the load factor; A is a pipe shape factor; N is a pipe bedding factor; x is a parameter dependent on the area over which lateral pressure effectively acts; and q is the ratio of total lateral pressure to total vertical load on the pipe.

Classes of bedding for projecting conduits are shown in Figure 60. The values of A for circular and elliptical pipe are:

Pipe Shape	A
Circular	1.431
Elliptical	
Horizontal elliptical	1.337
Vertical elliptical	1.021

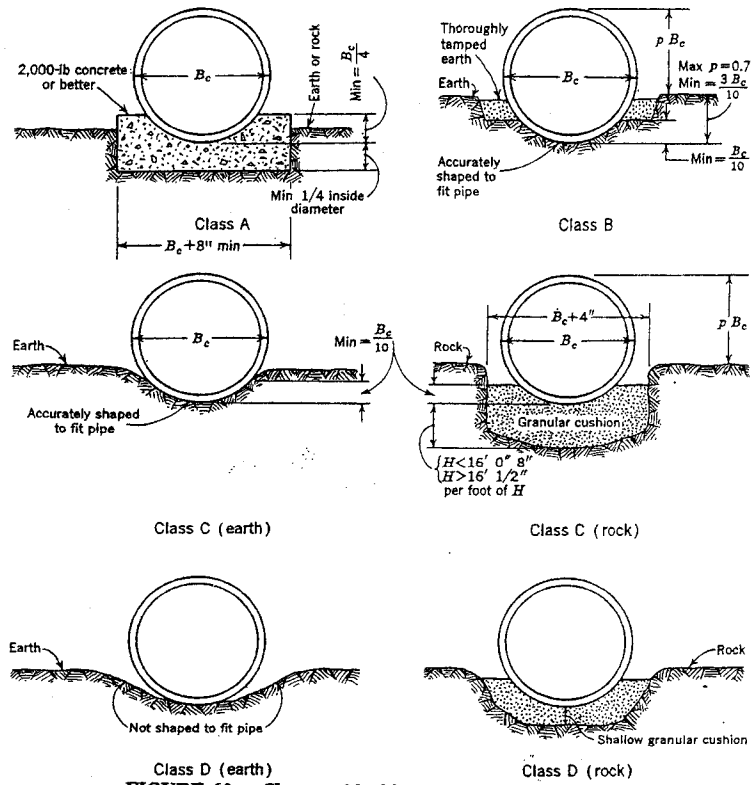


FIGURE 60.—Classes of bedding for projecting conduits.

Values of N for various classes of bedding are given in Table XXVII. Values of x for circular and elliptical pipe are listed in Table XXVIII.

The ratio, m, refers to the fraction of the pipe diameter over which lateral pressure is effective. For example, if lateral pressure acts on the top half of the pipe above the horizontal diameter, $m=0.5$.

The ratio of total lateral pressure to total vertical load, q, for positive projecting conduits may be estimated by the formula:

$$q=\frac{mk}{C_c}\left(\frac{H}{B_c}+\frac{m}{2}\right) \quad \text{.....14}$$

in which k is the ratio of unit lateral pressure to unit vertical pressure (Rankine's ratio). A value of $k=0.33$ usually will be sufficiently accurate for use in Equation 14.

(b) Negative Projecting Conduits.—The load factor for negative projecting conduits may be the same as for trench conduits for the various classes of bedding as given in Section 6. These load factors for Class

TABLE XXVII.—Values of N

Class of Bedding	Value of N — Pipe Shape: Circular	Horizontal Elliptical	Vertical Elliptical
A (reinforced cradle)	0.421 to 0.505	—	—
A (unreinforced cradle)	0.505 to 0.636	—	—
B	0.707	0.630	0.516
C	0.840	0.763	0.615
D	1.310	—	—

TABLE XXVIII.—Values of x

Fraction of Pipe Subjected to Lateral Pressure, m	x — Class A Bedding: Circular Pipe	Other than Class A Bedding: Circular Pipe	Horizontal Elliptical Pipe	Vertical Elliptical Pipe
0	0.150	0	0	0
0.3	0.743	0.217	0.146	0.238
0.5	0.856	0.423	0.268	0.457
0.7	0.811	0.594	0.369	0.639
0.9	0.678	0.655	0.421	0.718
1.0	0.638	0.638	—	—

B, C, and D bedding do not take into account lateral pressures against the sides of the pipe for the reason that unfavorable construction conditions often prevail at the bottom of a sewer trench. However, in the case of negative projecting conduits, conditions may be more favorable and it may be possible to compact the side-fill soils to the extent that some lateral pressure against the pipe can be relied on. If such favorable conditions are anticipated, it is suggested that the load factor be computed by means of Equations 13 and 14, using a value of $k=0.15$ for estimating the lateral pressure on the pipe.

(c) Imperfect Trench Conduits.—Imperfect trench conduits usually are installed as positive projecting conduits before the overlying soil is compacted and the imperfect trench is excavated. Therefore, lateral pressures are effective against the sides of the conduit, and the load factor should be calculated by Equations 13 and 14.

E. SUPPORTING STRENGTH OF FLEXIBLE PIPES

1. General Method

Flexible pipes under earth fills derive their ability to support load from their inherent strength plus the passive resistance pressure of the soil as they deflect and the sides of the pipe move outward against the soil side fills. This type of pipe fails by excessive deflection and collapse

or buckling rather than by rupture of the pipe walls as in the case of pipes made of brittle materials. Therefore, design of flexible pipes is directed toward determination of the deflection under load. A field supporting strength resulting in a deflection of five percent of the nominal diameter of the pipe is considered by many engineers to be a suitable criterion for design. Design criteria for buckling and longitudinal seam strength are suggested by Townsend (12).

A formula for calculating flexible pipe deflection under earth loading is

$$\triangle x = D_e \frac{KW_c r^3}{EI + 0.061\, E' r^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots 15$$

in which $\triangle x$ is the horizontal and vertical deflection of the pipe in in.; D_e is the deflection lag factor; K is a bedding constant dependent on the angle subtended by the pipe bedding; W_c is the vertical load on the pipe in lb/in.; r is the mean radius of the pipe in in.; E is the modulus of elasticity of the pipe material in psi; I is the moment of inertia per unit length of cross section of the pipe wall in in.^{4}/in.; $E' = er$ is the modulus of soil reaction in psi and e is the modulus of passive resistance of the enveloping soil in psi/in.

The deflection lag factor, empirically determined, compensates for the tendency of flexible pipes to continue to deform for some period of time after the full magnitude of load has developed on the pipe. Recommended values of this factor range from 1.25 to 1.50.

Values of the bedding constant, K, depending on the width of the pipe bedding are:

Bedding Angle (deg)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

There is much yet to be learned about the modulus of passive resistance of soil, e, and its influence on flexible pipe deflection. Some recent research has indicated that this modulus is influenced strongly by the size of the pipe and that, for a given type of soil in a given state of compaction, the product of the modulus times the radius of pipe, E', is constant, that is, for the same soil, the modulus varies inversely with the pipe radius. Also, observations on a limited number of pipes in service, where sufficient information is available to make an estimate, indicate that the value of E' varies widely—from a minimum of 234 psi (17 kg/sq cm) in the case of an uncompacted sandy clay loam to a maximum of 7,980 psi (560 kg/sq cm) for a crushed sandstone soil which was compacted to maximum density. On five culvert installations where the soil was compacted (although not necessarily to maximum density), the values

of E' ranged from 502 to 1,320 psi (35 to 93 kg/sq cm), the average being 765 psi (54 kg/sq cm). On the basis of these observations, a value of E' of 700 psi (49 kg/sq cm) is recommended in design if the side-fill soil is compacted to 90 percent or more of maximum density, AASHO T99, for a distance of two pipe diameters on each side of the pipe.

The first term in the denominator, EI, in Equation 15, reflects the influence of the inherent strength of the pipe on deflection; whereas the second term, $0.061\, E'r^3$, reflects the influence of the passive pressure on the sides of the pipe. The second term may be excessively predominant in the case of large-diameter pipes, with the result that a very light-weight pipe may appear to be satisfactory. Since the pipe wall must have sufficient local strength in bending and thrust to develop and utilize the passive resistance pressure on the sides of the pipe, it is recommended as a practical measure that the value of EI should never be less than about 10 to 15 percent of the term $0.061\, E'r^3$. Also, the gage of the metal must be sufficient to develop adequate strength of the bolted or riveted longitudinal joints of the pipe.

Almost the entire performance of a flexible conduit in retaining its shape and integrity is dependent on the selection, placement, and compaction of the envelope of earth surrounding the structure. For this reason, as much care should be taken in the design of the backfill as is used in the design of the conduit. The backfill material selected preferably should be of a granular nature to provide good shear characteristics. Cohesive soils can be used if careful attention is given to the moisture content.

If the material placed around the conduit is different from that used in the embankment or if for construction reasons fill is placed around the conduit before the embankment is built, the compacted backfill should cover the structure by at least 1 ft (0.3 m) and extend one diameter to either side of it.

2. Corrugated Metal Pipes

The most frequently used kind of flexible sewer pipe is constructed of corrugated metal. The sheets of pipes which are fabricated are of two general types, standard and structural plate. Standard corrugations are ½ in. (1.3 cm) deep and spaced 2⅔ in. (6.8 cm) center-to-center. Structural plate corrugations are 2 in. (5.1 cm) deep and spaced 6 in. (15.2 cm) center-to-center. The moment of inertia of the pipe wall for these two types of corrugations and various gage thicknesses of metal are shown in Table XXIX.

Equation 15 has been developed primarily for flexible conduits under embankments. Corrugated metal sewers that are to support a fill should not be placed directly on a cradle or pile bents. If such supports are necessary, they should have a flat top and be covered with a compressible earth cushion. Corrugated metal should not be encased in concrete.

TABLE XXIX.—Moments of Inertia of Corrugated Sheets

Gage	Thickness (in.)	Moment of Inertia (in.⁴/in.)	
		Standard Corrugations, ½ in. × 2⅔ in.	Structural plate Corrugations, 2 in. × 6 in.
1	0.2690	...	0.16541
3	0.2391	...	0.14588
5	0.2092	...	0.12670
7	0.1793	...	0.10777
8	0.1644	0.00550	0.09610
10	0.1345	0.00450	0.07812
12	0.1046	0.00350	0.05455
14	0.0747	0.00250	...
16	0.0598	0.00200	...

Note: In. × 2.54 = cm.

For corrugated metal pipes installed in trenches, reference is made to manufacturers' handbooks for recommended gages and corrugations.

Example 11. A 60-in. diam, 12-gage, structural plate corrugated metal pipe is to be constructed under a 25-ft embankment. What is the long-term deflection which may be expected to develop in the pipe?

Assume $B_c=5.33$ ft and $r_{sd}p=0$. Then the load on the pipe is $5.33\times 120\times 25=16{,}000$ lb/ft (26,000 kg/m) or 1,333 lb/in. (260 kg/cm).

Also assume $E=29{,}000{,}000$ psi; $E'=700$ psi; $K=0.100$; $D_e=1.50$; and $r=31$ in. The moment of inertia of 12-gage structural plate (Table XXIX) is $I=0.05455$: and $EI=1{,}581{,}950$.

Then, by Equation 15,

$$\triangle x=\frac{1.50\times 0.1\times 1{,}333\times 31^3}{1{,}581{,}950+0.061\times 700\times 31^3}=2.09 \text{ in. } (5.31 \text{ cm}).$$

3. Plastic Pipes

Several of the many types of plastics have been used for pipe of various classes. The wide range of physical properties of plastics makes possible the production of both flexible and rigid pipe, depending on the materials used. Some, such as plastic truss pipe, are semi-rigid. If semi-rigid pipe can deflect an allowable amount without failing, Equation 15 may be used in the strength calculations.

Plastic pipe technology is developing rapidly and new methods for structural design can be expected. For the present, in computing plastic pipe strength, a deflection lag factor of 1.50 and a bedding constant of 0.10 are suggested. A modulus of soil reaction, E', equal to 300 psi (21 kg/sq cm) is recommended when side fills are compacted by hand tamping to 65 percent of maximum density, AASHO T99, but an E' of 700 psi (49 kg/sq cm) can be used when the side fills are compacted properly by mechanical equipment to 90 percent or more of maximum density AASHO T99.

The suggested design values have been verified by laboratory and field-scale testing, but it must be remembered that, for a given pipeline installation, conditions may vary from one section to another. The deflection equation should be applied as a guide to design rather than an absolute rule, and the amount of deflection permitted in the installed pipe will depend on the factor of safety required by the design engineer. Several types of flexible plastic pipe are capable of undergoing parallel plate test deflections of more than 20 percent without cracking or other distress. Consideration should be given to allowable deflection so as not to impede cleaning operations in sewers or seriously impair flow capacity. For these types, a field supporting strength resulting in a deflection of five percent may be used.

F. FACTOR OF SAFETY

1. General

The term factor of safety is considered by most engineers, for any material, to be the amount the ultimate strength of the material is reduced to calculate the working strength used for design. Therefore, the factor of safety generally is independent of any technique used to determine the loads to be imposed on the material.

This definition is applicable to rigid sewer pipe where, for a given ultimate strength and a given factor of safety, the working strength remains constant regardless of any other conditions.

This definition is not, however, applicable to the design of flexible sewer pipe. Flexible sewer pipe may deflect as much as 20 percent of its original diameter before failure. However, a sewer pipe which has deflected to such an extent is no longer serviceable. In the case of flexible sewer pipe, the ultimate load which the installed conduit will support while retaining complete serviceability, under given conditions of bedding and backfilling, is considered to be the ultimate strength of the pipe. This load is termed the field supporting strength of the pipe. In the design of flexible sewer pipes, therefore, the factor of safety is the amount the field supporting strength is reduced to calculate the working, or safe supporting, strength of the pipe.

The factors of safety discussed herein are not to be used to compensate for poor inspection and construction. It is mandatory that design assumptions be realized in construction if pipe failures are to be prevented.

2. Rigid Pipes

Ultimate strengths of rigid pipe usually are measured in terms of the ultimate three-edge bearing strength for plain pipe, and of ultimate and 0.01-in. (0.025-cm) crack, three-edge bearing strengths for reinforced concrete pipe. Therefore, the specified minimum strength by the three-edge bearing method divided by the appropriate factor of safety gives the working strength in terms of three-edge bearing.

A factor of safety of at least 1.5 should be applied to the specified minimum ultimate three-edge bearing strength to determine the working strength for all rigid pipes.

3. Flexible Pipes

Flexible pipes are considered to have reached the limit of their serviceability when a deflection of five percent is attained. Therefore, the field supporting strength for flexible pipes is taken to be the load which produces the maximum deflection of five percent. A factor of safety of 1.25 should be applied to the field supporting strength to calculate the safe supporting strength of the flexible pipe.

G. DESIGN RELATIONSHIPS

1. Rigid Pipes

The various elements in the design of a rigid sewer have been discussed separately. Their combination into a safe and economical design may be expressed as follows:

$$\text{Safe supporting strength} = \frac{\text{field supporting strength}}{\text{factor of safety}}$$

in which field supporting strength equals the three-edge bearing strength times the load factor, or

$$\text{Safe supporting strength} = \frac{\text{three-edge bearing strength} \times \text{load factor}}{\text{factor of safety}}$$

and, since the three-edge bearing strength divided by the factor of safety is the working strength,

$$\text{Safe supporting strength} = \text{working strength} \times \text{load factor}$$

Also, since safe supporting strength is equal to the maximum allowable field load,

$$\text{Required three-edge bearing strength} = \frac{\text{maximum allowable field load} \times \text{factor of safety}}{\text{load factor}}$$

Example 12.

(*a*) Refer to *Example 1* wherein the backfill load on a 24-in. diam pipe with 14 ft of cover was found to be 4,720 lb/ft. If vitrified clay pipe is to be specified and a factor of safety of 1.5 is selected, the design load will be $4{,}720 \times 1.5 = 7{,}080$ lb/ft (10,560 kg/cm).

The crushing strength requirement of 24-in. diam extra-strength clay sewer pipe (ASTM C200) by the three-edge bearing method is 4,400 lb/ft. Dividing this into 7,080 lb/ft (the design load), the minimum required load factor, $7{,}080/4{,}400 = 1.61$, is obtained.

Figure 58 indicates that a Class B bedding is required for this installation.

(*b*) Refer to *Example 3*. If a 24-in. diam reinforced concrete sewer pipe (one line of reinforcement near center of wall) and a factor of safety of 1.50 based on the minimum ultimate test strength are selected, the design load will be $1.50 \times 5{,}750 = 8{,}620$ lb/ft (12,900 kg/m).

If the minimum ultimate test strength of the pipe is 6,000 lb/ft (ASTM C76, Class IV—3,000 D), the required load factor will be $8{,}620/6{,}000 = 1.44$. According to Figure 58, this installation will require a Class C bedding.

Example 13. Assume the 48-in. diam pipe in *Example 5* is bedded on an unreinforced-concrete cradle, with $p = m = 0.5$ and $k = 0.33$. Using a factor of safety of 1.5 based on the ultimate three-edge bearing strength of the pipe, $\frac{H}{B_c} = 6.63$; $C_c = 10.0$; $x = 0.856$; and $N = 0.575$.

By Equation 14,

$$q = \frac{0.5 \times 0.33}{10.0}(6.63 + 0.25) = 0.114;$$

and, by Equation 13,

$$L_f = \frac{1.431}{0.575 - (0.856 \times 0.114)} = 3.01$$

The required three-edge bearing strength at ultimate load is $\frac{29{,}100 \times 1.5}{3.01}$ $= 14{,}500$ lb/ft (21,600 kg/m) or 3,630 D (17,720 D in kg/m/diam). Use ASTM C76, Class V pipe.

2. Flexible Pipe

The combination of elements of design for a flexible pipe are stated below:

Field supporting strength = load producing five percent deflection

$$\text{Safe supporting strength} = \frac{\text{field supporting strength}}{\text{factor of safety}}$$

H. CHARTS FOR DETERMINING EARTH LOADS ON BURIED CONDUITS

Various tables and charts have been developed which allow direct and convenient determination of earth loads on buried conduits. It should be emphasized, however, that the designer should have a full understanding of the fundamental factors which determine the structural requirements of a sewer so that sound engineering judgment may be applied to the design.

One such method of computing the earth load on conduits involves the use of a set of curves of the type shown in Figure 61. These curves were developed for loads on reinforced concrete pipe buried in sand and gravel. Its practical value is illustrated in the example below:

Example 14. Determine the load on a 30-in. diam reinforced concrete pipe under 14 ft of sand and gravel cover in trench conditions if the width of trench at the top of conduit is 6 ft.

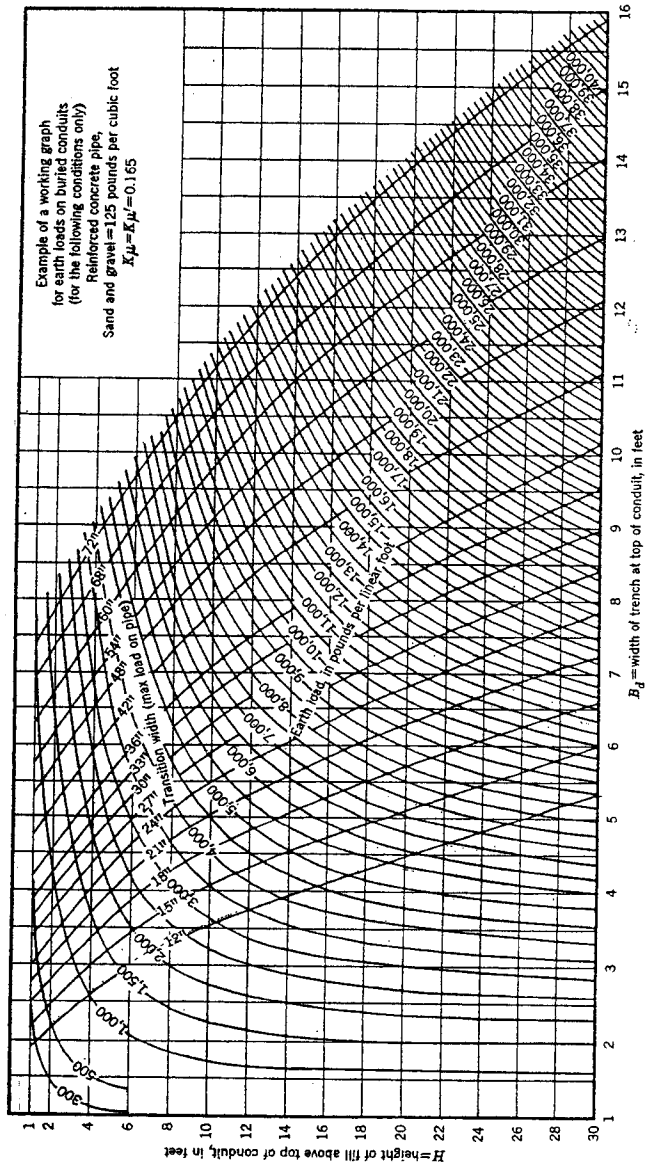


FIGURE 61.—Example of working graph for earth loads on buried conduits. (In. × 2.54 = cm; ft × 0.3 = m.)

Figure 61 gives the earth loads on conduits in lb/ft units for various trench widths and depths of sand and gravel backfill. Also plotted on these curves are the transition widths (trench widths at which further widening will have no effect on the load on the pipe) for each size of pipe.

For a fill height of 14 ft and a width of trench at the top of conduit of 6 ft, the load on the pipe is 7,250 lb/ft.

If a wide trench were used, the maximum load on the pipe would be 8,900 lb/ft at trench width (B_d) of 7 ft (7 ft is the transition width for 30-in. pipe with $H=14$ ft). Any further increase in the width of trench will not increase the load on the pipe.

The load per lineal foot determined from Figure 61 for the conditions given is the earth or dead load only. To this load must be added any live or superimposed load.

Following the determination of the total load and the selection of the proper factor of safety, the type of bedding and strength of pipe can be determined.

I. RECOMMENDATIONS FOR ATTAINING DESIGN LOAD SUPPORTING STRENGTH

1. Factor of Safety

The factor of safety against ultimate failure is generally at least 2.5 in the design of most engineering structures of monolithic concrete. The factor of safety of pipe sewers against ultimate collapse is considerably less. It is, therefore, important to guarantee that the loads imposed on the sewer are not greater than the design loads. To attain this objective the following procedures are recommended:

(*a*) Specifications should set forth limits for the width of trench below the top of pipe. The width limits should take into account the minimum width required to lay and joint pipe and the maximum allowable for each class of pipe and bedding to be used. Where the depth is such that a positive projecting condition will be obtained, maximum width should be specified as unlimited unless the width must be controlled for some reason other than to meet structural requirements of the pipe. Appropriate corrective measures should be specified in the event the maximum allowable width is exceeded. These may include provision for a higher class of bedding or concrete encasement. Maximum allowable construction live loads should be specified for various depths of cover if appropriate for the project.

(*b*) Construction should be observed by an experienced engineer or inspector who reports to a competent field engineer.

(*c*) Pipe testing should be under the supervision of a reliable testing laboratory and close liaison should be maintained between the laboratory and the field engineer.

(*d*) The field engineer should be furnished with sufficient design data to enable him to evaluate unforeseen conditions intelligently; and he should be instructed to confer with the design engineer if changes in design appear advisable.

(*e*) Where sheeting is to be removed, pulling should be done in stages, making certain that the space formerly occupied by the sheets is backfilled completely.

2. Effect of Trench Sheeting

Because of the various alternate methods employed in sheeting trenches, generalizations as to the proper construction procedure to follow to insure that the design load is not exceeded are risky and dangerous. Each method of sheeting and bracing should be studied separately. The effect of a particular system on the load on the conduit as well as the consequences of removing the sheeting or the bracing must be estimated.

It is difficult to obtain satisfactory filling and compaction of the void left by the pulling of wood sheeting. Sheeting driven alongside the pipe should be cut off and left in place to an elevation 1.5 ft (45 cm) above the top of the conduit.

If granular materials are used for backfill it is possible to fill and compact the voids left by the wood sheets if the material is placed in lifts and jetted as the sheeting is pulled. If cohesive materials are used for backfill, a void will be left by the pulling of the wood sheets and the full weight of the prism of earth contained between the sheeting will come to bear on the conduit.

Skeleton sheeting or bracing should be cut off and left in place to an elevation 1.5 ft (45 cm) over the top of the pipe if removal of the trench support might cause a collapse of the trench wall and a widening of the trench at the top of the conduit. Entire skeleton sheeting systems should be left in place if removal would cause collapse of the trench before backfill can be placed.

Where steel "soldier beams" with horizontal lagging between the beam flanges are used for sheeting trenches, efforts to reclaim the steel beams, before the trench is backfilled, may damage pipe joints. It is recommended that this type of sheeting be allowed when the beams are pulled after backfilling and the lagging is left in place.

Steel sheeting may be used and reused many times, and the relative economy of this type as compared with timber or timber and "soldier beams" should be explored. Because of the thinness of the sheets, it is often feasible to achieve reasonable compaction of backfill so that the steel sheets may be withdrawn with about the same factor of safety against settlement of the surfaces adjacent to the trench as that for other types of sheeting left in place.

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CHAPTER 10. CONSTRUCTION CONTRACT DOCUMENTS

A. INTRODUCTION

The purpose of the contract documents is to portray clearly by words and drawings the nature and extent of the work to be performed and the conditions known or anticipated under which the work is to be executed. Most sewer construction projects are accomplished by contracts entered into between an owner and a construction contractor and the contract documents constitute the construction contract.

Frequently, the work is divided into various items, with either unit price or lump sum bids received for each item of work. The contract documents must clearly describe and limit these items to obviate all possible confusion in the mind of the bidder with regard to methods of measurement and payment. The subdivision of the work often is based on local customs or the customs and conventions of the designing engineer.

Lump sum bids have been applied most generally to special structures which are detailed completely and not subject to alteration or quantity changes during construction. A schedule of unit adjustment prices may be included in the proposal to provide a basis for payment in the event that changes are necessary in lump sum bid items.

Unit price bids have been used most generally where quantities of work are likely to be adjusted or varied during construction. Lineal feet of sewers or manholes, and cubic yards of rock excavation or concrete cradle are examples of such unit price items.

Lump sum bids may be taken for entire sewer construction contracts where the contract documents define the work with sufficient completeness to permit the bidder to make an accurate determination of the quantities of work. Such contracts may contain unit adjustment prices for items of work, such as rock excavation, piles, additional excavation, selected fill material, and sheeting requirements which cannot be determined accurately beforehand. This method may, however, lead to non-competitive quotations for unit adjustment prices. To prevent this, the amount of the unit adjustment price may be stipulated in the proposal or an appropriate quantity of the unit price work may be included for comparison of bids. The administration of the project, provided extensive changes are not made during construction, is simplified in the lump sum type of contract.

Plans and specifications are supplementary to each other, and all work portrayed in either is considered to be a part of the contract.

COPY OF APPROPRIATE SECTIONS FOR:

Selig, E.T., "Soil Properties for Plastic Pipe Installations", Buried Plastic Pipe Technology, ASTM STP1093, Buczala and Cassady, Eds., 1990

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Ernest T. Selig

SOIL PROPERTIES FOR PLASTIC PIPE INSTALLATIONS

REFERENCE: Selig, E. T., "Soil Properties for Plastic Pipe Installations," Buried Plastic Pipe Technology, ASTM STP 1093, George S. Buczala and Michael J. Cassady, Eds., American Society for Testing and Materials, Philadelphia, 1990.

ABSTRACT: Soil property requirements for the basic trench and embankment installation conditions are discussed. Characteristics of compacted soils are described and representative stress-strain parameters given. Preliminary values of existing ground stiffness properties are suggested. The applications of these properties for analyzing pipe deflection, wall thrust and buckling strength are indicated.

KEYWORDS: soil properties, stress-strain behavior, strength, compaction, flexible pipe, plastic pipe, Young's modulus, Poisson's ratio, bulk modulus, constrained modulus, deflection, buckling, wall thrust.

INTRODUCTION

The installed shape of a buried plastic (flexible) pipe is strongly influenced by the soil placement process and the resulting soil stiffness properties. The long-term pipe deflections are controlled by soil deformation subsequent to installation in addition to the time-dependent pipe response. This soil deformation results from soil consolidation, creep, moisture changes, and erosion, as well as from loading changes. Pipe buckling stability is highly dependent on the value of soil stiffness. The pipe wall stresses and strains induced by earth and live loading are dependent on the relative stiffness of the soil and pipe. The type of soil and level of compaction are the fundamental factors determining these characteristics for placed soils. The soil type, in situ state, and stress history are the corresponding factors determining the relevant characteristics for undisturbed ground. To help illustrate these principles the relationships between soil type, amount of compaction and compaction effort will be discussed and their influence on resulting soil properties will be shown. The role of these soil properties in analyzing plastic pipe deflection, wall thrust, and buckling stability will be indicated.

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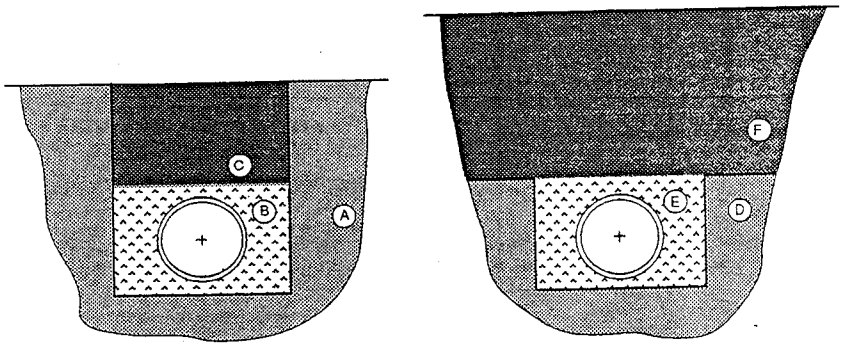


a) Trench b) Embankment

Fig. 1 -- Pipe installation type.

INSTALLATION TYPE

The two basic plastic pipe installation type are shown in Fig. 1. The trench case (Fig. 1a) represents a situation in which the existing ground (zone A) is excavated to the depth required for pipe installation. The resulting trench is backfilled with two zones of compacted soil. Zone B is the zone immediately surrounding the pipe which requires certain restrictions on the placement and compaction to avoid distressing the pipe, and restrictions on the type of soil to provide needed stiffness and stability. The remainder of the trench (zone C) is usually filled with the excavated soil appropriately placed and compacted. The specific trench dimensions as well as the dividing line between zones B and C depend on the requirements of the installation.

The embankment case (Fig. 1b) shows the pipe installed in a shallow trench excavated in the existing ground (zone D) and backfilled with zone E material meeting requirements similar to those of zone B. An earth embankment (zone F) is then constructed on top of the existing ground. This configuration is known as a negative projecting embankment pipe installation [1]. The pipe may also be installed above the existing ground, in which case zone E is laterally supported by embankment soil in zone F rather than by existing ground.

The soil property requirements for plastic pipe design are different in various ways for each zone in Fig. 1.

SOIL REQUIREMENTS

Existing Ground

In the case of existing ground in zone A the stress level remains essentially unchanged by the pipe installation. The main requirement is stability of the trench walls and bottom during construction. This is provided as needed by bracing and dewatering. Unless the existing ground is unsuitable, as may be the case with peats and organic deposits, the existing soil properties are accepted and the design and construction are carried out considering these properties. For analyzing the soil-pipe interaction, soil strength and stiffness during filling of the trench are the primary parameters required for zone A soil.

The requirements are different for existing ground in zone D because the stresses are significantly increased by construction of the embankment. It is necessary to insure that the ground is stable under the weight of the embankment and that excessive immediate and consolidation settlement will not occur. If the soil in zone D is not saturated then volumetric compression will occur under the embankment load. Whether or not the soil is saturated, shear strains will occur under the embankment load. Both of these characteristics result in immediate settlement. If the soil is saturated or becomes so because of compression under increased load, then consolidation will take place over a period of time after construction as the excess pore water pressure is dissipated. Thus for zone D soil knowledge of the strength and consolidation characteristics is required as well as the nonlinear stress-strain properties during construction.

Soil Envelope

Zones B and E which immediately surround the pipe will be termed the soil envelope. This envelope includes the bedding, the side fill, and the top fill (Fig. 2). The haunch zone is included within the bedding and side fill as shown in Fig. 2. Zones B and E will be considered together because their required properties are essentially the same.

The stability of flexible plastic pipe is substantially controlled by the properties of the material in the soil envelope. The following are the requirements of this envelope:

1. Constructability - ability to be placed and compacted to the desired properties without distorting the pipe.

2. Provide the stiffness needed for limiting the pipe deformations (the particularly important areas are those shown by arrows A and B in Fig. 3a).

3. Provide the stiffness needed to achieve adequate pipe buckling strength.

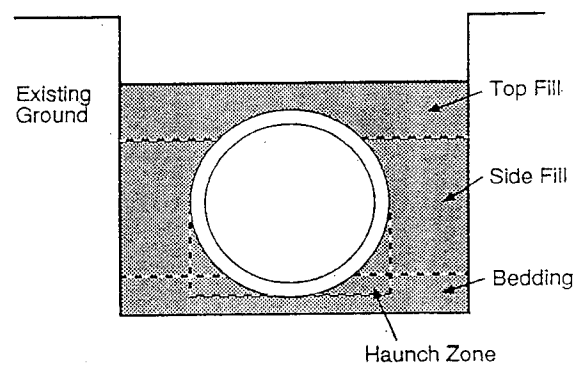


Fig. 2 -- Soil envelope.

4. Be stable under long-term moisture changes.

5. Exhibit little creep and consolidation deformation.

6. Provide drainage of excess pore water pressure.

7. Reduce the earth and live load carried by the pipe wall.

8. Prevent erosion or piping of surrounding fine soil as a result of pipe leaks or ground water movement.

These soil envelope requirements dictate the use of compacted coarse-grained soils (mainly sand and gravel components) in most cases. The material in the envelop thus may be referred to as structural backfill.

Trench Backfill

Zone C represents the trench backfill remaining above the structural backfill zone B. If a pavement or a structure requiring limited settlement is to be placed on the surface above the trench, then zone C soil must provide firm support (arrow C in Fig. 3a). Suitable material adequately compacted for zones B and C will be needed to prevent settlement as shown in Fig. 3b. The main mechanisms of settlement in zone C are: 1) volume reduction and shear strain from the surface load, particularly from repeated wheel loading, and 2) shrinkage from cycles of moisture change. These problems diminish with increased level of compaction, but even so soils whose behavior is controlled by fine-grained (silt and clay) components generally will not perform satisfactorily in this application. Thus coarse-grained soils (sand and gravel components) are most appropriate.

When surface settlement is not a concern, then zone C may be backfilled with the excavated soil using appropriate compaction. This is the most economical solution.

Embankment

In a negative projecting installation (Fig. 1b), the embankment, zone F, acts primarily as dead load. However some arching of the embankment load will occur which results in the pressure applied to

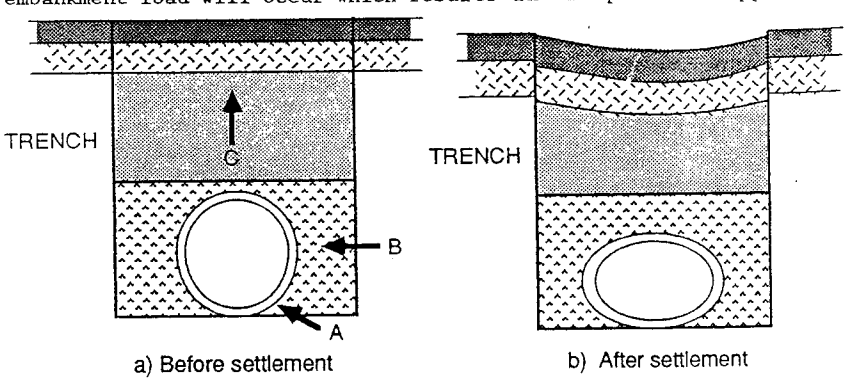


Fig. 3 -- Settlement with too little compaction.

the top of zone E being either more or less than the average pressure at the base of the embankment. The unit weight of embankment fill is thus its most important property. Also important is the soil stiffness in the lower part of the embankment, i.e., within 3 trench widths of the top of zone E.

If the pipe were installed in either a positive projecting or imperfect trench condition [1], then the embankment stiffness properties would become much more important.

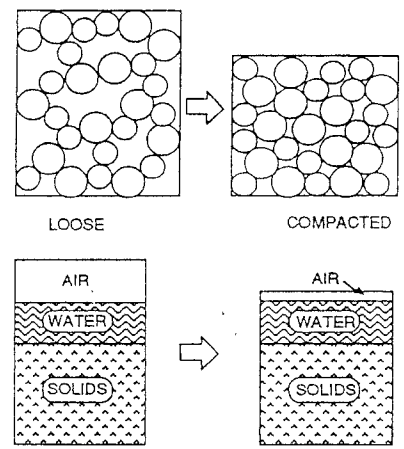


Fig. 4 -- Illustration of compaction.

CHARACTERISTICS OF COMPACTED SOILS

Compaction Reference Test

Compaction is immediate densification of soil by mechanical means. The water content remains constant and the void air space is reduced (Fig. 4). Consolidation, in contrast, is gradual squeezing out of water from saturated soils (no air in voids) which results in some densification.

Compaction is performed to achieve suitable properties of soil being placed. Increasing the amount of compaction increases strength, decreases compressibility, decreases permeability, reduces collapse potential, and reduces swelling and shrinking with moisture change. The magnitude of these effects depends on the soil type.

Standardized tests by ASTM and AASHTO are used to determine the amount of compaction that can be achieved for each soil with specified standard compaction efforts. For cohesionless, free-draining material (clean sands and gravels) the soil is vibrated vertically in a rigid mold with a surcharge weight placed on the soil surface (ASTM Test for Maximum Index Density of Soils Using a Vibratory Table D4253) as illustrated in Fig. 5a. The maximum density achieved is used as a reference for field compaction.

percent compaction is achieved, the resulting stiffness and strength properties are not the same for all soils. This results in a dramatic difference in stiffness among soils when related to compaction effort as illustrated in Fig. 8b. Quantitative examples of these comparisons may be found in Refs. [2-4]. These characteristics are not generally considered in compaction specifications because the same percent compaction is commonly specified regardless of the backfill soil type.

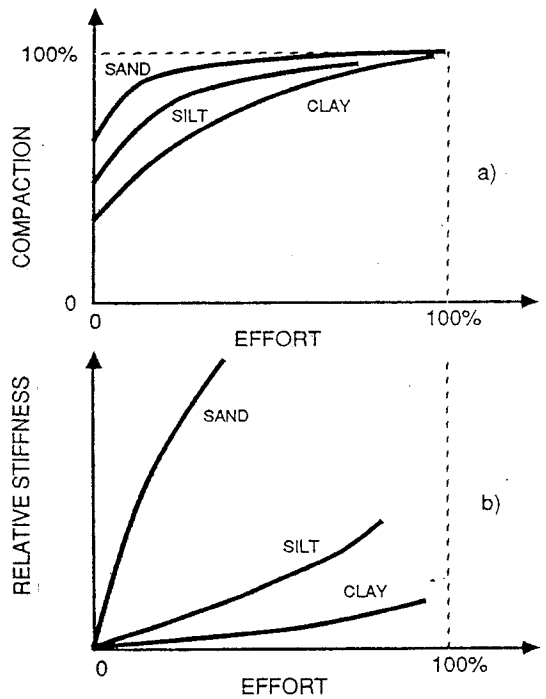


Fig. 8 -- Effect of soil type on variation of percent compaction and soil stiffness with compaction effort.

The relative compactability illustrated in Fig. 8 is very important in flexible pipe installation, because, for a given soil stiffness required to support the pipe, the less the required compaction effort the less the pipe distortion during placement of the soil envelope. This is one of the reasons for using coarse-grained soils for the envelope.

Changes After Compaction

When soils are subject to wetting and drying cycles after compaction, they will decrease in volume over time from the effects of the water. With increasing compaction the magnitude of this effect diminishes. The magnitude of volume change is much more significant for clays than for silts, and for silts it is much more significant than for sands.

The strength and stiffness of any soil will be higher when compacted at water contents less than optimum, than at optimum, but clay soils will swell more if the water content should increase later. This will cause a reduction in strength and stiffness. Conversely strength and stiffness will be lower when compacted at water contents higher than optimum, but fine-grained soils, especially clays, will shrink more upon drying. Compaction of soils that are too wet should be avoided because low strength and stiffness will result.

When soils are placed loosely around buried pipe they are subject to substantial volume reduction if they should become saturated. This phenomenon, known as collapse, will result in pipe deflection after construction. The reason for this behavior is that loosely placed soils are unsaturated and develop their resistance to deformation from effective stress induced by capillary water tension. When these soils become saturated the capillary tension is lost, causing the soil particles to settle into a denser packing.

The collapse characteristic is illustrated in Fig. 9 from tests on a silty sand. To perform the test the soil first was lightly compacted at around optimum moisture content in an oedometer. For one test (dashed curve) the soil was loaded in steps and then unloaded with the moisture content remaining at around optimum. In the other test (solid curve) the sample was loaded at optimum moisture content to 3.5 psi (24 kPa) and then allowed to saturate. As water entered the sample a sudden large strain occurred under constant load. Further loading while saturated gradually produced additional strain as in the moist sample case.

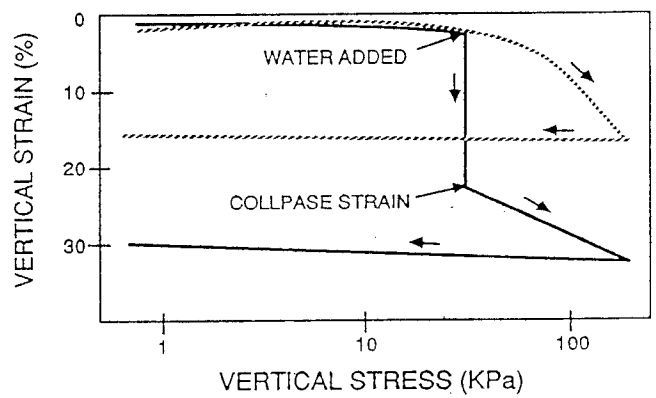


Fig. 9 -- Collapse of lightly compacted silty sand from soaking.

Tests on a variety of specimens showed that the magnitude of the collapse strain decreased as the amount of compaction increased, and diminished to an insignificant amount when the percent compaction reached about 85 to 90% D698 or about 85% D1557 maximum dry density.

Another cause of pipe deformation after construction is migration of fine soil particles from the trench walls into the soil envelope.

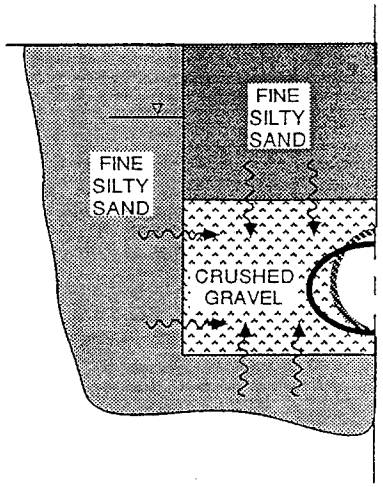


Fig. 10 -- Migration of fine soil into coarse soil envelope.

This occurs as a result of groundwater movement when the soil envelope gradation is much coarser than that of the existing ground. An example is given in Fig. 10. This problem should not occur if the traditional filter criteria [5] are used in selecting the soil envelope gradation or if a geotextile (filter fabric) is used to provide proper separation.

PROPERTIES OF COMPACTED SOILS

Finite element analysis has been shown to be a good approach for evaluating soil-pipe interaction. Duncan [6] has proposed the use of a hyperbolic model for representing the non-linear, stress-dependent soil behavior. Modifications to this model were proposed by Boscardin, Selig, Lin and Yang [3] and design parameters determined for a variety of soils and compaction levels from laboratory tests [2]. These parameters were modified for flexible pipe by Haggag [4]. The values were then extended to other soil type and compaction levels by the writer and incorporated into CANDE [7]. The values are given in Table 1. These parameters may be used to calculate tangent Young's modulus and bulk modulus as a function of stress state using the appropriate equations in the literature [2,3].

The linear-elastic model is a special case of the hyperbolic model in which the parameters are constant, independent of stress state. This is the simplest model for representing soil behavior in soil-pipe interaction analysis. Two independent elastic constants are needed. The choice is normally from among Young's modulus (E_s), bulk modulus (B), Poisson's ratio (ν_s), and shear modulus (G). Values of Young's modulus were estimated from the hyperbolic model for various values of maximum principal stress (σ_1) with the minimum principal stress (σ_3) equal to one-half to one times the maximum principal

Table 1 -- Recommended hyperbolic parameters for compacted soils.

Tested Soil	Soil Type USCS	Soil Type AASHTO	% T-99	% T-180	Soil No.	Wet Density (lb/ft³)	Wet Density (Mg/m³)	K	n	R_f	c (psi)	c (kPa)	ϕ_o (deg)	$\Delta\phi$ (deg)	B_i/P_a	ε_u
Gravelly Sand (SW)	SW, SP, GW, GP	A1, A3	100	95	27	148	2.37	1300	0.90	0.65	0	0	54	15	272.0	0.007
			95	90	21	141	2.25	950	0.60	0.70	0	0	48	8	187.0	0.014
			90	85	1	134	2.14	640	0.43	0.75	0	0	42	4	102.0	0.036
			85	80	22	126	2.02	450	0.35	0.80	0	0	38	2	31.8	0.057
			80	75	2	119	1.90	320	0.35	0.83	0	0	36	1	15.3	0.078
			61	59	3	91	1.46	54	0.85	0.90	0	0	29	0	4.3	0.163
Sandy Silt (ML)	GM, SM, ML; Also GC, SC with < 20% passing #200 seive.	A2, A4	100	95	28	134	2.14	800	0.54	1.02	5.5	38	36	0	197.5	0.021
			95	90	23	127	2.03	440	0.40	0.95	4	28	34	0	120.8	0.043
			90	85	4	120	1.92	200	0.26	0.89	3.5	24	32	0	46.0	0.071
			85	80	24	114	1.82	110	0.25	0.85	3	21	30	0	23.8	0.100
			80	75	5	107	1.71	75	0.25	0.80	2.5	17	28	0	12.8	0.134
			49	46	6	66	1.06	16	0.95	0.55	0	0	23	0	3.3	0.305
Silty Clay (CL)	CL, MH, GC, SC	A5, A6	100	90	29	125	2.00	170	0.37	1.07	11	76	12	0	81.3	0.064
			95	85	25	119	1.90	120	0.45	1.00	9	62	15	4	53.0	0.092
			90	80	7	112	1.79	75	0.54	0.94	7	48	17	7	25.5	0.121
			85	75	26	106	1.69	50	0.60	0.90	6	41	18	8	13.0	0.149
			80	70	8	100	1.60	35	0.66	0.87	5	34	19	8.5	8.8	0.178
			45	40	9	56	0.90	16	0.95	0.75	0	0	23	11	1.8	0.391
	CH	A7	100	90	7	112	1.79	75	0.54	0.94	7	48	17	7	25.5	0.121
			95	85	26	106	1.69	50	0.60	0.90	6	41	18	8	13.0	0.149
			90	80	8	100	1.60	35	0.66	0.87	5	34	19	8.5	8.8	0.178
			45	40	9	56	0.89	16	0.95	0.75	0	0	23	11	1.8	0.391

stress. The hyperbolic parameters used were those in Table 1. Values of bulk modulus were estimated in the same manner. Then Poisson's ratio, ν_s, was derived from the relationship

$$\nu_s = 0.5\ (1 - \frac{E_s}{3B}) \quad .$$

The resulting parameter values are given in Table 2.

Table 2 -- Elastic soil parameters.

Soil Type: SW, SP, GW, GP						
Stress level	95% D698			85% D698		
psi (kPa)	E_s	B	ν_s	E_s	B	ν_s
1 (7)	1600 (11)	2800 (19)	0.40	1300 (9)	900 (6)	0.26
5 (34)	4100 (28)	3300 (23)	0.29	2100 (14)	1200 (8)	0.21
10 (70)	6000 (41)	3900 (27)	0.24	2600 (18)	1400 (10)	0.19
20 (140)	8600 (59)	5300 (37)	0.23	3300 (23)	1800 (12)	0.19
40 (280)	13000 (90)	8700 (60)	0.25	4100 (28)	2500 (17)	0.23
60 (410)	16000 (110)	13000 (90)	0.29	4700 (32)	3500 (24)	0.28

Soil Type: GM, SM, ML, and GC, SC with < 20% fines						
Stress level	95% D698			85% D698		
psi (kPa)	E_s	B	ν_s	E_s	B	ν_s
1 (7)	1800 (12)	1900 (13)	0.34	600 (4)	400 (3)	0.25
5 (34)	2500 (17)	2000 (14)	0.29	700 (5)	450 (3)	0.24
10 (70)	2900 (20)	2100 (14)	0.27	800 (6)	500 (3)	0.23
20 (140)	3200 (22)	2500 (17)	0.29	850 (6)	700 (5)	0.30
40 (280)	3700 (25)	3400 (23)	0.32	900 (6)	1200 (8)	0.38
60 (410)	4100 (28)	4500 (31)	0.35	1000 (7)	1800 (12)	0.41

Soil Type: CL, MH, CC, SC						
Stress level	95% D698			85% D698		
psi (kPa)	E_s	B	ν_s	E_s	B	ν_s
1 (7)	400 (3)	800 (6)	0.42	100 (1)	100 (1)	0.33
5 (34)	800 (6)	900 (6)	0.35	250 (2)	200 (1)	0.29
10 (70)	1100 (8)	1000 (7)	0.32	400 (3)	300 (2)	0.28
20 (140)	1300 (9)	1100 (8)	0.30	600 (4)	400 (3)	0.25
40 (280)	1400 (10)	1600 (11)	0.35	700 (5)	800 (6)	0.35
60 (410)	1500 (10)	2100 (14)	0.38	800 (6)	1300 (9)	0.40

Note: Units of E_s and B are psi (MPa).

Deflections of buried flexible pipe are commonly calculated using the Iowa formula [1] which uses the modulus of soil reaction (E') as the parameter representing soil stiffness. Since E' is not a directly measureable soil parameter, but must be determined by back-calculation using observed pipe deflections, studies have been carried out to seek a correlation between E' and soil stiffness parameters such as Young's modulus (E_s) and constrained modulus (M_s), where E_s and M_s are related

through Poisson's ratio (ν_s) by

$$M_s = \frac{E_s\ (1 - \nu_s)}{(1 + \nu_s)(1 - 2\nu_s)} \quad . \qquad (1)$$

These studies [8-10] and analysis by the writer indicate that for

$$E' = k\ M_s \quad , \qquad (2)$$

the value of k may vary from 0.7 to 2.3, with k = 1.5 as a representative value. For ν_s = 0.3, combining Eqs. 1 and 2 gives

$$E' = 2E_s \quad , \qquad (3)$$

although the factor k could easily be higher than a value of 2.

The E' values developed by Howard [11] based on back-calculation from field observations may be converted to E_s values for comparison with the values in Table 2 for σ_1 = 5 to 10 psi (34 to 69 kPa). The comparison is as follows for compaction levels of 85 to 95% D698:

	E_s (psi/MPa)	
Soil Type	Howard	Table 2
CL	200/1.4	250-1100/1.7-7.6
ML	500/3.5	700-2900/4.8-20
SW	1000/7	2100-6000/14-41

PROPERTIES OF EXISTING GROUND

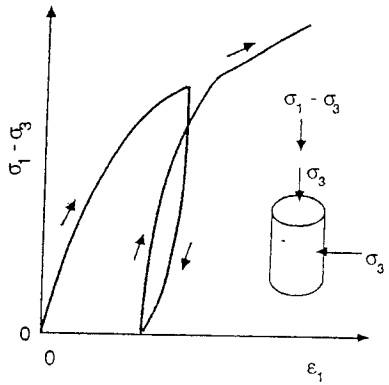
A thorough review of the characteristics of existing ground is beyond the scope of this paper, and indeed encompasses most of the field of soil behavior. The complexity of soil behavior is part of the problem in defining the required soil properties for analysis. Equally critical is the spatial variability of natural soils combined with the practical necessity to estimate the properties from a very limited amount of sampling and testing.

Time-dependent stress-strain response, characterized by consolidation and creep, is often an important consideration for existing ground. However, the present state-of-the-art does not provide means for incorporating this response in pipe design except with very rough approximations.

A typical static-triaxial test stress-strain curve with unloading and reloading is illustrated in Fig. 11. This figure shows that unloading and reloading behavior is considerably more linear than the primary loading curve. This observation together with the recognized complexities of existing ground already discussed has resulted in approximating existing ground in zone A (Fig. 1) by constant modulus values representing linear elastic behavior. This approach is not as satisfactory for zone D (Fig. 1) because the stress-strain relationships may be very non-linear. If the embankment loading

produces stresses well above those previously experienced by thenatural ground (considering stress history) then nonlinear modeling such as used for compacted soil may be desired.

Fig. 11 -- Static triaxial test results: increase in axial strain with increase in axial stress.



The existing ground parameters proposed for concrete pipe design [12] are listed in Table 3. These are preliminary estimates which need considerable refining by more study.

Table 3 -- Existing ground properties.

Material	Wet density (pcf)	Wet density (Mg/m^3)	E_s (psi)	E_s (Mpa)	ν_s
1. Coarse-grained					
A. Dense	145	2.32	10000	69	0.49
B. Medium	130	2.08	6000	41	0.35
C. Loose	115	1.84	2000	14	0.20
2. Fine-grained					
A. Stiff	125	2.00	6000	41	0.3
B. Firm	118	1.89	3500	24	0.4
C. Soft	110	1.76	1000	7	0.49
3. Concrete	150	2.40	$3x10^6$	$21x10^3$	0.17
4. Rock					
A. Weak	145	2.32	$0.1x10^6$	700	0.2
B. Competent	160	2.56	$5x10^6$	$34x10^3$	0.3

APPLICATIONS OF SOIL PROPERTIES

There are three common calculations in pipe design using soil properties: 1) deflection, 2) wall thrust, and 3) buckling strength. Examples of each will be given to illustrate the use of soil properties.

Deflection

The use of the Iowa formula to calculate pipe deflection has already been mentioned. The deflection given is the horizontal diameter change produced by earth load placed above the crown of the pipe. Deflection caused by placing the soil envelope around the pipe is not included in the Iowa formula. The earth load needs to consider arching action caused by the installation conditions, for example the difference between trench and embankment as shown in Fig. 1. The required soil parameter (really a soil-structure interaction parameter) is E'. Design values of E' may be estimated from the Howard table [11], or from experience with similar installations.

An alternative approach which uses the conventional soil properties E_s and ν_s is the elasticity solution by Burns and Richard [13]. As for the Iowa formula, the deflection is just for earth load above the crown, which also needs to be adjusted for arching because the solution is based on a pipe deeply buried in a homogeneous soil and subjected to uniform surface pressure. The Burns and Richard solution not only provides horizontal pipe deflection, but also pipe deflection, wall thrust, bending moment and radial pressure at any point on the circumference for both no-slip and frictionless conditions at the soil-pipe interface. Soil properties may be estimated by: 1) using values in Table 2, 2) conducting field or lab tests on representative soil, or 3) back calculation with the elasticity solution for similar installations. The Burns and Richard solution is available as part of the CANDE computer program [7,14].

In critical or unusual cases more precise deflection analysis may be performed using finite element methods such as in CANDE. The soil may be represented by properties in Tables 1 and 3, unless data are available from tests on the specific soils involved. In most installations at least two zones of soil surround the pipe such as shown in Fig. 1. Only the finite element method is capable of determining the composite effect of these separate zones from a knowledge of properties of the individual zones.

Wall Thrust

Wall thrust can be estimated by the Burns and Richard solution or by the finite element methods described for the deflection analysis. The Marston-Spangler method may also be applicable [1].

Buckling Strength

Buckling strength is an important consideration in the design of buried flexible pipe. Buckling strength is normally determined for plastic pipe using equations based on some form of elastic spring soil model (Fig. 12) such as derived by Luscher [15]. The soil properties represented by the spring constant suffer the same limitation as E' in that they can not be directly measured, although approximate correlations with M_s and E_s have been proposed. Empirical corrections for depth of cover have also been suggested.

The approach representing soil as an elastic continuum (Fig. 13) is recommended as more suitable because it gives a more realistic representation of the soil-pipe interaction, it used directly

measureable soil parameters, E_s and ν_s, and it provides a means of accounting for such factors as pipe shape, shallow cover and nonhomogeneous soil conditions [16,17]. The solution is presented in the form of critical hoop (wall) thrust, N_c, which is compared with actual wall thrust to determine the factor of safety against buckling. The critical hoop thrust is given by

$$N_c = 0.55\ N_{ch}\ R_h \quad , \quad (4)$$

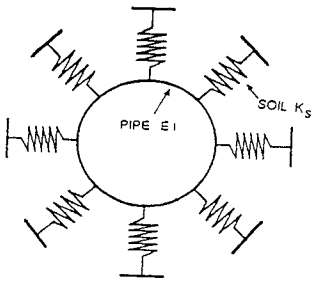


Fig. 12 -- Soil spring model for buckling.

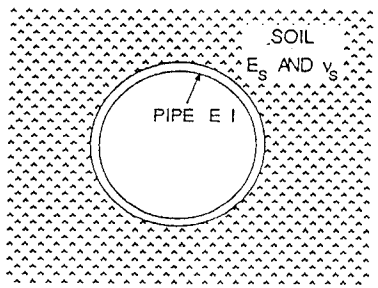


Fig. 13 -- Soil continuum model for buckling.

where R_h is a correction factor for shallow burial and nonhomogeneous soil (see examples in [4,17]), and N_{ch} is the critical thrust for a circular pipe deeply embedded in a homogeneous soil. For a smooth soil-pipe interface (conservative assumption) and for $EI/E_s^* < 0.01$, then

$$N_{ch} = 1.2\ (EI)^{1/3}\ (E_s^*)^{2/3} \quad , \quad (5)$$

where

- E = pipe Young's modulus,
- I = pipe wall moment of inertia,
- E_s^* = $E_s/(1 - \nu_s^2)$,
- E_s = soil Young's modulus,
- ν_s = soil Poisson's ratio.

For deep burial in homogeneous soil then Eq. 4 becomes

$$N_c = 0.7\ (EI)^{1/3}\ [E_s/(1 - \nu_s^2)]^{2/3} \quad . \quad (6)$$

The soil properties, E_s and ν_s, may be estimated from Table 3.

SUMMARY

The main requirements for the different soil zones encountered in buried plastic pipe installations were discussed. Characteristics of compacted soils were described, including the relative ease of compaction and the changes after compaction. Representative values of stress-strain properties were provided for compacted soils and for existing ground. Applications of these properties in analysis of pipe deflections, wall thrust and buckling stability were described.

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IS PVC PIPE STRAIN LIMITED AFTER ALL THESE YEARS?

REFERENCE: Moser, A. P., Shupe O. K., and Bishop, R.R., "Is PVC Pipe Strain Limited After All These Years," Buried Plastic Pipe Technology, ASTM STP 1093, George S. Buczala and Michael J. Cassady, Eds., American Society for Testing Materials, Philadelphia, 1990.

ABSTRACT: PVC (Polyvinyl chloride) sewer pipes have seen wide use in the United States and this has prompted concern for an appropriate material property design limit. It had been proposed that the imposition of a strain limit derived from long-term creep testing would also be appropriate for buried gravity flow pipes subjected to constant strain. Laboratory tests of pipe ring samples exposed to various strains and temperatures have been conducted for the past 13 years on filled and unfilled PVC compound formulations. Samples of pipe, from a test installation of buried pipe, have been excavated after 14 years and a post evaluation has been conducted. These test results are used to draw some conclusions concerning the applicability of a material strain limit for constant strain design conditions.

KEYWORDS: buried pipes, PVC (polyvinyl chloride) Pipes, stress-relaxation, strain, filled PVC

INTRODUCTION

The use of PVC (polyvinyl chloride) pipe as sewer pipe in the United States began in the early to mid 1960's as early manufacturers of PVC resin looked for potentially high volume applications for their resin. Throughout the sixty's, PVC pipe of various types were provided for gravity sewer applications. Formal Standards [ASTM D3033 "Type PSP Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings," and D3034 "Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings"] were adopted in 1972 launching a virtual explosion of PVC sewer pipe use. Today, 90 percent of all sewer pipes in sizes 4 - 15 inches used in the United States, are made of PVC. (Note: ASTM D3033 was dropped as a formal standard in 1989.)

The first issue of ASTM D3034 and D3033 contained material requirements for a single PVC cell class of 12454B as described in ASTM D1784 "Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC)." The second issue published in 1973 contained a 13364B cell class as a second option.

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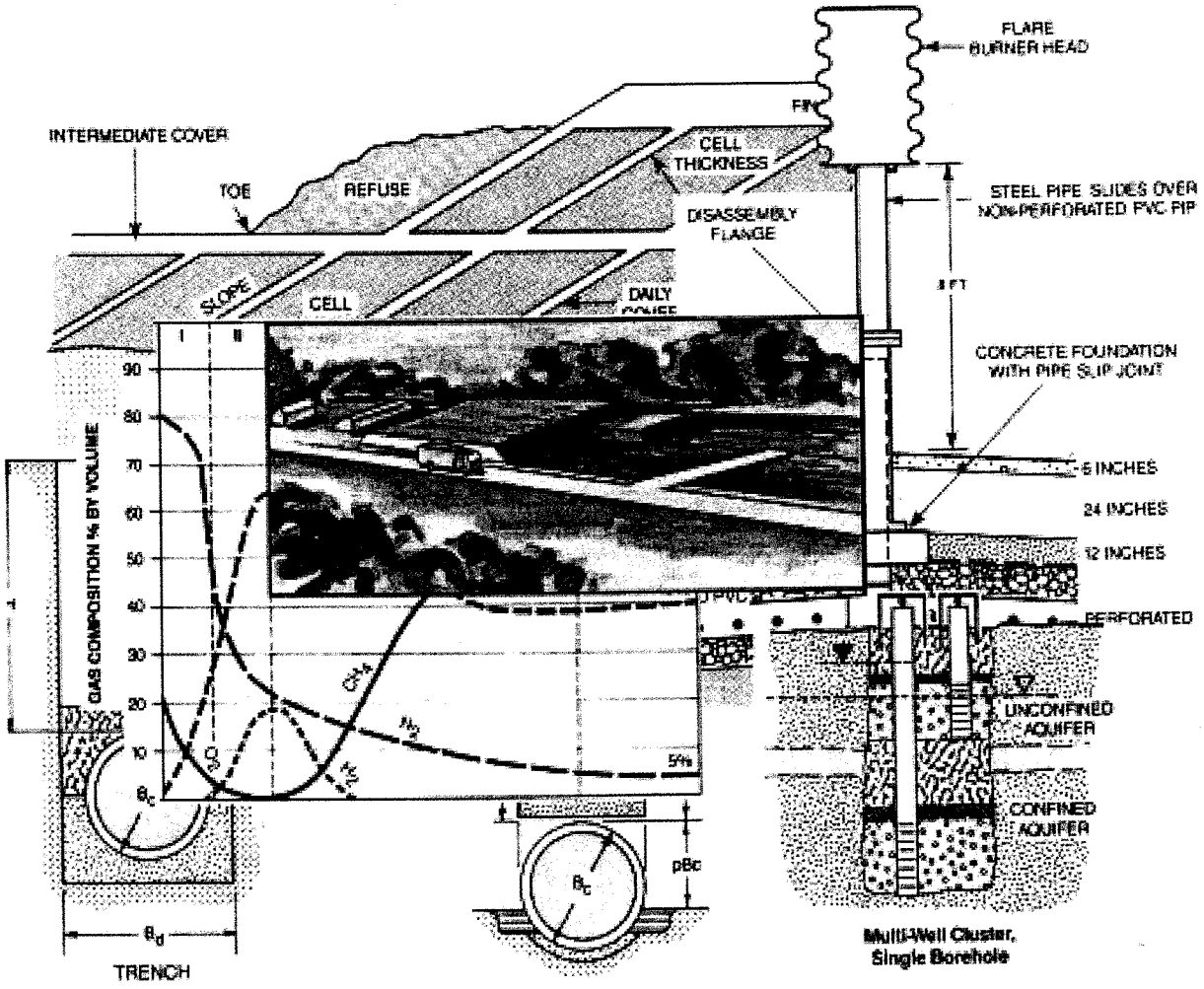
COPY OF REFERENCE SOURCE:

Washington State Department of Ecology, *Solid Waste Landfill Design Manual.* Publication No. 87-13, Washington, 1987.

Solid Waste Landfill Design Manual

Washington State Department of Ecology



SOLID WASTE LANDFILL DESIGN MANUAL

by

Parametrix, Inc.
13020 Northup Way, Suite 8
Bellevue, Washington

for

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Contract Officer

Marc E. Crooks, P.E.
Project Manager

Washington State Department of Ecology
Grants Section
Olympia, Washington

June 1987

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4C.1.1 Pipe Perforations

By nature of their intended use, leachate collection lines must be perforated. The size and spacing of the openings sho ld be determined based on hydraulic considerations. The effects of the perforations should be considered in the structural design of the leachate collection pipes.

4C.1.1.1 Size and Spacing

A leachate collection line, to function correctly, must be capable of accepting all the leachate flowing to it through the gravel drainage layer. After the pipe is sized to handle the flow, the size and spacing of the perforations should be selected. The rate of flow into the leachate collection pipes through the perforations is dependent on several factors, including the hydraulic conductivity of the gravel material around the pipe and the head loss due to convergence of flow to the perforations in the pipe.

W.T. Moody, as cited in U.S * Department of the Interior (1978) determined the theoretical relationship among the above factors and concluded that increasing the hydraulic conductivity of the gravel envelope around the pipe was a more effective method for increasing the rate~of flow into the pipe than increasing the size of the openings. Therefore, the selection of the size and spacing of the perforations should be based on: consideration of standard perforated pipe commonly available from manufacturer; bedding and backfill requirements for the particular installation; and effects on pipe strength. For a given rate of leachate inflow and a perforated pipe, the minimum required hydraulic conductivity of the gravel envelope around the pipe can be determined using a procedure similar to that presented in U.S. Department of the Interior (1978).

4C.1.1.2 Effects on Load Capacity

The various design procedures for rigid and flexible pipes and the various pipe performance limits are based on solid wall pipe. Pacey, et al., as cited in Dietzler (1984) has suggested that the effect of perforations could be compensated by arbitrarily increasing the earth load on the pipe. Data presented in Dietzler (1984) indicated the inclusion of typical perforations in'the lover quarters of 6-inch ABS and PVC pipe has little influence on pipe stiffness and deflection versus load performance. Others have stated there are indications that perforations will reduce the effective length of pipe available to carry loads and resist deflection suggest taking the effect of perforations into account by increasing the load in proportion to the reduction in the effective length. This later method appears to be an adequately conservative approach. If Lp equals the cumulative length of the perforations per unit length of the pipe, L, then thelactual load on the pipe should be increased as follows:

$$\text{Design Load} = \text{Actual Load} \times \frac{L}{L-Lp} \qquad (4C\text{-}1)$$

Methods to determine the actual load are discussed in the following sections.

COPY OF REFERENCE SOURCE:

Phillips 66, "Driscopipe System Design," Manufacturer's Literature, No. 1089-91 A17, Phillips 66, 1991.

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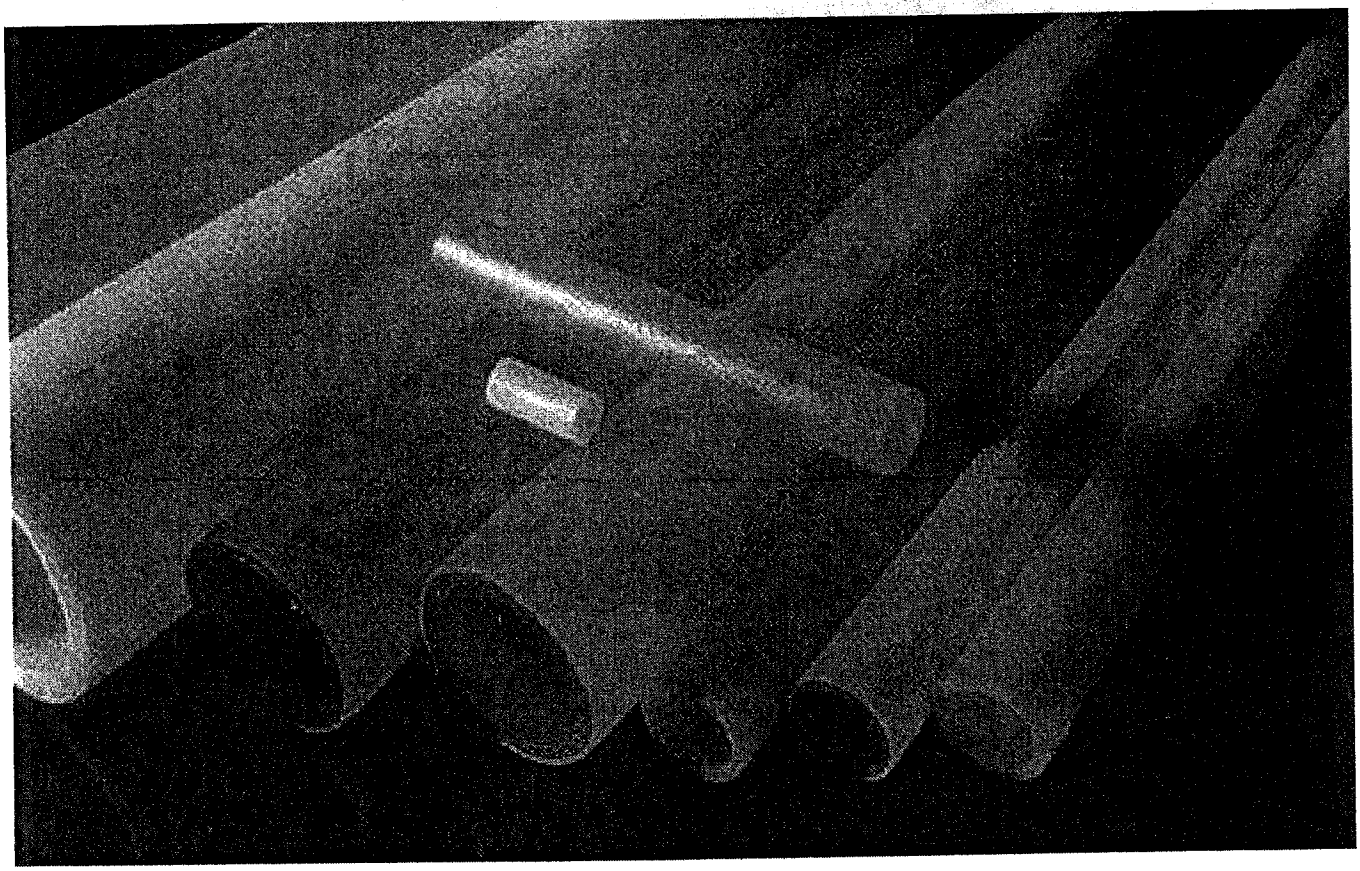

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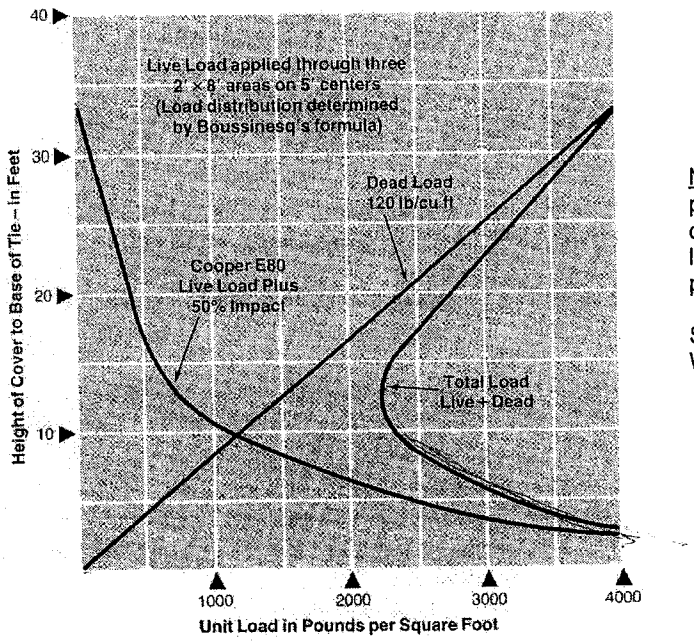
FIGURE 6: COOPER E-80 LIVE LOADING



Note: Cooper E-80 live load assumes 80,000 pounds applied to three 2' x 8' areas on 5' centers such as might be encountered through live loading from a locomotive with three 80,000 pound axle loads.

Source: American Iron and Steel Institute, Washington, DC

APPARENT EXTERNAL PRESSURE DUE TO INTERNAL VACUUM, P_I Vacuum generates a compressive hoop stress in the wall of a pipe and acts to collapse the pipeline. Under vacuum conditions, the value of P_I is positive. P_I is added to the other two external pressure components, P_S and P_L, to obtain the total external pressure, P_T, acting on the pipe. An internal vacuum generates pressure equal to the absolute value of the vacuum. The maximum apparent external pressure due to a vacuum inside the pipe is 14.7 psi (2,117 psf).

BURIAL DESIGN GUIDELINES

The design engineer must select the proper pipe DR and specify the backfill conditions to obtain the desired performance of the "pipe-soil" system.

DESIGN BY WALL CRUSHING Wall crushing occurs when external vertical pressure causes the compressive stress in the pipe wall to exceed the long-term compressive strength of the pipe material. To design for wall crushing, the following check should be made:

$$S_A = \frac{(SDR-1)}{2} P_T$$

Where:
S_A = Actual compressive stress, psi
SDR = Standard Dimension Ratio
P_T = Total external pressure on the top of the pipe, psi

Safety Factor = 1500 psi /S_A (where 1500 psi is the compressive yield strength of Driscopipe HDPE pipe)

DESIGN BY WALL BUCKLING Local wall buckling is a longitudinal wrinkling of the pipe wall. Buckling can occur over the long term in non-pressurized pipe if the total external soil pressure, P_T, exceeds the pipe-soil system's critical buckling pressure, P_{cb}. Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_T < P_{cb}$. All pipe diameters with the same DR in the same burial situation have the same critical collapse and critical buckling endurance.

1. Calculate or estimate the total soil pressure, P_T, at the top of the pipe.

2. Calculate the stress, S_a, in the pipe wall:

$$S_a = \frac{(SDR-1)P_T}{2}$$

3. Based upon the stress S_a and the estimated time duration of non-pressurization, find the value of the pipe's modulus of elasticity, E, in psi (approximate value for E is 35,000 psi).

4. Calculate the pipes hydrostatic, critical-collapse differential pressure, P_c

$$P_c = \frac{2E(t/D)^3(D_{MIN}/D_{MAX})^3}{(1-\mu^2)} \quad \text{or} \quad P_c = \frac{2.32(E)}{SDR^3}$$

Where:
$(D_{MIN}/D_{MAX}) = 0.95$
μ = Poission's Ratio = 0.45 for polyethylene pipe
E = stress and time dependent tensile modulus of elasticity, psi
E = 35,000 psi (approximate)
D = Outside Diameter, in.
t = thickness, in.

5 Calculate the soil modulus, E', by plotting the total external soil pressure, P_T, against a specified soil density to derive the soil strain as shown in the example problem below Figure 7.

6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{cb} = 0.8\sqrt{(E')(P_c)}$$

Where:
P_{cb} = Critical buckling soil pressure at the top of the pipe, psi
E' = Soil Modulus, psi
P_c = Hydrostatic critical-collapse differential pressure, psi

7. Calculate the Safety Factor: SF = P_{cb} / P_T.

8. The above procedures can be reversed to calculate the minimum pipe DR required for a given soil pressure and an estimated soil density.

In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

COPY OF REFERENCE SOURCE:

Koerner, R.M., "Designing with Geosynthetics," Fourth Edition, 1998.

Fourth Edition

Designing with Geosynthetics

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TABLE 7.9 U.S. BUREAU OF RECLAMATION VALUES OF MODULUS OF SOIL REACTION E' (kPa) FOR BURIED PIPELINES

Class ASTM D-2321	Soil type for pipe bedding material (Unified Classification System[A])	Dumped	Slight < 85% Std. Proctor[C] < 40% Rel. Den.[D]	Moderate 85–95% Std. Proctor 40–70% Rel. Den.	High > 95% Std. Proctor >70% Rel. Den.
I	Crushed rock: manufactured angular, granular material with little or no fines (6 to 38 mm)	7,000	21,000	21,000	21,000
II	Coarse-grained soils with little or no fines: GW, GP, SW, SP[B] containing less than 12 percent fines (max. particle size 38 mm)	NR	7,000	14,000	21,000
III	Coarse-grained soils with fines: GM, GC, SM, SC[B] containing more than 12 percent fines (max. particle size 38 mm)	NR	NR	7,000	14,000
IV(a)	Fine-grained soil (LL < 50): Soils with medium to no plasticity CL, ML, ML-CL, with more than 25 percent coarse-grained particles	NR	NR	7,000[E]	14,000[E]
IV(b)	Fine-grained soils (LL > 50): Soils with high plasticity CH, MH, CH-MH	NR	NR	NR	NR
	Fine-grained soils (LL < 50): Soils with medium to no plasticity CL, ML, ML-CL with less than 25 percent coarse-grained particles				

Organic soils OL, OM, and PT as well as soils containing frozen earth, debris, and large rocks are not recommended for initial backfill; NR = Not recommended for use per ASTM D-2321; LL = Liquid Limit.

[A]ASTM Designation D-2487

[B]Or any borderline soil beginning with some of these symbols (i.e., GM, GC, GC-SC).

[C]Percent Proctor based on laboratory maximum dry density from test standards using about 598,000 joules/m^3 (ASTM D-698)

[D]Relative Density per ASTM D-2049.

[E]Under some circumstances Class IV(a) soils are suitable as primary initial backfill. They are not suitable under heavy dead loads, dynamic loads, or beneath the water table. Compact with moisture content at optimum or slightly dry of optimum. Consult a Geotechnical Engineer before using.

Source: After Howard [14].

The ring stiffness constant (RSC) reflects the sensitivity of the pipe to installation stresses. It is defined in terms of the pipe's deflection resulting from the load applied between parallel plates as per ASTM D2412 (recall Section 7.1.2). As described in ASTM F-894, RSC is the value obtained by dividing the parallel plate load by the resulting deflection (in percent) at 3% deflection. Note that most plastic pipe manufacturers have an empirical formula, along with the necessary tables of their pipe products, for the evaluation of RSC values (e.g., see [15]). Eq. (7.18) also reflects strongly on the type, condition, and placement of backfill both on the sides of the pipe and above it (recall Table 7.9) for values of the modulus of soil reaction E'.

Due to the importance of the above formulation, several full-scale field and large-scale laboratory trials have been published, which give valuable information. Watkins and Reeve [3] have evaluated 375, 450, and 600 mm corrugated plastic pipe under standard H-20 truck loadings to determine the minimum cover necessary to prevent pipe damage and have also performed high pressure large-scale laboratory tests. Regarding the minimum cover tests, their results show the response given in Figure 7.9. Here it can be seen that for a limiting ring deflection of 5% (for this particular pipe) 300 to 375 mm of soil cover is necessary. For the large-scale laboratory tests, the setup and typical data is shown in Figure 7.4e.

Using the finite element computer program "Culvert Analysis and Design" (CANDE), Katona [16] has developed a series of design charts for allowable maximum fill heights. The program has the pipe and surrounding soil in an incremental plane strain formulation. The pipe is modeled with connected beam-column elements and the soil with continuous elements. The assumptions used are all reasonable, with the possible exceptions of a bonded pipe-to-soil interface and linear elastic polyethylene properties. Allowable fill heights for 108 cases are analyzed. The variations are as follows: pipe diameters ranging from 100 to 750 mm; three pipe corrugation areas in each pipe size; good and fair soil backfills; and short-term and long-term pipe properties ($E = 750$ MPa and $\sigma_y = 20$ MPa for a short-term life of 0.05 years, and $E = 150$ MPa

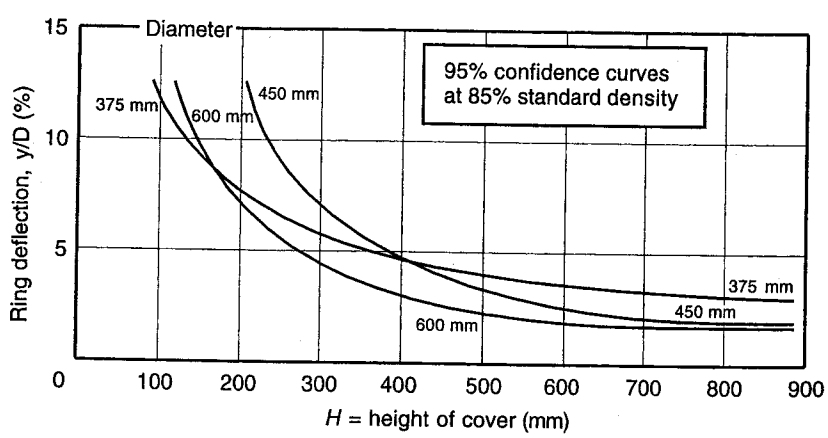

Figure 7.9 Minimum cover values for H-20 loading on HDPE pipe. (After Watkins and Reeve [3])

7.2.2 Deflection Issues

An engineering approach to the quantification of deflection of buried pipelines has been developed by a sequential group of research faculty and students at Iowa State University. Beginning with Marston in the 1920s evaluating rigid conduits (the term used for shallow buried pipes), followed by Spangler in 1950–1970 evaluating flexible conduits, and into the present by Watkins, the group and their colleagues have "written the book" for this type of research [12]. Key issues in the development are the use of arching theory for gravitational force dissipation, the importance of subgrade stability, backfill type, and compaction conditions, and finally the flexibility of the pipe structure itself. Moser [13] presents the following equation, summarizing the Iowa State group's effort for the deflection behavior of flexible (in our case plastic) pipe.

$$\Delta X = \frac{D_L K_b W_c}{(\mathrm{EI}/r^3) + (0.061E')} \cong y \tag{7.17}$$

where

ΔX = horizontal increase in diameter (m),
y = vertical deflection (m),
D_L = deflection lag factor, which varies from 1.0 to 1.5 (dimensionless),
K_b = bedding constant, which varies from 0.83 to 0.110 (dimensionless),
W_c = Marston's prism load per unit length of pipe (kN/m) (note that arching is not taken into account in this formula),
E = modulus of elasticity of the pipe material (kPa),
I = moment of inertia of the pipe wall per unit length (m^3),
EI = bedding stiffness of the pipe ring per unit length (kN-m),
r = mean radius of the pipe (m), and
E' = modulus of soil reaction (kPa).

The last term (E′) has been the subject of intense discussion and research. Howard [14] of the U.S. Bureau of Reclamation has recommended the values given in Table 7.9, which have relatively wide acceptance.

Eq. (7.17) can also be cast in terms of the laboratory plate loading test with the following result. The equation assumes a bedding constant $K_b = 0.2$ and uses the ring stiffness constant (RSC).

$$\frac{y}{D} = \frac{P(0.1L)}{[14.9(\mathrm{RSC})/D + 0.061E']} \tag{7.18}$$

where

y = vertical deflection (m),
D = inside pipe diameter (m),
P = load on pipe (kPa),
L = deflection lag factor (usually 1.0 to 1.5),
RSC = ring stiffness constant (kN/m), and
E' = modulus of soil reaction (kPa).

COPY OF REFERENCE SOURCE:

Chevron Chemical Company, "Plexco/Spirolite Engineering Manual, 2. System Design," Chevron Chemical Company, Bensenville, IL, August 1994.

7. Buried Pipe Design

The design of a subsurface pipe installation is based on principles of soil-structure interaction, that is the pipe and the surrounding soil act together to control the pipes performance. The role each plays in controlling performance depends on their stiffness relative to each other.

Pipes that are more stiff than the surrounding soil are typically called rigid. With rigid pipes, soil and surcharge loads are transmitted around the pipes ring from crown (top) to invert (bottom) by virtue of the pipes internal bending and compressive strength. Rigid pipes undergo little deflection. In some circumstances, polyethylene pipes may behave as a rigid pipe, such as the installation of low DR pipe in marsh soils. Here the pipe has greater stiffness than the surrounding soil, so the pipe properties become the major determinant of burial strength.

Pipes that are less stiff than the surrounding soil are called flexible. With weak soil support, relatively small earth loads may cause flexible pipe deflection. However, when properly buried, the surrounding soil greatly increases the pipes load-carrying capability as well as reduces the earth loads reaching the pipe.

The earth load and surcharge pressures applied to the soil backfill cause vertical and horizontal pipe deflection. The horizontal deflection, usually extension, results in the pipe wall pushing into the embedment soil. This action mobilizes passive resistance forces, which in turn limits horizontal deflection and balances the vertical load. More passive resistance is mobilized with stiffer surrounding soil, so less deflection occurs. Most polyethylene pipe should be considered flexible because the pipes contribution to resisting deflection is usually less than that of the surrounding soil.

Therefore, with polyethylene pipe it is important to check each application to ensure that the installed design (which would include both pipe and embedment soils) is adequate. The design procedures in this section may be applied to both rigid and flexible pipes

General Design Procedure

Once the pipe diameter has been determined, a pipe is selected by its wall construction. Lower DR PLEXCO pipes, and higher RSC SPIROLITE pipes have greater external load capacity. However, greater load capacity is also more costly, so the optimum design is a balance of the pipe strength and embedment quality that is capable of handling the imposed loads. The completed buried pipe design should specify the pipe size (OD or ID), wall construction (DR or RSC Class), required embedment materials, and placement (installation) requirements for that embedment.

The initial design step is to determine dead loads and surcharge loads. Following this, the pipe selection is checked for its ability to carry the imposed loads

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Spangler recommended addressing visco-elastic effects by using a deflection lag factor in the Iowa Formula. Recommended values range from 1.0 to 1.5.

Lytton and Brown published time factors based on a visco-elastic solution for long term deflection of pipe installed in saturated clay. The ratio of the 50-year deflection to the 30 day (or short term) deflection gave a lag factor of 1.5. Field measurements of HDPE pipe have confirmed values in the same range.

Example 7-12

Estimate the vertical deflection of a SPIROLITE 36" Class 100 installed under 18 feet of cover. The embedment material is a well-graded sandy gravel, compacted to a minimum 90 percent of Standard Proctor density.

Solution: Use the prism load, Equation (7-1) (page 39), Table 7-7, and Equation (7-35). Table 7-7 gives an E´ for a compacted sandy gravel or GW-SW soil as 2000 lb/in^2. To estimate maximum long-term deflection, this value will be reduced by 25%, or to 1500 lb/in^2. (The Duncan-Hartley value in Table 7-8 for this material with 18 ft of cover is 1700 psi.)

The prism load on the pipe is equal to:

$$P_E = (120)(18) = 2160\ lb/ft^2$$

Substituting these values into Equation (7-35) gives:

$$\frac{\Delta X}{D_I} = \frac{2160}{144}\left(\frac{(0.1)(1.5)}{\frac{(1.24)(100)}{36+2(0.58)} + (0.061)(1500)}\right)$$

$$\frac{\Delta X}{D_I} = 0.0237$$

$$\%DEFLECTION = \frac{\Delta X}{D_I}(100) = 2.37$$

Deflection Limits

Pipe deflection is a natural, essential, response to soil loading. Deflection mobilizes passive resistance in the surrounding soil, and promotes arching. Small deflections are desirable, but large deflections should be limited.

SPIROLITE pipe is manufactured to ASTM F 894 which states that profile pipe designed for 7.5% deflection will perform satisfactorily when installed in accordance with ASTM D 2321, and deflection is measured not less than 30 days following installation.

Manufacturing processes differ for SPIROLITE and PLEXCO pipe. Deflection limitations for PLEXCO pipe are controlled by long term material strain.

Ring Bending Strain

As pipe deflects, bending strains occur in the pipe wall. For an elliptically deformed pipe, the pipe wall ring bending strain, ε, can be related to deflection by:

$$\varepsilon = f_D \frac{\Delta Y}{D_M}\frac{2C}{D_M} \qquad (7\text{-}37)$$

Where terms are previously defined, and:

- ε = wall strain, %
- f_D = deformation shape factor
- D_M = mean diameter, in, (Equations (7-25) + (7-26))
- C = outer fiber to wall centroid, in (2C = Wall Thickness)

SPIROLITE Pipe:

$$C = h - z \qquad (7\text{-}38)$$

PLEXCO Pipe:

$$C = 0.5\,(1.06\,t). \qquad (7\text{-}39)$$

- h = pipe wall height, in
- z = pipe wall centroid, in
- t = pipe minimum wall thickness, in

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For elliptical deformation, $f_D = 4.28$. However, buried pipe rarely has a perfectly elliptical shape. Irregular deformation can occur from installation forces such as compaction variation alongside the pipe. To account for the non-elliptical shape many designers use $f_D = 6.0$.

Lytton and Chua report that for high performance polyethylene materials such as those used by PLEXCO, 4.2% ring bending strain is a conservative value for non-pressure pipe. Jansen reports that high performance polyethylene material at an 8% strain level has a life expectancy of a least 50 years.

When designing non-pressure heavy wall (< SDR 17) PLEXCO pipe, and high RSC (several hundred) SPIROLITE pipe, the ring bending strain at the predicted deflection should be calculated and compared to the allowable strain.

In pressure pipe, stress from deflection and internal pressure should not exceed the materials long term design stress rating. See Table 7-9, below.

Table 7-9 **Safe Pressure Pipe Deflection**

DR or SDR	*Safe Deflection as % of Diameter*
32.5	8.5
26	7.0
21	6.0
17	5.0
13.5	4.0
11	3.0
9	2.5

Example 7-13

Find the ring bending strain in the wall of the SPIROLITE 36" Class 100 pipe in Example 7-12.

Solution: Use Equation (7-37) and $f_D = 6.0$. Bulletin No. 910 gives: h = 2.02 in., and z = 0.58 in.

$$\varepsilon = 6\,(0.0237)\,\frac{2.02 - 0.58}{36 + 2\,(0.58)}$$

$$\varepsilon = 0.0055 = 0.55\%$$

The strain is well below the allowable strain of 4.2 percent for profile pipe.

Design Considerations For Shallow Cover Pipe

Pipe installed under shallow cover does not develop a complete soil structure interaction, so design methods must be modified for these installations. The designer should consider the following three cases: (1) flotation due to insufficient soil cover, (2) ring bending due to live load, and (3) upward buckling due to flooding or high groundwater levels.

The exact depth of cover required to develop the full soil structure interaction depends on the particular installation conditions.

Shallow Cover Surcharge Load

The preceding design methods assume that the pipe behaves primarily as a membrane structure, that is, the pipe is almost perfectly flexible with little ability to resist bending.

At depths of cover less than one pipe diameter, this membrane action may not be fully developed. So, an applied surcharge load or live load places a bending load on the pipe crown. For this reason, flexible

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ATTACHMENT 10

SECTION 02770

GEOMEMBRANE

PART 1 GENERAL

1.01 SCOPE

A. The section includes requirements for geomembrane products and installation.

1.02 RELATED SECTIONS AND PLANS

A. Section 02100 - Surveying

B. Section 02200 - Earthwork

C. Section 02215 - Trenching and Backfilling

D. Section 02740 - Geocomposites

E. Section 02780 - Geosynthetic Clay Liner (GCL)

F. Construction Quality Assurance (CQA) Plan

1.03 REFERENCES

A. Latest version of the American Society for Testing and Materials (ASTM) standards:

~~1. ASTM D 638. Standard Test Method for Tensile Properties of Plastics.~~

1. ASTM D 6693. Standard Test Method for Tensile Properties of Plastics.

2. ASTM D 746. Standard Test Method for Brittleness, Temperature of Plastics and Elastomers by Impact.
3. ASTM D 792. Standard Test Methods for Specific Gravity (Relative Density) and Density of Plastics by Displacement.
4. ASTM D 1004. Standard Test Method of Initial Tear Resistance of Plastic Film and Sheeting.
5. ASTM D 1204. Standard Plastics Test Method for Linear Dimensional Changes of Nonrigid Thermoplastic Sheeting or Film at Elevated Temperature.

6. ASTM D 1238. Standard Test Method for Flow Rates of Thermoplastics by Extrusion Plastometer.
7. ASTM D 1505. Standard Test Methods for Density of Plastics by Density-Gradient Technique.
8. ASTM D 1603. Standard Test Method for Carbon Black in Olefin Plastics.
9. ASTM D 1693. Standard Test Method for Environmental Stress Cracking of Ethylene Plastics
10. ASTM D 4437. Standard Test Methods for Determining the Integrity of Field Seams Used in Joining Flexible Polymeric Geomembranes.
11. ASTM D 5199. Standard Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes.
12. ASTM D 5397. Standard Test Method for Evaluation of Stress Crack Resistance of Polyolefin Geomembranes Using Notched Constant Tensile Load Test.
13. ASTM D 5596. Recommended Practice for Microscopical Examination of Pigment Dispersion in Plastic Compounds.
14. ASTM D 5994. Standard Test Method for Measuring the Core Thickness of Textured Geomembranes.
15. ASTM D 6392. Standard Test Methods for Determining the Integrity of Nonreinforced Geomembrane Seams Produced Using Thermo-Fusion Methods.
16. ASTM E96-00. Standard Test Methods for Water Vapor Transmission of Materials (Procedure BW).
17. ASTM D 3895 Test Method for Oxidative Induction Time of Polyolefins by Thermal Analysis
18. ASTM D 5885 Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High Pressure Differential Scanning Calorimetry
19. ASTM D 5321 Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method

B. Latest version of the Geosynthetic Research Institute (GRI) test methods:
1. GRI-GM13 Test Properties, Testing Frequency and Recommended Warranty for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes.

2. GRI-GM19 Seam Strength and Related Properties of Thermally Bonded Polyolefin Geomembranes

C. Latest version of Federal Test Method Standard (FTMS).
   1. FTMS 101/2065 Federal Test Method Standard for Puncture Resistance and Elongation Test (1/8 Inch Radius Probe Method).

1.04 WARRANTY

A. Furnish a 20-year written warranty against defects in materials. Warranty conditions concerning limits of liability will be evaluated by, and be acceptable to, the Engineer.

1.05 SUBMITTALS

A. Submit the following information to the Engineer for review not less than 45 calendar days prior to geomembrane use.
   1. Geomembrane manufacturer capabilities, including:
      a. daily production capacity available for this Contract; and
      b. manufacturing quality control procedures.
   2. A list of 10 completed facilities for which the manufacturer has supplied a minimum total of 10,000,000 square feet of polyethylene geomembrane. Provide the following information for each facility:
      a. name, location, purpose of facility, and date of installation;
      b. names of owner, project manager, design engineer, and installer; and
      c. thickness and surface area of geomembrane provided.
   3. Origin (resin supplier's name, resin production plant) and identification (brand name, number) of the polyethylene resin used.
   4. Certification of minimum average roll values (95 percent lower confidence limit) for physical, mechanical, and environmental properties and the corresponding test procedures for the geomembrane properties listed in Table 02770-1. Submit values that are specific to the resin used in manufacture.
   5. Certification that welding rod or granules are compatible with the specifications and the resin of the geomembrane furnished for this project
   6. Manufacturer warranty as specified in this section.

B. Submit to the Engineer for review not less than 30 calendar days prior to geomembrane use the following documentation on the resin used to manufacture the geomembranes:
   1. Copies of quality control certificates issued by the resin supplier including the production dates and origin of the resin used to manufacture the geomembrane for this Contract.
   2. Results of tests conducted by the manufacturer to verify the quality of the resin used to manufacture the geomembrane rolls assigned to the project.

3. Certification that no reclaimed polymer is added to the resin during the manufacturing of the geomembrane to be used for this project.

C. Submit to the Engineer for review the following documentation on geomembrane roll production at least 14 calendar days prior to transporting any geomembrane to the site.
   1. Manufacturing certificates for each shift's production of geomembrane, signed by the manufacturer quality control manager.
   2. Certificate shall include:
      a. roll numbers and identification;
      b. sampling procedures; and
      c. results of manufacturer quality control tests, including descriptions of the test methods used (the manufacturer quality control tests to be performed are given in Part 2 of this section).

D. Submit to the Engineer for review the following information from the installer at least 14 calendar days prior to mobilization of the installer to the site.
   1. Layout drawings showing the installation layout identifying geomembrane panel configurations, dimensions, details, locations of seams, as well as any variance or additional details which deviate from the Construction Drawings. The layout drawings shall be adequate for use as a construction plan and shall include dimensions, details, etc. The layout drawings, as modified and/or approved by the Engineer, shall become part of the contract.
   2. Installation schedule.
   3. Copy of installer's letter of approval or license by the manufacturer.
   4. Installation capabilities, including:
      a. information on equipment proposed for this project;
      b. average daily production anticipated for this project; and
      c. quality control procedures to include quality control organization.
   5. A list of 10 completed facilities for which the installer has installed a minimum of 5,000,000 square feet of polyethylene geomembrane. The following information shall be provided for each facility:
      a. the name and purpose of the facility, its location, and dates of installation;
      b. the names of the owner, project manager, and geomembrane manufacturer;
      c. name and qualifications of the supervisor of the installation crew;
      d. thickness and surface area of installed geomembrane;
      e. type of seaming and type of seaming apparatus used; and
      f. duration of installation.
   6. Resumes of the installer superintendent and quality control chief to be assigned to this project, including dates and duration of employment.

7. Resumes of all personnel who will perform seaming operations on this project, including dates and duration of employment.
8. Evidence that the installation crew has the following experience.
   a. The superintendent shall have supervised the installation of a minimum of 2,000,000 square feet of polyethylene geomembrane.
   b. At least one seamer shall have experience seaming a minimum of 500,000 square feet of polyethylene geomembrane using the same type of seaming apparatus to be used at this site. Seamers with such experience will be designated "master seamers" and shall provide direct supervision over less experienced seamers.
   c. All other seaming personnel shall have seamed at least 100,000 square feet of polyethylene geomembrane using the same type of seaming apparatus to be used at this site. Personnel who have seamed less than 100,000 square feet of seams shall be allowed to seam only under the direct supervision of the master seamer or Superintendent.

E. Submit to the Engineer for review at least 14 days prior to geomembrane placement, a certificate of calibration less than 12 months old for the field tensiometer. Tensiometer shall be calibrated within one year of date of test. Calibration shall be traceable to national or industry recognized standards where possible.

F. Submit subgrade acceptance certificates, signed by the Installer, for each area to be covered by the geomembrane prior to that area being covered by geomembrane.

G. Within 14 calendar days of completion of the geomembrane installation, submit to the Engineer the executed installation warranty as specified in this section.

1.06 CONSTRUCTION QUALITY ASSURANCE

A. The construction of the geomembrane component of the liner system system will be monitored by the CQA Consultant as required in the CQA Plan.

B. The CQA Consultant will perform material conformance testing of geomembrane materials and installation quality assurance testing of the geomembrane liner seams.

C. The Contractor shall be aware of the activities required of the CQA Consultant by the CQA Plan and shall account for these activities in the construction schedule.

D. The Contractor shall correct all deficiencies and nonconformances identified by the CQA Consultant at no additional cost to the Owner.

PART 2 PRODUCTS

2.01 RESIN

A. Provide geomembrane manufactured from new, first-quality polyethylene resin. Do not add reclaimed polymer to the resin. The use of polymer recycled during the manufacturing process is permitted if performed with appropriate cleanliness and if the recycled polymer during the manufacturing process does not exceed 2 percent by weight of the total polymer weight.

B. Use high density polyethylene (HDPE) resin for liner system geomembranes having the following properties:
   1. Specific Gravity: 0.932 minimum (ASTM D 792 Method A, or ASTM D 1505)
   2. Melt Index: 1.0 g/10 min., maximum (ASTM D 1238 Condition E)

2.02 GEOMEMBRANE PROPERTIES

A. Furnish 60-mil HDPE textured geomembranes having properties that comply with the required values shown in Table 02770-1.

B. In addition, furnish geomembrane that:
   1. contains a maximum of 1 percent by weight of additives, fillers, or extenders not including carbon black;
   2. does not have striations, pinholes, bubbles, blisters, nodules, undispersed raw materials, or any sign of contamination by foreign matter on the surface or in the interior;
   3. is free of holes, blisters, modules, undispersed raw materials, or any sign of contamination by foreign matter; and
   4. is manufactured in a single layer (thinner layers shall not be welded together to produce the final required thickness).

C. For CQA laboratory testing, the certified testing laboratory shall follow the specific procedures and conditions listed below:
   1. Place the materials to be tested in the shear box. For the geomembrane-geomembrane interface shear strength tests:
      a. Use a test specimen configuration of (from bottom to top): rigid substrate with textured gripping surface, 60-mil textured HDPE geomembrane, 60-mil textured HDPE geomembrane, and rigid substrate with textured gripping surface.

2. Perform the direct shear tests at normal stresses of 500 pounds per square foot. Report the peak and large-displacement (2-inch displacement) shearing resistance for each test.
3. Use fresh specimens for each normal stress.
4. Repeat any tests for which the shear displacements do not occur along the desired interface
5. For the geomembrane-geomembrane interface, the testing laboratory shall report peak and large displacement shear strengths for each of the respective tests in terms of secant friction angle. The results shall meet or exceed a shear strength envelope that is defined by a peak shear strength secant angle of 8° under a normal load of 500 psf.

2.03 MANUFACTURING QUALITY CONTROL

A. Resin:
1. Sample and test resin at a minimum frequency of one test per rail car to demonstrate that the resin complies with the requirements of this section. Perform tests on resin after the addition of additives to the virgin resin. Certify in writing that the resin meets the requirements of this section.
2. Do not use any noncomplying resin.

B. Rolls:
1. Continuously monitor for geomembrane defects during manufacture. Geomembranes shall be subjected to continuous spark testing by the Manufacturer at the factory.
2. Do not supply geomembrane that exhibits any defects.
3. Regularly monitor for geomembrane thickness during manufacture.
4. Do not supply geomembrane that fails to meet the specified thickness.
5. Sample and test the geomembrane, to demonstrate that its properties conform to the values specified in Table 02770-1. Perform the following quality control tests at a minimum of once every 50,000 square feet, with the exception of thickness, which shall be measured for each roll:

Test	Procedure
thickness	ASTM D 5199 (smooth) or ASTM D 5994 (textured)
yield strength	ASTM D ~~638~~ 6693
yield elongation	ASTM D ~~638~~ 6693
tensile strength	ASTM D ~~638~~ 6693
tensile elongation	ASTM D ~~638~~ 6693
tear resistance	ASTM D 1004
carbon black	ASTM D 1603
carbon black dispersion	ASTM D 5596
specific gravity	ASTM D 792, Method A or ASTM D 1505

6. If a geomembrane sample fails to meet the quality control requirements of this Section, sample and test rolls manufactured, in the same resin batch, or at the same time, as the failing roll. Continue to sample and test the rolls until the extent of the failing rolls are bracketed by passing rolls. Do not supply any failing rolls.
7. The following tests shall be run a minimum of once per every 250,000 square feet. Provide written certification that the geomembrane meets the material requirements as per the following test procedures. Provide written certification that these tests have been performed on geomembrane samples representative of rolls delivered to the site.

Test	Procedure
SP-NCTL	ASTM D 5397

C. Permit the CQA Consultant and/or Engineer to visit the manufacturing plant for project specific visits. If possible, such visits will be prior to, or during, the manufacturing of the geomembrane rolls for this project.

2.04 LABELING

A. Label the geomembrane rolls with the following information.
   1. thickness of the material;
   2. length and width of the roll;
   3. name of Manufacturer;
   4. product identification;
   5. lot number; and
   6. roll number.

B. Geomembrane rolls not labeled in accordance with this Section or on which labels are illegible upon arrival at the site will be rejected and replaced at no additional expense to the Owner.

2.05 TRANSPORTATION, HANDLING AND STORAGE

A. Deliver geomembranes to the site at least 14 calendar days prior to the planned deployment date to allow the CQA Consultant adequate time to perform conformance testing on the geomembrane samples as described in the CQA Plan.

B. Provide proper handling and storage of the geomembrane at the site. Protect the geomembrane from excessive heat or cold, dirt, puncture, cutting, or other damaging or deleterious conditions. Provide any additional storage procedures required by the Manufacturer.

C. Store geomembrane rolls on pallets or other elevated structures. Do not store geomembrane rolls directly on the ground surface. Do not store more than 3 rolls high.

PART 3 EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work described in this section, the Contractor shall become thoroughly familiar with all portions of the work falling within this section.

B. Inspection:
   1. Prior to implementing any of the work in this section, the Contractor shall carefully inspect the installed work of all other sections and verify that all work is complete to the point where the installation of this section may properly commence without adverse impact.
   2. If the Contractor has any concerns regarding the installed work of other sections, the Contractor shall immediately notify the Engineer in writing. Failure to inform the Engineer in writing or continuance of installation of the geomembrane will be construed as the Contractor's acceptance of the related work of all other sections.

3.02 SUBGRADE SURFACE PREPARATION

A. The Contractor shall provide certification in writing that the surface on which the geomembrane will be installed is acceptable. Where a GCL is installed on the subgrade

prior to the geomembrane, the Contractor shall inspect the subgrade prior to GCL installation. This certification of acceptance shall be given to the CQA Consultant prior to commencement of geomembrane installation in the area under consideration.

B. Special care shall be taken to maintain the prepared surface.

C. No geomembrane shall be placed onto areas of standing water or hydrated GCL

D. Any damage to the GCL or prepared subgrade caused by installation activities shall be repaired at the Contractor's expense.

3.03 GEOMEMBRANE DEPLOYMENT

A. General:
1. Textured geomembrane is to be used for all liner construction indicated on the Construction drawings.
2. The Contractor shall produce layout drawings prior to geomembrane deployment. These drawings shall indicate the geomembrane configuration, dimensions, details, locations of seams, etc. The layout drawings must be approved by the Engineer prior to the installation of any geomembranes. The layout drawings, as modified and/or approved by the Engineer, shall become part of these specifications.
3. Do not deploy geomembrane until the layout drawings are approved by the Engineer.
4. Do not deploy a geomembrane panel in an area until the CQA Consultant has been provided with a certificate of subgrade acceptance for that area.
5. Do not deploy geomembranes until CQA Consultant completes conformance evaluation of the geomembrane and performance evaluation of previous work, including evaluation of Contractor's survey results for previous work.
6. Deploy each geomembrane panel in accordance with the approved layout drawings.

B. Field Panel Identification:
1. A geomembrane field panel is a roll or a portion of roll cut in the field.
2. Give each field panel an identification code (number or letter-number). This identification code shall be agreed upon by the CQA Consultant and the Installer.

C. Field Panel Placement:
1. Place each geomembrane panel one at a time and seam each panel immediately after its placement.
2. Use temporary rub sheets as required to prevent displacement or damage to underlying geosynthetics. High spots in geomembrane-backed geosynthetic clay

liners shall be covered by a temporary rub sheets during placement of geomembrane.

3. Do not place geomembrane panels when the ambient temperature is below 40° Fahrenheit (F), unless authorized in writing by the Engineer. For cold weather (<40°F) deployment, use the additional procedures authorized in writing by the Engineer.
4. Do not place geomembranes during any precipitation, in the presence of heavy fog or dew, in an area of ponded water, or in the presence of high wind.
5. Ensure that:
   a. No vehicular traffic drives directly on the geomembrane.
   b. Equipment used does not damage the geomembrane by handling, trafficking, or leakage of hydrocarbons (i.e., fuels).
   c. Personnel working on the geomembrane do not smoke, bring glass onto the geomembrane, or engage in other activities that could damage the geomembrane.
   d. The method used to unroll the panels does not scratch or crimp the geomembrane and does not damage lower geosynthetics or the supporting soil.
   e. The method used to place the panels minimizes wrinkles (especially differential wrinkles between adjacent panels). The method used to place the panels results in intimate contact with geosynthetic clay liner. Adjust or repair any area of geomembrane wrinkles where the wrinkle height, measured perpendicular to the slope during the hottest portion of the day, is more than 4 inches.
   f. The method used to place the panels does not cause the panels to lift up or trampoline during the coolest portion of the day.
   g. The geomembrane is anchored or weighted with sandbags, or the equivalent, to prevent damage or uplift from wind. Install sufficient anchoring or weighting to prevent uplift and maintain such system until overlying material is placed.
6. Replace any field panel or portion thereof that becomes damaged (torn, twisted, or crimped). Remove from the work area damaged panels or portions of damaged panels.
7. Geomembrane installation shall not occur during non-daylight hours.

D. Do not install geomembrane between one hour before sunset and one hour after sunrise unless approved by the Engineer.

3.04 FIELD SEAMING

A. Personnel shall be experienced as specified in this section. Do not perform seaming unless a "master seamer" and the CQA Consultant are on-site.

B. Orient seams parallel to the line of maximum slope (i.e., oriented down, not across, the slope). Minimize the number of seams in corners and at odd-shaped geometric locations. No horizontal seam shall be less than 10 feet from the toe of the slope, except where approved by the Engineer. Do not locate seams at an area of potential stress concentration.

C. Weather Conditions for Seaming:
   1. Do not seam geomembrane at ambient temperatures below 40°F or above 104°F, unless authorized in writing by the Engineer. For cold (<40°F) or hot (>104°F) weather seaming, use the additional procedures authorized in writing by the Engineer.
   2. Measure ambient temperatures between 0 to 6 inches above the geomembrane surface.
   3. In all cases the geomembrane seam areas shall be dry and protected from wind.

D. Overlapping and Temporary Bonding:
   1. Geomembrane panels shall have a minimum finished overlap of 4 inches ~~Sufficiently overlap geomembrane panels~~ for welding and to allow peel tests to be performed on the seam. Any seams that cannot be destructively tested because of insufficient overlap are failing seams.
   2. Control the temperature of the air at the nozzle of heat bonding apparatus such that the geomembrane is not damaged.

E. Seam Preparation:
   1. Prior to seaming, clean the seam area and ensure that area to be bonded is free of moisture, dust, dirt, debris of any kind, and foreign material.
   2. If seam overlap grinding is required, complete the process according to the Manufacturer's instructions or within 60 minutes of the seaming operation. Do not grind to a depth that exceeds ten percent of the geomembrane thickness. Grinding marks shall not appear beyond 0.25 inch of the extrudate after it is placed.
   3. Align seams with the fewest possible number of wrinkles and no "fishmouths".

F. General Seaming Requirements:
   1. Extend seams to the outside edge of panels to be placed in the anchor trench.

2. If required, place a firm substrate such as a flat board or similar hard surface directly under the seam overlap to achieve proper support.
3. Cut fishmouths or wrinkles at the seam overlaps along the ridge of the wrinkle to achieve a flat overlap. Seam the cut fishmouths or wrinkles and patch any portion where the overlap is less than 6 inches with an oval or round patch of geomembrane that extends a minimum of 6 inches beyond the cut in all directions.
4. Place the electric generator used for power supply to the welding machines outside the area to be lined or mount it on soft tires such that no damage occurs to the geomembrane. Properly ground the electric generator. Place a smooth insulating plate or fabric beneath the hot welding apparatus after use.

G. Seaming Process:
1. Approved processes for field seaming are extrusion welding and fusion welding. The primary method of welding shall be fusion. Seaming equipment shall not damage the geomembrane. Use only geomembrane Manufacturer-approved equipment.
2. Extrusion Equipment and Procedures:
   a. Maintain at least one spare operable seaming apparatus on site.
   b. Equip extrusion welding apparatus with gauges giving the temperature in the apparatus and at the nozzle.
   c. Prior to beginning a seam, purge the extruder until all heat-degraded extrudate has been removed from the barrel. Whenever the extruder is stopped, purge the barrel of all heat-degraded extrudate.
3. Fusion Equipment and Procedures:
   a. Maintain at least one spare operable seaming apparatus on site.
   b. Fusion-welding apparatus shall be automated self-propelled devices equipped with gauges giving the applicable temperatures and pressures.
   c. Fusion-welding apparatus shall produce a double-track seam.
   d. Abrade the edges of cross seams to a smooth incline (top and bottom) prior to extrusion welding.

H. Trial Seams:
1. Make trial seams on excess pieces of geomembrane to verify that seaming conditions are adequate. Conduct trial seams on the same material to be installed and under similar field conditions as production seams. Conduct trial seaming at the beginning of each seaming period, and at least once each five hours, for each seaming apparatus used that day prior to seaming. Also, each seamer shall make at least one trial seam each day, for each day that seaming is performed by that seamer. Conduct trial seaming under the same conditions as the actual seaming. Prepare trial seams that are at least 15 feet long by 1 foot wide (after seaming) with the seam

centered lengthwise for fusion equipment and at least 3 feet long by 1 foot wide for extrusion equipment. Prepare seam overlap as indicated in the "Overlapping and Temporary Bonding" Article of this Part.

2. Cut four specimens, each 1.0 inch wide, from the trial seam sample. Test two specimens in shear and two in peel, using a field tensiometer. The test specimens shall not fail in the seam. If a specimen fails, repeat the entire operation. If the additional specimen fails, do not accept the seaming apparatus or seamer until the deficiencies are corrected and two consecutive successful trial seams are achieved. A seamer may start production seaming prior to testing of the trial seams. In the event the trial seam fails, all production seams by the seamer are failed seams.

I. Nondestructive Seam Continuity Testing:
   1. Nondestructively test field seams for continuity over their full length. Perform continuity testing as the seaming work progresses, not at the completion of field seaming. Complete any required repairs in accordance with the "Defects and Repairs" Article of this Part. Apply the following procedures:
      a. use vacuum testing for extrusion welds; and
      b. use air pressure testing for double-track fusion seams.
   2. Vacuum Testing:
      a. Use the following equipment:
         i. A vacuum box assembly consisting of a stiff housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge.
         ii. A system for applying 5 pound per square inch (psi) gauge suction to the box.
         iii. A bucket of soapy solution and applicator.

      b. Follow these procedures:
         i. Energize the vacuum pump and reduce the tank pressure to 5 ±1 psi gauge.
         ii. Wet an area of the geomembrane seam larger than the vacuum box with the soapy solution.
         iii. Place the box over the wetted area.
         iv. Close the bleed valve and open the vacuum valve.
         v. Ensure that a leak tight seal is created.
         vi. Examine the geomembrane through the viewing window for the presence of soap bubbles for not less than 20 seconds.
         vii. If no bubbles appear after 20 seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3 inch overlap, and repeat the process.

viii. Mark all areas where soap bubbles appear with a marker that will not damage the geomembrane and repair in accordance with the "Defects and Repairs" Article of this Part.

3. Air Pressure Testing:
   a. Use the following equipment:
      i. an air pump (manual or motor driven) or air reservoir, equipped with a pressure gauge, capable of generating and sustaining a pressure between 25 and 30 pounds per square inch;
      ii. a rubber hose with fittings and connections; and
      iii. a hollow needle, or other approved pressure feed device..
   b. Follow these procedures:
      i. Seal both ends of the seam to be tested.
      ii. Insert needle, or other approved pressure feed device, into the tunnel created by the fusion weld.
      iii. Insert a protective cushion between the air pump and the geomembrane.
      iv. Energize the air pump to a pressure between 25 and 30 pounds per square inches, close valve, and sustain the pressure for not less than 5 minutes.
      v. If loss of pressure exceeds 3 pounds per square inches, or does not stabilize, locate faulty area and repair in accordance with the "Defects and Repairs" Article of this Part.
      vi. Cut opposite end of air channel from pressure gauge and observe release of pressure to ensure air channel is not blocked.
      vii. Remove needle, or other approved pressure feed device, and seal both ends in accordance with the "Defects and Repairs" Article of this Part.

J. Destructive Testing:
   1. Perform destructive seam tests to evaluate seam strength and integrity. Perform destructive testing as the seaming work progresses, not at the completion of field seaming.
   2. Sampling and Testing:
      a. Collect destructive test samples at a minimum average frequency of one test location per 200 feet of seam length and at additional locations of suspected nonperformance. The CQA Consultant will select test locations, including locations with evidence of excess geomembrane crystallinity, contamination, offset seams, or any other evidence of inadequate seaming.
      b. Cut samples at the locations designated by the CQA Consultant at the time the locations are designated. Number each sample and identify the sample number and location on the panel layout drawing. Immediately repair all holes in the geomembrane resulting from the destructive seam sampling in accordance with

the repair procedures described in the "Defects and Repairs" Article of this Part. Test the continuity of the new seams in the repaired areas according to "Nondestructive Seam Continuity Testing" Article of this Part.

c. Cut a minimum of two strips 1 inch wide and 12 inch long with the seam centered parallel to the width from either side of the sample location. The distance between these two specimens shall be 42 inches. Test the two 1-inch wide strips in ~~the~~ a gauged field tensiometer in the peel mode. The CQA Consultant may request an additional test in the shear mode. If these samples pass the field test, prepare a laboratory sample between the two field test strips. The laboratory sample shall be at least 1 foot wide by 3.5 feet long with the seam centered lengthwise. Cut the laboratory sample into three parts and distribute as follows:

i. one portion 1 foot long to the Installer;

ii. one portion 1.5 feet long to the CQA Consultant for laboratory testing; and

iii. one portion 1 foot long to the Engineer for archival storage.

3. In the event of failing field or laboratory test results, the Contractor may reconstruct the entire seam between two passing destructive tests; otherwise, the CQA Consultant will identify the extent of the nonconforming area following the procedures given in the CQA Plan. Obtain additional samples for testing as requested by the CQA Consultant.

K. Defects and Repairs:

1. Inspect the geomembrane before and after seaming for evidence of defects, holes, blisters, undispersed raw materials, and any sign of contamination by foreign matter. The surface of the geomembrane shall be clean at the time of inspection. Sweep or wash the geomembrane surface if surface contamination inhibits inspection.
2. Test each suspect location, both in seam and non-seam areas, using the methods described in the "Nondestructive Seam Continuity Testing" Article of this Part. Repair each location that fails nondestructive testing.
3. Cut and reseam wrinkles not conforming with Part 2 of this Section. Test the seams thus produced like any other seam.
4. Repair Procedures:

a. Repair any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test. Use the most appropriate of the available procedures:

i. patching, used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter;

ii. abrading and reseaming, used to repair small sections of extruded seams;

iii. spot seaming, used to repair minor, localized flaws;

iv. capping, used to repair long lengths of failed seams;
v. topping, used to repair areas of inadequate seams, which have an exposed edge less than 4 inches in length; and
vi. removing bad seam and replacing with a strip of new material seamed into place (used with long lengths of fusion seams).

b. When making repairs, satisfy the following:
i. abrade surfaces of the geomembrane that are to be repaired no more than 60 minutes prior to the repair;
ii. clean and dry all geomembrane surfaces immediately prior to repair;
iii. only use approved seaming equipment;
iv. extend patches or caps at least 6 inches beyond the edge of the defect, and round corners of patches to a radius of at least 3 inches; and
v. cut the geomembrane below large caps to avoid potential for water or gas collection between the two sheets.

5. Repair Verification:
a. Test each repair using the methods described in the "Nondestructive Seam Continuity Testing" Article of this Part. Repairs that pass the nondestructive test are adequate unless the CQA Consultant elects to also perform destructive tests. Re-repair and retest failed tests.

3.05 ANCHORAGE SYSTEM

A. The anchor trench shall be excavated prior to geomembrane placement to the lines, grades, and configuration indicated on the Construction Drawings.

B. Slightly rounded corners shall be provided in the trench where the geomembrane adjoins the trench to avoid sharp bends in the geomembrane.

C. Temporarily anchor each geomembrane panel in the anchor trench at the crest of the slope as soon as the panel is deployed or positioned.

D. Do not entrap loose soil, sand bags, or other materials between or beneath the geosynthetic layers.

E. Do not backfill the anchor trench until all geosynthetic layers are installed in the anchor trench. Backfill in accordance with the Construction Drawings and Section 02215.

F. Do not damage any geosynthetic layer when backfilling the anchor trench.

3.06 MATERIALS IN CONTACT WITH THE GEOMEMBRANE

A. Take all necessary precautions to prevent damage to the geomembrane during the installation of other components of the liner and final cover system.

B. Do not drive equipment directly on the geomembrane. Only use equipment above the geomembrane that meets the following ground pressure requirements.

Maximum Allowable Equipment Ground Pressure (pounds per square inches)	Minimum Thickness of Overlying Material (inches)
<5	12
<10	18
<20	24
>20	36

3.07 SURVEY CONTROL

A. Survey the installed geomembrane liner and final cover in accordance with Section 02100.

3.08 GEOMEMBRANE ACCEPTANCE

A. The Contractor shall retain all ownership and responsibility for the geomembrane until accepted by the Owner.

B. The geomembrane shall be accepted by the Owner when:
   1. the installation is finished;
   2. all documentation of installation is completed including the CQA Consultant's final report; and
   3. verification of the adequacy of all field seams and repairs, including associated testing, is complete.

3.09 PROTECTION OF WORK

A. The Contractor shall use all means necessary to protect all prior work and all materials and completed work of other sections.

B. In the event of damage, the Contractor shall make all repairs and replacements necessary at no additional cost to Owner.

TABLE 02770-1
REQUIRED HDPE GEOMEMBRANE PROPERTIES

Properties	Qualifiers	Units[1]	Specified Values Textured	Test Method
Physical Properties				
Thickness	Nominal Minimum	mils	54	ASTM D 5994 (T)
Specific Gravity	Minimum	N/A	0.94	ASTM D 792 Method A or ASTM D 1505
Carbon Black Content	Range	%	2-3	ASTM D 1603
Carbon Black Dispersion	N/A	none	8 of 10 in Category 1 or 2 and all in Category 1, 2, or 3	ASTM D 5596
Oxidative Induction Time (OIT)[3]				
(a) Standard OIT			100	ASTM D 3895
OR	Minimum	minutes		
(b) High Pressure (OIT)			400	ASTM D 5885
Mechanical Properties				
Tensile Properties				
1. Force Per Unit Width at Yield	Minimum	lb/in	~~130~~ 126	ASTM D 6693
2. Tensile Strength (force per unit width at break)	Minimum	lb/in	~~72~~ 90	ASTM D 6693
3. Elongation at Yield	Minimum	%	12	ASTM D 6693
4. Elongation at Break	Minimum	%	100	ASTM D 6693
Tear Resistance	Minimum	lb	40	ASTM D 1004 Die C Puncture
Puncture Resistance	Minimum	lb	80	ASTM D 4833

TABLE 02770-1 (continued)

Properties	Qualifiers	Units[1]	Specified Values Textured	Test Method
Environmental Properties				
SP-NCTL	Minimum	hrs	200[2]	ASTM D 5397

Notes: 1. % = percent
g = grams
min = minutes
lb/in = pounds per inch
lb = pound
°C = degrees Celsius
hrs = hours

2. Time-to-failure at a tensile stress of 30 percent of the tensile yield strength. For textured geomembrane, test is conducted on smooth geomembrane from the same resin lot (batch) as the textured geomembrane furnished.
3. The manufacturer has the option to select either one of the OIT methods listed to evaluate the antioxidant content in the geomembrane.

TABLE 02770-2

REQUIRED HDPE GEOMEMBRANE SEAM PROPERTIES

Properties	Qualifiers	Units[3]	Specified Values		Test Method
			Smooth	Textured	
Shear Strength[1]					
fusion	Minimum	lb/in	120	120	ASTM D 6392
extrusion	Minimum	lb/in	~~108~~ 120	~~108~~ 120	ASTM D 6392
Peel Adhesion					
			FTB[2]	FTB[2]	
fusion	Minimum	lb/in	~~78~~ 91	~~78~~ 91	ASTM D 6392
extrusion	Minimum	lb/in	~~70~~ 78	~~70~~ 78	ASTM D 6392

Notes:
1. Also called "Bonded Seam Strength". Value is at material yield point and failure shall occur in material outside of seam area.
2. FTB = Film Tear Bond. (Maximum 10 percent seam separation)
3. lb/in = pounds per inch

[END OF SECTION]

ATTACHMENT 11

COPY OF REFERENCE SOURCE:

Martin, J.P., Koerner, R.M., and Whitty, J.E., "Experimental Friction Evaluation of Slippage Between Geomembranes and Geotextiles," Proceedings of the International Conference on Geomembranes, Denver, Colorado, pp. 191-196, 1984.

MARTIN, J. P., KOERNER, R. M., and WHITTY, J. E.
Drexel University, Philadelphia, Pennsylvania, USA

Experimental Friction Evaluation of Slippage Between Geomembranes, Geotextiles and Soils

A common failure mechanism of geomembrane lined side slopes of impoundments and reservoirs is by slipping of components within the liner system or of the cover soil. While safe design is indeed possible, the friction values between individual components are required and are essentially not available to date. This study focuses on presenting a test methodology and data base for friction values between three soil tyes, four geomembranes and four geotextiles. Seen is that the values vary widely in accordance with the materials being used. Mobilized friction values from 60% to 100% of the intrinsic values of the material by itself were determined. Details of the tests and individual values are reported.

INTRODUCTION

The usual design goal of excavated or built-up impoundments is to build the side slopes as steeply as possible. This is particularly true at sites of high water table or in containing large volumes with respect to the available land area. To eliminate, or minimize, the loss of the contained liquids or generated leachates it is usually necessary to line both the bottom and sides of such impoundments. For the purpose of this study, the primary liner will be assumed to be a flexible membrane liner (FML), i.e., a geomembrane, made from polymeric materials into relatively thin sheets, 20 mils to 100 mils thick, and adequately seamed together wherever joints are necessary. In some circumstances it is necessary to sandwich this geomembrane between one or two geotextiles, which are porous woven or nonwoven fabrics that serve the following functions:

The geotextile underliner:

- prevents underlying stones and sharp objects from puncturing the geomembrane
- provides a clean working surface for placement of the geomembrane and the making of field seams
- provides some support (reinforcement) over weak areas in the subgrade
- acts as a lateral transmitter of water and gas which may come up from the subsurface soil beneath the geomembrane -- in this case, one must select a bulky, needled nonwoven geotextile which possesses adequate transmissivity.(1,2)

The geotextile overliner:

- protects the geomembrane from puncture of stones in the cover soil or in the landfilled material itself
- provides some load spreading capability for heavy objects in the landfill, i.e. reinforcement
- protects the geomembrane from ozone and ultraviolet attack for cases where the liner system is not soil covered

Usually, but certainly not always, the sandwiched geomembrane liner is covered with a layer of soil. This cover soil should be select material with good gradation and strength characteristics so that it can be easily placed and compacted in as thin a layer as possible. Usually its thickness is from 30.48 to 91.44 centimeters. In many cases it serves a dual role as protection to the liner system and as a leachate collection system containment media, i.e., pipe underdrains are placed within it.

With the above thoughts in mind, the general cross section of the side slopes of lined impoundments containing liquids and/or solids is presented in Figure 1. Note that the following alternates for the liner system can be used:

- geomembrane alone (GM)
- geomembrane plus cover soil (GM/CS)
- geotextile underliner plus geomembrane (GTU/GM)
- geotextile underliner plus geomembrane plus cover soil (GTU/GM/CS)
- geotextile underliner plus geomembrane plus geotextile overliner (GTU/GM/GTO)
- geotextile underliner plus geomembrane plus geotextile overliner plus cover soil (GTU/GM/GTO/CS)

Upon the decision as to the choice of above liner system and a knowledge of the depth of the impoundment, the critical variable becomes the slope angle and the general stability of the lined side slopes.

The analysis of slope stability for both homogeneous and heterogeneous soil masses is well developed in geotechnical engineering practice. However, the analysis of stability when flexible synthetic sheets (geomembranes and geotextiles) under tension are placed on the slope face is still in its infancy. This situation falls into the general classification of soil-structure interaction problems. The three major elements necessary to extend organized slope stability analysis into membrane-lined impoundments are:

(a) Data on limiting shear strength along interfaces between soils, geomembranes and geotextiles.
(b) Effect of tension in the liner system (provided for by the anchor trench) on the overall slope stability.

(c) Effect of slippage between soils, geomembranes and geotextiles and its relationship to the general stress-strain behavior of the materials.

This paper is a report of experimental work that concentrates primarily on item (a). It extends published data on friction between geotextiles and soils, and presents new data on frictional behavior between soils and geomembranes and also between geotextiles and geomembranes. Item (b) is more analytical than experimental, and is only considered briefly herein. However, a review of analytical methods is included since it provides a basis for further work in this area with the experimental data obtained and presented. A brief discussion of item (c) is included, but it is actually a summary of a more extended report soon to be available.(3,4)

ANALYSIS OF STABILITY OF LINED SIDE SLOPES

There are two major areas of concern with respect to stability of the side slopes of a lined enclosure: slope stability of the soil subgrade, natural formations and compacted embankment under the liner and slippage within the liner system consisting of geomembrane, geotextiles and cover soil.

Analysis of general slope stability involves determination of the factor of safety against shear failure along an undefined critical surface, usually assumed to be circular. Design centers around the selection of the appropriate geometry, materials and other measures to obtain the desired factor of safety. The driving force for most slope failures is the applied stress along a continuous surface that results from body and surcharge forces. The resistance is provided by the cohesive and frictional strength of the soil and other materials along a slip surface. Schematically, this type of a failure is shown in Figure 2(a). Proper design to prevent this situation from occurring is well within the state-of-the-art of geotechnical engineering. It is, indeed, an important consideration but it is beyond the scope of this paper.

Slippage between the various components of the liner system, however, is of very real concern and is the general thrust of this study. It is shown schematically in Figure 2(b) for both the geomembrane liner system and the cover soil over the liner. The design procedure in this case of a liner failure along a known surface is straighforward once the values of friction are known between the various interfaces involved. Assuming these values are known a force polygon can be drawn consisting of the following items which are shown and illustrated in Figure 3.

- The weight of the liner system and cover soil (if present); which act vertically downward (W_A and W_{NB})
- The tensile strength of the liner system (geomembrane plus geotextiles, if present); which acts along the slope and is eventually mobilized in the anchor trench (T)
- The possible resistance to failure of a small wedge of cover soil at the toe of the slope; which also acts along the slope (E_A and E_{NB})
- The unknown frictional forces (F_A and F_{NB}) which act at different friction angles (δ_A and δ_{NB}), where the friction angle δ_A is the minimum value between any interface in the liner system and must be determined experimentally (this item is the specific focus of this paper) and the friction angle δ_{NB} which is completely within the cover soil and is generally equal to the friction angle of the soil.

This type of problem is best solved by assuming a factor of safety and applying it to δ_A and δ_{NB}. A force polygon for the neutral block is drawn to obtain a trial value for E_{NB}. This value is then made equal to E_A and is used in construction of a force polygon for the active zone. If closure of the active zone polygon is obtained, the initially assumed factor of safety is correct. If not, successive trials using different values will be required until a graph can be drawn to accurately assess the actual factor of safety. Usually three or four trials are necessary.

Critical in this design process, and not available in the required form as far as the authors are aware, is the value for interface friction between components of the liner system, i.e., δ_A values. The design value will be the minimum value between any component of the liner system; soil, geomembrane or geotextile. It is, of course, material dependent so that each specific material will have to be experimentally evaluated. This paper describes such experiments and presents data on a wide range of soil types, geomembranes and geotextiles.

TEST DETAILS AND PROCEDURES

A modified direct shear apparatus was used to evaluate friction values between soils, geomembranes and geotextiles in various combinations. In this type of test, the two materials being evaluated were placed in a split shear box, as shown in Figure 4. The shear box used had dimensions of 10.16 X 10.16 centimeters. For soil testing the depth in each part of the shear box was 2.54 centimeters of soil. For composite soil and geomembrane or soil and geotextile testing, the soil was placed in the upper half of the shearbox and the fabric was in the lower half. Rather than laying loose in the lower half of the shear box, the geomembrane or geotextile was firmly attached to a plexiglass block so that wrinkling could not occur. For geomembrane on geotextile testing, each material was attached to a separate plexiglass block and placed opposing one another in the two parts of the test device. All materials were tested in saturated condition, with the soils being placed at about 90% of their maximum density (ASRM D-698). This apparatus and techniques appears to be easier to perform than other shear box tests and be more representative of field boundary conditions than pullout tests, see Collios, et a.(5)

The normal stress range used in these tests was varied from 2.0 psi to 15 psi. These values are somewhat lower than in normal geotechnical testing but probably better reflect the low normal stresses that shallow cover soils impose on typical liner systems. The shear phase of the test was deformation controlled at a displacement rate of .127 millimeters/min. This low deformation rate assured complete dissipation of pore water pressures during the test. Typical data that resulted from these tests are shown in Figure 5. Here a set of different types of geomembranes were each tested with a concrete sand (sieved through a #10 sieve) at 6.0 psi of normal stress. Typical elastic-plastic response curves are observed, each having a well defined maximum value of shear stress.

Upon testing these same sets of materials at different normal stresses one can plot the peak shear stress versus applied normal stress on Mohr's stress space, as shown in Figure 6. Note that all failure envelopes pass through the origin attesting to the fact that there is no (or non-measurable) cohesion in the soils tested nor adhesion between these soils and the fabrics evaluated. (This would not have been the case if fine grained soils such as clays or cohesive silts had been used). The slope of these curves, often presented as an angle, is the desired value for design purposes. In all cases in this study, the response was

linear and the data spread in a given locus of points was nominal.

After each shear failure, the direction of deformation was reversed, and the test repeated. The purpose of this exercise was to indicate residual friction angles where membrane tension is alternately increased and reduced as the level of a storage lagoon changes. Such reversals of strain direction may tend to align particles along the shear plane, and reduce slip resistance. However, the difference between initial and repeated shear strengths was negligible in all cases.(3)

MATERIALS TESTED AND RESULTS

Three granular soil types were used in these tests:

(1) Ottawa sand (SP) with d_{10} = 0.42 mm; CU = 1.9 and rounded particle shapes.
(2) Concrete sand (SP) with d_{10} = 0.20 mm; CU = 2.6 and angular particle shapes.
(3) Mica schist silty sand (SM) with d_{10} = 0.057 mm; CU = 5.1 and angular particle shapes.

Thus the three soil types selected give a contrast in particle shape, size and uniformity. They are limited however, to granular soils with essentially no plasticity.

Four types of geomembranes (using five separate surfaces) were used in these tests. They were all tested in their manufactured directions.

(1) High density polyethylene (HDPE) which was 20 mils thick and can be characterized as being stiff, hard and smooth as far as physical or frictional characteristics are concerned.
(2) Ethylene propylene diene monomer (EPDM) which was 30 mils thick and can be characterized as being flexible, soft and smooth.
(3) Polyvinyl chloride (PVC) which was 30 mils thick and characterized as being of medium stiffness and hardness and rough on one side while smooth on the other side. Both sides were used during these tests.
(4) Chlorosulfonated polyethylene (CSPE) which was reinforced with a fabric scrim and was 36 mils thick. It is characterized as being of medium stiffness and hardness, but was of wavy roughness due to the laminated 10 x 10 scrim reinforcement contained within it.

Four types of geotextiles were used in these tests which represented each of the general manufacturing classifications of these materials.(6) They were all tested in their manufactured directions.

(1) Woven monofilament polypropylene fabric (Carthage Mills Polyfilter X) which is characterized as being a thin, stiff fabric with a relatively high percent open area as far as physical or frictional characteristics are concerned.
(2) Woven silt film (tape) polypropylene fabric (Mirafi 500 X) which is characterized as being a thin, flexible fabric with a low percent open area.
(3) Nonwoven heat set polypropylene fabric (duPont 3401) which is characterized as being a thin, flexible fabric with a relatively low open area.
(4) Nonwoven needled polypropylene fabric (Crown Zellerbach 600) which is characterized as being a compressible, thick, bulky, very flexible fabric with a relatively high open area.

These three soil types, four geomembranes types and four geotextile types were tested within their own categories and against one another in the manner described in the previous section. The results are given in Table 1 in two ways. The principal information (for design purposes) is given as angular values of friction angle; "ϕ" values for the soil by itself and "δ" values for the composite behavior. In parenthesis is given the relative amount (for comparison purposes) of mobilized soil strength that the geomembrane or geotextile gives, i.e.,

$$E = \frac{\tan \delta}{\tan \phi}$$

where

E = efficiency ratio
$\tan \delta$ = tangent of soil to material friction angle
$\tan \phi$ = tangent of soil friction angle, where
$\tau = c + \bar{\sigma}_n \tan \phi$
c = cohesion (zero for these granular soils)
$\bar{\sigma}_n$ = effective normal stress

Table 1 - Summary of Friction Angle and Efficiencies (in Parentheses) For Soils, Geomembranes and Geotextiles Testing in this Study

(a) Soil to Geomembrane Friction Angles

Geomembrane \ Soil	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist (ϕ = 26°)
EPDM	24° (.80)	20° (.71)	24° (.92)
PVC (Rough)	27° (.90)	-	25° (.96)
PVC (Smooth)	25° (.83)	-	21° (.81)
CSPE	25° (.83)	21° (.75)	23° (.88)
HDPE	18° (.60)	18° (.64)	17° (.65)

(a) Soil to Geotextile Friction Angles

Geomembrane \ Soil	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist (ϕ = 26°)
CZ 600	30° (1.00)	26° (.93)	25° (.96)
Typar 3401	26° (.87)	-	-
Polyfilter X	26° (.87)	-	-
500 X	24° (.80)	24° (.86)	23° (.88)

(c) Geomembrane to Geotextile Friction Angles

Geotextile \ Geomembrane	EPDM	PVC (R)	PVC (S)	CSPE	HDPE
CZ 600	23°	23°	21°	15°	8°
Typar 3401	18°	20°	18°	21°	11°
Polyfilter X	17°	11°	10°	9°	6°
500 S	21°	28°	24°	13°	10°

INTERPRETATION OF RESULTS

Table 1, parts "a" and "b" show the results of the direct shear tests for friction between various soils and synthetic materials in terms of friction angle (ϕ or δ)

and relative efficiency (E). It can be seen that the friction between all soils and the geotextiles or geomembranes is less than that of the soil itself. Consequently, soil to fabric friction governs the design of a slope, recall Figure 3. Soil to geotextile friction generally exceeds soil to geomembrane friction. Therefore, placement of a geotextile over or under a liner (as discussed in the introduction) will tend to allow a steeper slope, provided that both fabrics are securely anchored. If the anchor fails, then the safe slope angle will obviously be decreased. Part "c" of this table shows that geotextile to geomembrane friction is relatively low and depends greatly on the particular type of geomembrane being used.

Certain additional trends can be inferred from the data of Table 1 that allow prediction of the behavior of other materials not represented in the testing program. The three soils were selected to indicate the influence of particle angularity and gradation. For instance, EPDM is a smooth, flexible and surficially soft material. The friction angle with angular soil is higher than that with rounded soil. Here, the higher friction resulted from surface penetration, and surface scratches in the geomembrane were noted with the concrete sand tests. A high relative efficiency (92%) was obtained with the well graded silty sand probably due to the high contact area between the soil and the geomembrane and the surface roughness induced by distorting, but not piercing, the soft surface. Thus, it is worthwhile to use angular and well graded cover soils on soft membranes.

In contrast, the stiff, hard and smooth HDPE was fairly insensitive to soil type. Surface roughness is not induced by normal stress on the HDPE to soil interface, and low friction angles and relative efficiency indexes result. It would appear that it is necessary to place and anchor a geotextile over the material in order to build a steep slope with HDPE.

As expected, the angular soil readily penetrated into most of the geotextiles, and the relative efficiencies of all geotextiles and particularly, the needled-punched fabric, are particularly high. One generalization that can be made is that is it easier to estimate soil to geotextile friction for nonwoven than woven fabrics. There are a wide range of fabric openings in the non-wovens, whereas the woven geotextiles have a more regular pattern and limited opening size range. Hence, while the specific gradation of one soil type may allow considerable fabric penetration, a slightly coarser soil will not interlock as well. However, this analysis does not take into account the tensile strength or puncture resistance of woven materials; parameters which may be of equal importance in a particular situation.

Certain additional trends are evident in part "c" of Table 1. The pliable EPDM readily takes on the imprint of the opposing geotextile during conducting of the test, producing a surface roughness resulting in improved behavior. Hence, special care must be taken to assure that an overlying or underlying geotextile is securely anchored. The relatively stiff woven monofilament geotextile, substantially interacts (mechanically) with only the EPDM. The effect of geotextile stiffness is particularly evident with the scrim-reinforcement CSPE, such that the relatively stiff monofilament geotextile imprints the CSPE material around the reinforcement grid, but does not deform sufficiently to contact much of the soft material below and between the grid.

It must be noted, however, that the selection of a liner system (geomembrane, geotextile and soil cover) is dependent not only on the above friction behavior but also on the basis of chemical resistance to the impounded materials, availability and cost. As noted in the introduction, geotextiles are employed with liners for purposes other than friction. Finally, the subgrade soil is usually that which is native to the site. Consequently, the cover soil is often the only material of concern which can be selected largely on the basis of its mechanical properties.

SUMMARY AND CONCLUSIONS

Proper design of geomembrane lined side slopes is necessary whenever slopes greater than approximately 4 (horizontal) on 1 (vertical) are contemplated. Since this usually is the case (except in areas where large land areas are available), one must consider at least two different failure mechanisms. One is a general slope stability failure of a large mass consisting of the liner system and subsoils which is an area beyond the scope of this paper but well within the state-of-the-art. The other is linear slippage between individual components of the liner system or of the cover soil. This latter aspect was the concentration in this study. Elements of the general design were presented illustrating the need for experimental data on friction between soils, geomembranes and geotextiles. Toward supplying this needed data base, a modified direct shear test was used on a variety of materials of different interfaces.

Three soil types, four geomembranes and four geotextiles were evaluated, where the geomembranes mobilized from 60% to 86% of the soil friction and the geotextiles mobilized from 80% to 100% of the soil friction of those soils tested. Friction values for geomembranes on geotextiles were relatively low, suggesting the need for careful choice between materials when used in a composite manner and high assurance of anchor integrity. The need for additional data in this regard seems justified.

Concerning additional investigations on this subject, the lack of data using soils with cohesion is obvious. Indeed, such soils are encountered as subgrade materials, and their shear strength values (cohesion and friction) should be evaluated. Regarding design much remains. Included here was a limit equilibrium method of analysis. Needed is a method which is based on the entire stress vs. strain behavior of the materials involved. Work is currently ongoing in this regard.

ACKNOWLEDGEMENTS

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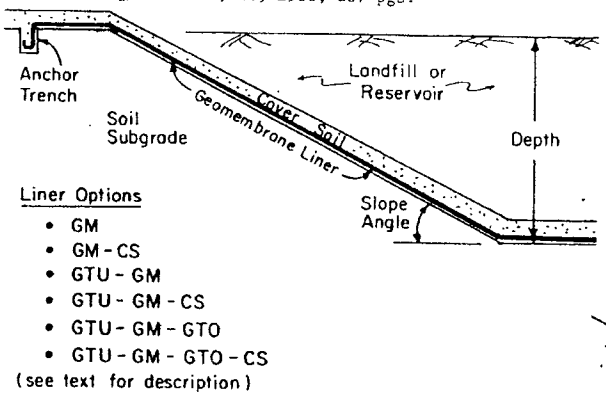


Fig. 1. - Typical Cross Section of Impoundment or Reservoir Slope with Geomembrane Liner System and Cover Soil

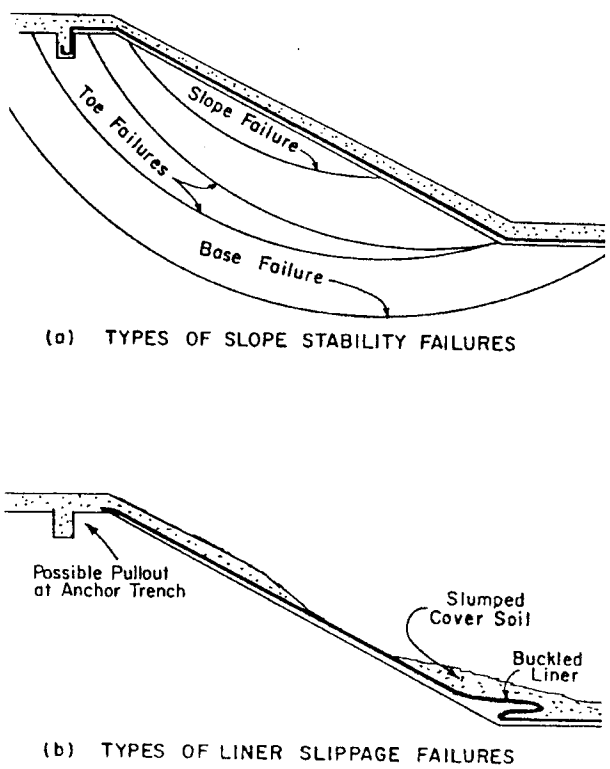


Fig. 2. - General Types of Failures of Lined Impoundments or Reservoir Slopes

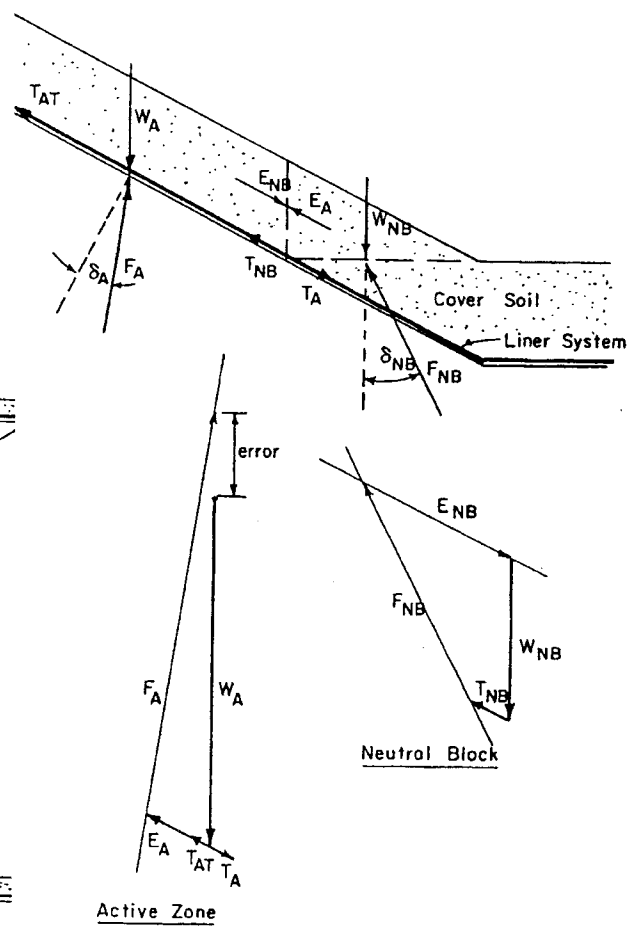
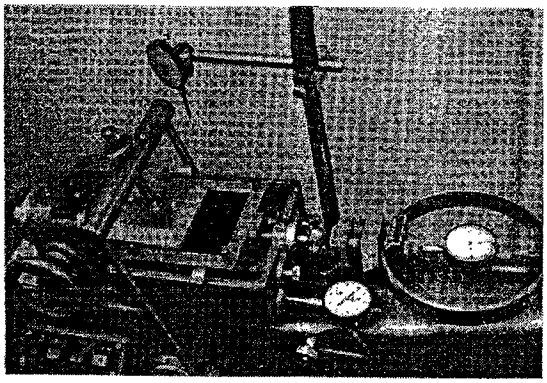
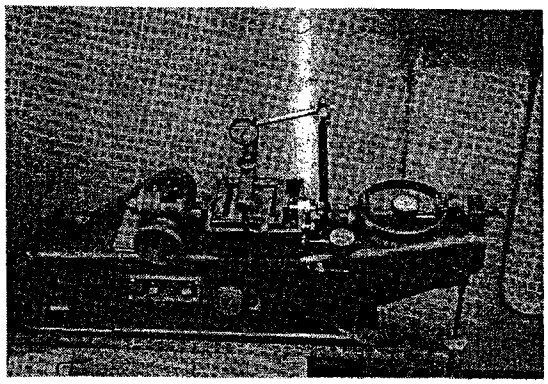


Fig. 3 - Design Details of Geomembrane Liner and Cover Soil Under Incipient Slippage Failure with Corresponding Force Polygons





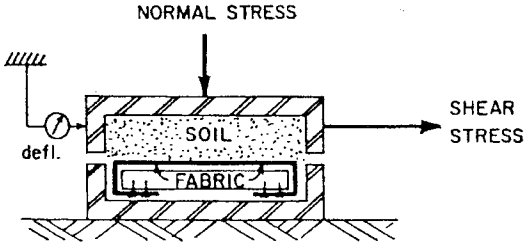


Fig. 4 - Photograph and Schematic Drawing of Direct Shear Device Used to Determine Friction Values in this Study

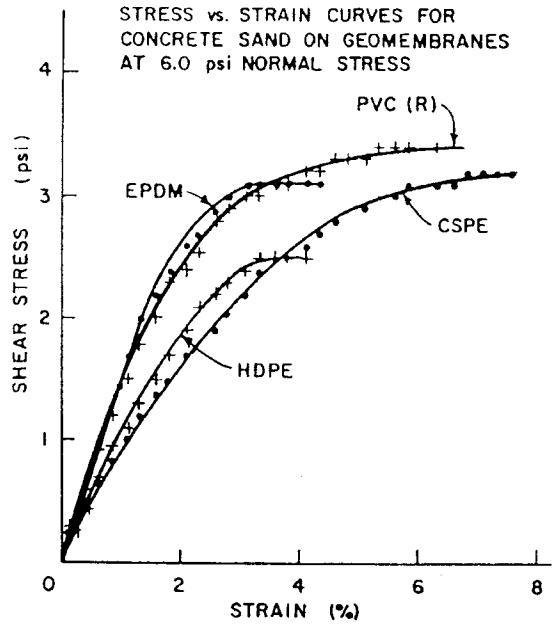


Fig. 5. - Typical Shear Stress vs. Strain Curves Generated in this Study. Illustrated is Concrete Sand on Four Geomembranes at 6.0 psi Normal Stress (Values Include 0.50 psi Machine Tare)

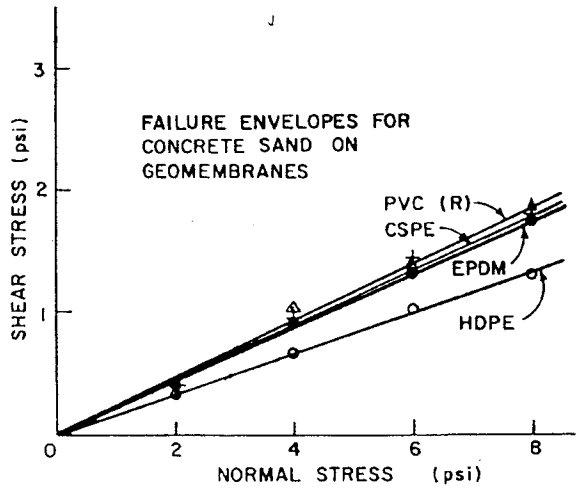


Fig. 6. - Failure Envelopes for Concrete Sand on Four Geomembranes Where Resulting Friction Angles Range from 18° to 27° (Figure 5 Values are Included here at 6.0 psi Normal Stress)

COPY OF REFERENCE SOURCE:

Williams, N.D., and Houlihan, M.F., "Evaluation of Friction Coefficients Between Geomembranes, Geotextiles, and Related Products," Proceedings of the 3rd International Conference on Geotextiles, IFAI, Vienna, 1986.

Third International Conference on Geotextiles, 1986, Vienna, Austria

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EVALUATION OF FRICTION COEFFICIENTS BETWEEN GEOMEMBRANES, GEOTEXTILES AND RELATED PRODUCTS

EVALUATION DU COEFFICIENT DE FROTTEMENT ENTRE GEOMEMBRANES, GEOTEXTILES ET PRODUITS ASSIMILES

BESTIMMUNG VON REIBUNGSWINKELN ZWISCHEN GEOTEXTILIEN, GEOMEMBRANEN UND VERWANDTEN PRODUKTEN

Interface friction parameters are presented for 16 geosynthetics on 42 interfaces. The friction parameters were measured using a modified direct shear device under conditions which closely model field conditions. Descriptions of the analytical methods and equipment are presented. The analytical results indicate that the interface friction angle is largely a function of the type of geosynthetic, polymer type, and the contact surface area and the geometry of synthetic drainage materials. The type of soil and soil compressibility may also impact the magnitude of the interface friction angle, particularly when placed against flexible geosynthetics which are in direct contact with open matrix or highly porous materials such as synthetic drainage nets.

Traennflaechen-Reibungskoeffizienten werden fuer 16 Geo-Kunststoffe auf 42 verschiedenen Trennflaechen vorgestellt. Die Reibungskoeffizienten wurden mit Hilfe eines direkten Scherversuchs-Geraet bestimmt, das den wirklichen Gegebenheiten im Feld sehr nahe kommt. Die analytischen Methoden und Geraete werden beschrieben. Die Ergebnise zeigen, das der Trennflaechen-Reibungskoeffizient hauptsaechlich eine Funktion vom Polymer-Typ, Oberflaechenrauhigkeit, Faesergroesse(bzw-durchmesser) und Maschenabstand ist. Die Bodenart und Bodensteifigkeit haben auch einen Einfluss auf die Groesse des Trennflaechen-Reibungswinkels. Besonders dann, wenn der Boden gegen flexible Geo-Kunststoffe, welche in direkten Kontakt mit offenen Maschen oder besonders poroesen Materialien z.B. synthetische Entwaesserungsnetze, anliegt.

1. INTRODUCTION

Recent releases of toxic chemicals to the environment such as the Dioxin contamination and subsequent evacuation of the city of Times Beach, Missouri, have increased the public awareness of the problems associated with disposal of chemical waste materials. In an effort to provide safe, long term containment of chemical wastes, and in response to congressional requirements, the United States Environmental Protection Agency (EPA) has provided a guidance document for the design of hazardous waste landfills and surface impoundments (1984,8).

The guidance document mandates the use of double liners with leachate collection and leak detection layers as shown in Figure 1. The guidance document recommends a landfill profile consisting of a leachate collection layer, a primary liner, a leak detection layer, and a composite secondary liner. Conventional leachate collection and leak detection layers typically consist of 30.5 cm (12 inch) thick layers of sand with minimum hydraulic conductivities of .01 cm/sec. The guidance document makes provision for the use of "innovative" materials in the leachate collection and leak detection layers. These "innovative" materials may be thick, highly transmissive woven or nonwoven geotextiles, synthetic nets, or other drainage materials.

The primary liner is a flexible membrane of sufficient thickness to resist puncture or degradation due to chemical contact or applied stress. The secondary composite liner consists of a flexible membrane overlying a 91 cm (36 inch) thick layer of compacted clay with a maximum hydraulic conductivity of 1x10-7 cm/sec.

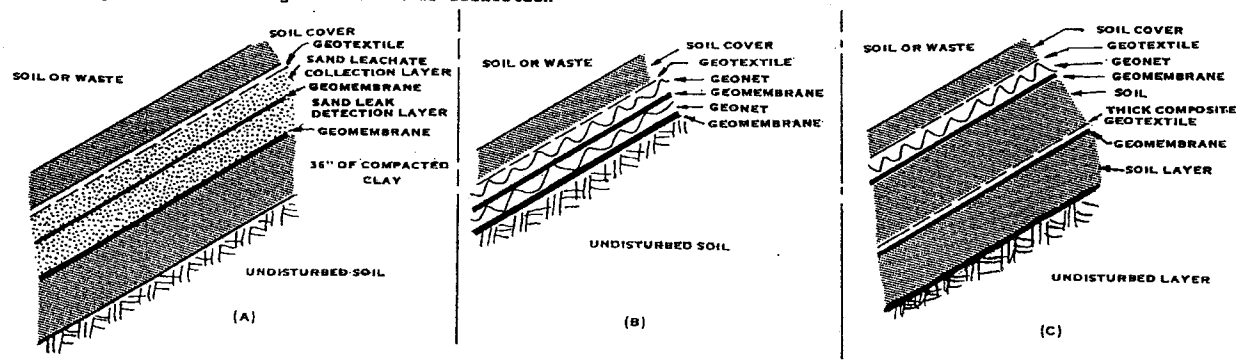

FIGURE 1. TYPICAL CROSS-SECTIONS OF COMPOSITE DOUBLE LINER SYSTEMS FOR HAZARDOUS WASTE LANDFILLS.

Due to the expense involved in constructing hazardous waste landfills and surface impoundments, and the difficulty in obtaining an operating permit, the optimization of space within the landfill is a primary consideration in design. Since the surface area of the landfill is typically limited, the most efficient use of available space typically requires that the side slopes of the landfill be constructed at as steep an angle as possible.

In order to construct the double liner systems on steeper slopes than is possible using sand or gravel leachate collection or leak detection layers, synthetic nets and other drainage materials have become widely used. However, the evaluation of stability of the double liner systems, especially when synthetic layers are placed in direct contact, is not straightforward. Indeed, heretofore, data did not exist to evaluate friction between layers and the overall stability of a slope with multiple layers of synthetics parallel to the slope.

A modified direct shear device has been used to evaluate the friction coefficients between four types of flexible membrane liners (FML), seven types of geotextiles, three types of synthetic nets, and two other types of drainage materials. The equipment, methodology and results of the analyses are discussed subsequently.

2. BACKGROUND

The direct shear device is widely used to evaluate interface friction coefficients between cohesionless soils and geosynthetics (3,6,7, and 8), and between multiple layers of geosynthetics (6). The analyses are typically performed at constant rates of strain ranging from 0.127 to 100 mm/min (0.005 to 4 inches/min) at normal stresses ranging from 10 to 383 kPa (209 to 8000 psf). The type of test, description of the sliding interface, coefficient of friction, interface friction angle and adhesion from the previous analyses are summarized in Table 1.

There are several fundamental differences in the testing equipment and methodologies used to evaluate the friction parameters. Two basic types of analyses are performed: pullout and direct shear. In the pullout test (2) the geosynthetic is pulled relative to the adjacent soil layers. The distribution of shear stress is nonlinear until translation occurs. Since the horizontal load is applied directly to the geosynthetic, the geosynthetic tends to stretch relative to the soil, creating a very smooth surface. Therefore, the friction parameters measured in a pullout test are likely to be lower than those measured in a direct shear test.

In a direct shear test (3,6,7, and 8), the horizontal lead is applied to the top soil layer, thus modeling the actual stress transfer conditions in the field. The geosynthetic may be placed loosely between soil layers (Section 4) and allowed to slide on the plane of minimum resistance. Since the geosynthetic surface may be irregular (when it isn't stretched), greater horizontal loads are required to cause sliding because work must be performed as dilation occurs at the interface.

The method used to mount the sample in the direct shear tests varies from test to test. Martin et al. (1984, 6) attach the geosynthetic to a plexiglass plate in such a way that no sliding can occur. A soil layer is displaced relative to the plexiglass plate/geosynthetic layer to evaluate friction coefficients. A similar procedure is employed by Myles (1982, 7). Due to the stiff, smooth geosynthetic surface and the small specimen dimensions (6), the interface friction parameters are slightly lower than the field values.

TABLE 1. SUMMARY OF THE INTERFACE FRICTION PARAMETERS MEASURED IN PREVIOUS ANALYSES

REF NUM	TEST METH	INTERFACE DESCRIPTION	u (-)	d (deg)	a (psf)
3	DS	NW Geotextile/HDPE	0.16	9	0
3	DS	NW Geotextile/PVC	0.25 to 0.19	14 to 24	0 to 100
6	DS	Fibertex 600/EPDM	0.42	23	-
6	DS	Fibertex 600/PVC	0.40	22	-
6	DS	Fibertex 600/CSPE	0.27	15	-
6	DS	Fibertex 600/HDPE	0.14	8	-
6	DS	Typar 3401/EPDM	0.32	18	-
6	DS	Typar 3401/PVC	0.34	19	-
6	DS	Typar 3401/CSPE	0.38	21	-
6	DS	Typar 3401/HDPE	0.19	11	-
6	DS	Polyfilter X/EPDM	0.31	17	-
6	DS	Polyfilter X/PVC	0.19	11	-
6	DS	Polyfilter X/CSPE	0.16	9	-
6	DS	Polyfilter X/HDPE	0.11	6	-
6	DS	Mirafi 500S/EPDM	0.38	21	-
6	DS	Mirafi 500S/PVC	0.49	26	-
6	DS	Mirafi 500S/CSPE	0.23	13	-
6	DS	Mirafi 500S/HDPE	0.18	10	-

DS = Direct shear device
u = Coefficient of friction
d = Interface friction angle
a = Adhesion

Saxena and Budiman (1985,9) utilize a modified direct shear device to accommodate a geosynthetic layer between two soil layers. The top soil layer used in the analyses is a synthesized clayey sand while the bottom layer is a crushed limestone. The geosynthetic is constrained on one boundary so that sliding could occur either between the geosynthetic and the clayey sand or between the geosynthetic and the limestone. Due to the irregular boundary at the geosynthetic/limestone interface, the interface friction values between the geosynthetic and clayey sand may be slightly higher than they would be if the clayey sand is placed on both sides of the geosynthetic.

3. EQUIPMENT

The modified direct shear device at the Georgia Institute of Technology (See Figure 2) accommodates specimens with dimensions of 30.5 by 30.5 cm (12 by 12 inches) placed between two soil layers which are about 5 cm (2 inches) thick. A force is applied perpendicular to the sliding surface by a pneumatic piston and yoke which is mounted on lerar ball bushings to provide frictionless movement during sliding. The bottom soil layer is anchored to a table and remains stationary during the test. The horizontal force is provided by another pneumatic piston device attached to the top soil mold. The horizontal and normal forces acting on the specimen are measured using load cells and signal conditioning equipment.

The maximum horizontal and vertical stress which can be applied by the apparatus as presently configured is about 100 kPa (2000 psf). The rate of deformation of the horizontal piston can be varied from about 0.003 to 0.3 mm/min (0.0001 to 0.01 in/min). The horizontal displacement of the top soil layer relative to the bottom soil layer is monitored using either a dial gauge or LVDT.

Due to the low total stress capacity of the system, the apparatus is best described as compliant. That is, the stress builds up slowly until the peak stress is reached, at which point there is sliding along a surface at a constant rate of strain. The slow build-up of stress with the compliant system makes it possible to measure

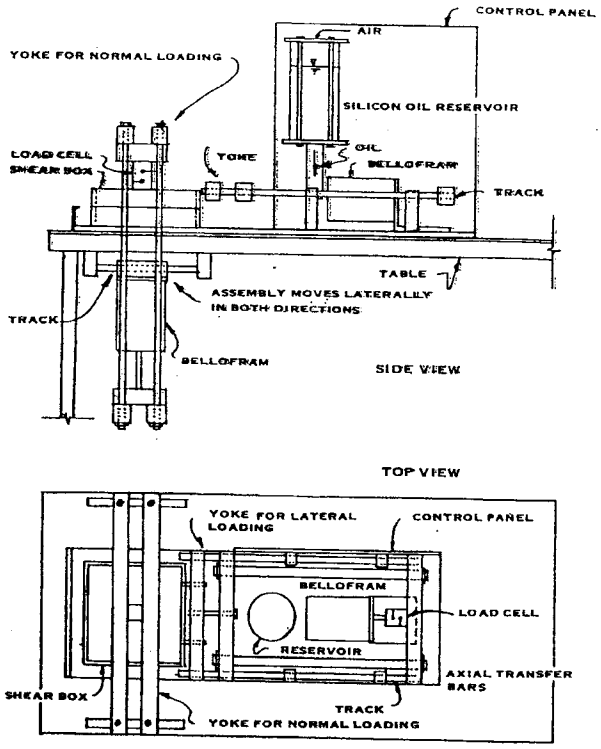

FIGURE 2. MODIFIED DIRECT SHEAR DEVICE

the peak stress and static coefficient of friction as well as the residual stress and dynamic coefficient of friction.

4. APPROACH

Interface friction values are measured for sixteen different types of geosynthetics and 42 interfaces. The geosynthetic layers are placed between two layers of soil as described in section 5. The geosynthetics used in the friction analyses can be divided into three classes of materials; flexible membrane liners (FML), geotextiles, and synthetic drainage products. The geosynthetics used in the analyses were selected because they are widely used for drainage applications at hazardous waste containment facilities.

4.1 Flexible Membrane Liners

4.1.1 Polyvinyl Chloride (PVC). Smooth, 30-mil thick PVC from Staff Industries with a grab tensile strength of 32 kN/m (183 lbs/in, ASTM D-751A).
4.1.2 High Density Polyethylene (HDPE). Smooth, 60-mil thick sheets of HDPE from Gundle Lining Systems with a grab tensile strength of 63 kN/m (360 lbs/in) and an elastic modulus of 760,000 kPa (110,000 psi).
4.1.3 Linear Low Density Polyethylene (LLDPE). Smooth 80-mil thick LLDPE from National Seal Corp. with a grab tensile strength of 29 kN/m (167 lbs/in) and an elastic modulus of 620,000 kPa (90,000 psi).
4.1.4 Chlorosulfonated Polyethylene (Hypalon). Polyester reinforced (10 by 10 scrim) Hypalon from Staff Industries with a grab tensile strength of 35 kN/m (200 lbs/in).

4.2 Geotextiles

4.2.1 Trevira 2125. Nonwoven, needlepunched, staple polyester geotextile from Hoechst Fibers Industries (HFI). The fabric is double-punched on one side and single-punched from the other side. The tensile strength is 25 kN/m (140 lbs/in, ASTM D-1682).
4.2.2 Trevira 1135. Nonwoven, needlepunched, continuous filament polyester geotextile from HFI. The grab tensile strength is 60 kN/m (340 lbs/in).
4.2.3 Trevira 6117. Nonwoven, needlepunched, staple, heat-calendared polyester geotextile from HFI. The grab tensile strength is 22 kN/m (125 lbs/in, MD).
4.2.4 Geolon 1500. Woven geotextile from Nicolon. Bi-directional strands are continuous and made of polypropylene (warp) and polyester (fill). The grab tensile strength is 193 kN/m (1100 lbs/in) in the warp direction and 490 kN/m (2800 lbs/in) in the fill direction.
4.2.5 Typar 3401. Nonwoven, continuous filament polypropylene heat bonded geotextile from Dupont. The grab tensile strength is 26 kN/m (150 lbs/in).
4.2.6 Fibertex 300. Nonwoven, continuous filament needle-punched polypropylene geotextile from Crown Zellerbach. The grab tensile strength is 37 kN/m (210 lbs/in).
4.2.7 Mirafi 140N. Nonwoven, continuous filament, needle-punched heat-bonded geotextile from Mirafi. Tensile strength is 21 kN/m (120 lbs/in).

4.3 Drainage Products

4.3.1 Tensar DN3W. Medium density polyethylene drainage net from the Tensar Corporation. The tensile strength is 4.4 kN/m (25 lbs/in).
4.3.2 Gundnett G-3. High density polyethylene drainage net from Gundle Lining Systems.
4.3.3 J-DRain 100. High density polyethylene drainage net with a geotextile glued to one side. Tensile strength is 5.3 kN/m (30 lbs/in).
4.3.4 Enkadrain. Nylon matting from Enka of America heat bonded to a nonwoven, polyester geotextile from Stabilenka. The grab tensile strength is 16 kN/m (94 lbs/in) in the machine direction and 9.5 kN/m (54 lbs/in) in the cross direction.
4.3.5 Miradrain. Polystyrene waffle structure drainage core from Mirafi glued to a nonwoven, heat-bonded polypropylene geotextile (Mirafi 140S).

4.4 Soil

The soil used in the friction analyses is a sandy clay soil, synthetically produced in the laboratory. The soil consists of 90% (by weight) Ottawa 20/30 sand and 10% Bentonite clay. The sand has a D50 of 0.7 mm and a D10 of 0.58 mm, with a uniformity coefficient of 1.3. The plasticity index of the soil is about 15.

Modified and Standard proctor tests (ASTM D1557 and D698, respectively) are performed to evaluate the relationship between compactive effort and the dry density and water content of the soil. The soil is compacted in the top and bottom molds at a specified density and water content. For the friction analyses the soil was compacted to a dry density of 14.2 kN/cubic meter (112.6 pcf) corresponding to 95% of the Modified proctor maximum dry density, and a water content of 16.5%.

5. METHODOLOGY FOR FRICTION ANALYSES

Following compaction of the soil in the bottom portion of the shear box, the geosynthetics are trimmed to 30.5 by 46 cm (12 by 18 inches) and mounted above the soil as shown in Figure 3. The top layer of soil is then placed

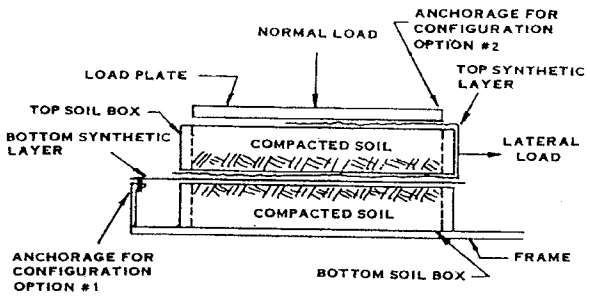

FIGURE 3. SPECIMEN CONFIGURATION

above the upper geosynthetic layer and compacted to the desired moisture content and density.

Three different specimen mounting procedures are used in the analyses. The first mounting configuration is used to evaluate the interface friction parameters between drainage layers and geotextiles. For these analyses, geotextile layers are placed above and below the synthetic drainage core. Sliding occurs on one of the planes between the geotextile and the drainage layer.

The second and third mounting procedures are used when it is necessary to measure the interface friction coefficients on an interface which has a higher coefficient of friction than another interface in the specimen. The second mounting prodedure is used in the evaluation of friction coefficient between the FML and the geotextiles or drainage cores. Since the lowest interface friction coefficients are between the soil and the FML, the FML must be attached to the frame (See Figure 3). This constrains sliding to be between the FML and the other geosynthetic.

In the event sliding occurs on the plane between the upper geosynthetic and the upper soil layer, the third mounting procedure is employed. Using this procedure, the lower geosynthetic layer is attached to the frame as in procedure 2. The top geosynthetic layer is wrapped around the leading edge of the upper shear box and placed under the load plate (See Figure 3). This constrains sliding to be between the two layer of geosynthetics.

After the specimen has been placed in the device and properly anchored, the top portion of the shear box is attached to the yoke, as show in Figure 2. The normal load (N) is then applied (890, 1110, 2225, or 4450 Newtons, or 100, 250, 500 or 1000 lbs, respectively) using a pressure regulator and the vertical Bellofram Piston Device. Once the normal stress has been applied, the oil reservoir is pressurized to 1670 kPa (80 psi) using the high pressure regulator.

The needle valve is adjusted to provide the desired rate of strain, and the three-way-valve is set to strain control. The horizontal displacement (HDG) and horizontal load (T) are then measured as a function of time for the duration of the test.

Once the peak horizontal load is reached, the pressure in the horizontal and vertical Bellofram Piston Devices is released, the soil is removed and recompacted, the specimen reconfigured, and the test sequence repeated at the desired normal stress.

6. DATA REDUCTION

The coefficient of friction, which is tabulated in Table 2, is the dynamic coefficient of friction at a strain rate of approximately 0.001/min. The residual stress

TABLE 2. RESULTS OF FRICTION ANALYSES

TEST NUM.	SLIDING SURFACE	u (-)	d (deg)	a (kPa)
1	HDPE/Trevira 2125	.179	10	1.9
2	HDPE/Trevira 1135	.217	12	1.2
3	HDPE/Fibertex 300	.177	10	1.6
4	HDPE/Geolon 1500	.165	9	1.7
5	PVC/Trevira 2125	.326	18	2.3
6	PVC/Trevira 1135	.327	18	2.1
7	PVC/Fibertex 300	.285	16	2.0
8	PVC/Geolon 1500	.361	20	1.9
9	LLDPE/Trevira 2125	.174	10	1.4
10	LLDPE/Trevira 1135	.217	12	1.9
11	LLDPE/Fibertex 300	.228	13	1.8
12	LLDPE/Geolon 1500	.171	10	1.3
13	Hypalon/Trevira 2125	.302	17	2.4
14	Hypalon/Trevira 1135	.435	24	1.2
15	Same as 14 (Quality Control Sample)			
16	Hypalon/Geolon 1500	.359	20	1.8
17	Hypalon/Fibertex 300	.453	24	1.5
18	HDPE/Tensar DN3W	.266	15	1.4
19	PVC/Tensar DN3W	.261	15	1.0
20	Hypalon/Tensar DN3W	.322	18	2.3
21	HDPE/Miradrain	.100	6	0.9
22	PVC/Miradrain	.436	24	1.9
23	Hypalon/Miradrain	.524	28	0.9
24	HDPE/Enkadrain	.161	9	1.3
25	PVC/Enkadrain	.311	17	0.9
26	Hypalon/Enkadrain	.439	23	0.0
27	Trevira 2125/DN3W	.350	19	1.9
28	Mirafi 140N/DN3W	.384	21	2.4
29	Typar 3401/DN3W	.385	21	0.3
30	Trevira 6117/DN3W	.390	21	1.0
31	Trevira 2125/J-DRain	.393	21	0.8
32	Mirafi 140N/J-DRain	.508	27	0.0
33	Typar 3401/J-DRain	.344	19	0.5
34	Same as 33 (Quality Control Sample)			
35	Trevira 6117/J-DRain	.378	21	1.6
36	Trevira 2125/Gun. G3	.376	21	1.9
37	Mirafi 140N/Gun. G3	.410	22	1.7
38	Typar 3401/Gun. G3	.344	19	0.5
39	Trevira 6117/Gun. G3	.374	21	2.9
40	Hypalon/J-DRain 100	.260	15	1.3
41	PVC/J-DRain 100	.197	11	1.3
42	HDPE/J-DRain 100	.200	11	1.9

during sliding is obtained from the graph of shear stress versus strain. The coefficient of friction at the interface is evaluated from a graph of the residual shear stress as a funtion of the applied normal stress (See Figure 4). The residual shear stress is calculated as follows:

$$\tau = \frac{T - f}{A} \qquad (1)$$

Where, T = The horizontal load (F),
f = The device friction (F),
A = The contact area between the sliding layers at the interface (L2),
τ = The shear stress (F/L2).

The contact area between the sliding layers at the interface varies as a function of time because the top portion of the shear box slides relative to the bottom portion of the shear box. The contact area at any instant in time is evaluated using the horizontal displacement:

$$A = Ai - (W)HDG \qquad (2)$$

Where, Ai = The initial contact area at the beginning of the analysis (L2),

W = The width of the specimen (L), and
HDG = The horizontal displacement (L).

The normal stress, σ, is equal to the vertical load, N, divided by the contact area, A. The coefficient of friction, μ, is the slope of the best fit straight line when the residual shear stress is plotted versus the corresponding normal stress. The interface friction angle, δ, is computed as follows:

$$\delta = \tan^{-1}(\mu) \qquad (3)$$

The intercept of the best fit straight line is the adhesion at the interface. The adhesion, a, is the shear stress at the interface when the normal stress is zero. Even though the adhesion contributes to the shear stress and friction mobilized at the interface, the adhesion is typically neglected in stability evaluations.

7. RESULTS OF FRICTION ANALYSES

The results of the 42 friction analyses, which are summarized in Table 2, can be divided into three types of analyses: Sliding between FMLs and geotextiles; sliding between FMLs and synthetic drainage materials; and, sliding between geotextiles and drainage nets.

7.1 FML/Geotextile Sliding

The geotextiles selected for this group of analyses are typically utilized for membrane underlayers. In general, the highest interface friction angles were measured against Hypalon, followed by PVC, LLDPE and HDPE. The interface friction angles varied from 17 degrees (Trevira 2125) to 24 degrees (Fibertex 300) for Hypalon, 16 degrees (Fibertex 300) to 20 degrees (Geolon 1500) for PVC, 10 degrees (Geolon 1500) to 13 degrees (Fibertex 300) for LLDPE, and 9 degrees (Geolon 1500) to 12 degrees (Trevira 1135) for HDPE.

The frictional resistance between geosynthetic layers is due primarily to sliding between layers and dilation at the interface. For very rough membranes, like Hypalon, dilation at the interface is the major component of the friction at the interface. As deformation occurs, energy is expended to displace the overlying soil layer upward to allow the goetextile to slide relative to the Hypalon. The work required for dilation is a function of the applied normal stress and the flexibility and stiffness of the geotextile.

For displacement of a geotextile relative to very flexible membranes such as PVC, sliding and dilation are also important. The displacement of the FML against a conformable soil boundary results in the formation of a slightly undulated surface. Due to the surface undulation, dilation occurs at the interface during sliding, resulting in a relatively high interface friction angle. Non-uniform woven geotextiles such as Geolon 1500 accentuate the formation of the undulations especially at high normal stresses, resulting in higher friction values. In addition, since PVC is a relatively soft material, the adhesion between the geotextile and PVC is slightly higher than it is for stiffer FMLs.

Both LLDPE and HDPE are relatively stiff, smooth FMLs when compared with Hypalon and PVC. Since the surface is relatively stiff and smooth, the primary source of friction between the geotextiles and the FMLs is sliding. The magnitude of the interface friction angles for displacement of geotextiles relative to LLDPE and HDPE is primarily a function of the polymer type, the contact surface area, and the surface roughness of both the geotextile and the FML. Woven geotextiles, which tend to have less area in contact with the geotextile, generally

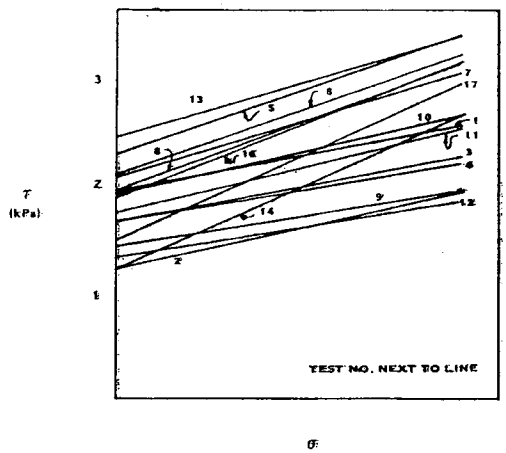

FIGURE 4. SLIDING BETWEEN FMLs AND GEOTEXTILES

have lower interface friction angles than do nonwoven geotextiles against HDPE and LLDPE. In addition, polyester geotextiles have slightly higher friction coefficients than polypropylene geotextiles for sliding against FMLs.

Increasing the surface roughness of the FML (embossment) can result in a very large increase in the interface friction angle. New embossment techniques have recently been introduced which provide a three-fold increase in the interface friction angle between geotextiles and the embossed FML.

7.2 FML/Drainage Layer Sliding

The friction parameters of four drainage materials; Tensar DN3W, Miradrain, Enkadrain and J-DRain 100, were evaluated against 3 FMLs; HDPE, PVC, and Hypalon. In general, the highest friction values were measured for sliding against Hypalon, followed by PVC, and HDPE. The interface friction angles varied from 15 degrees (J-DRain 100) to 28 degrees (Miradrain) for Hypalon, 11 degrees (J-DRain 100) to 24 degrees (Miradrain) for PVC, and 6 degrees (Miradrain) to 15 degrees (Tensar DN3W) for HDPE.

Since Hypalon has large surface undulations above the scrim, the interface friction coefficients are largely a result of work required for dilation. The highest interface friction angles were measured for Miradrain, which is a relatively stiff material. Since Miradrain is so stiff, it does not deform appreciably when the drainage layer is displaced relative to the FML. Therefore, more dilation or verticle displacement is required to get the Miradrain to move relative to the Hypalon. The other drainage materials are more flexible than Miradrain with less contact area, therefore the dilation and sliding components of the friction are lower.

The PVC liner is very flexible and conforms to the surface of the soil. At high normal stresses, stress concentrations develop at the contact points between the nodes of the drainage material and the membrane. Since the membrane is very flexible and the soil is compressible, the membrane/soil interface becomes distorted. The interface friction angle between the PVC and the drainage material is primarily a function of the compressibility, shear strength and drainage conditions

of the soil; the tensile strength and modulus of the FML; the stiffness, contact area and geometry of the drainage material; and, the normal stress. For very stiff drainage materials like Miradrain, the drainage material does not deform very much in shear so more work is required for dilation at the interface. Therefore, the interface friction values against PVC are higher for Miradrain than for less stiff, more conformable materials like J-DRain 100 and Tensar DN3W.

The HDPE liner is relatively stiff and smooth compared to Hypalon and PVC. The interface friction results primarily from sliding and is a function of the contact area, the polymer type and the stiffness of the FML and drainage material and the normal stress. Medium density polyethylene nets, like Tensar DN3W, have a relatively high contact surface area and a higher interface friction angle. Very stiff drainage materials with smooth contact surfaces like Miradrain have very low interface friction angles against HDPE.

7.3 Geotextile/Drainage Net Sliding

The interface friction properties between geotextiles and drainage materials are evaluated between four geotextiles (Trevira 2125, Mirafi 140N, Typar 3401 and Trevira 6117) and three drainage nets (Tensar DN3W, J-DRain 100 and Gundnett G3). In general, the interface friction angles are higher for Mirafi 140N, followed by Trevira 6117, Trevira 2125 and Typar 3401.

Since the geotextile is very flexible and the drainage nets are stiff compared to the soil, the soil compresses more adjacent to the nodes of the drainage nets. This results in the embedment of the geotextile and drainge net in the soil. The amount of embedment is primarily a function of the polymer type and stiffness of the drainage net; the contact area of the nodes; the polymer type, primary and secondary bonding, grab tensile strength and elastic modulus of the geotextile; and, the type, compressibility and shear strength of the soil.

For a given type of soil, the highest interface friction angles occur for the geotextiles with the lowest tensile strength and modulus. The amount of embedment is greater for these materials, therefore, more work is required to displace the net relative to the geotextile.

8. CONCLUSIONS

The following conclusions and observations are developed as a result of the friction analyses for the materials evaluated:

8.1 For sliding between FMLs and geotextiles, the highest average interface friction angles are against Hypalon (21 deg.), followed by PVC (18 deg.), LLDPE (11 deg.), and HDPE (10 deg.).
8.2 For sliding between FMLs and synthetic drainage materials the highest average interface friction angles are against Hypalon (25 deg.), followed by PVC (17 deg.), and HDPE (10 deg.).
8.3 For sliding between geotextiles and drainage nets the highest average interface friction angles are against Mirafi 140N (23 deg.), followed by Trevira 6117 (21 deg.), Trevira 2125 (20 deg.), and Typar 3401 (20 deg.).
8.4 The primary components of interface friction between multiple layers of geosynthetics are sliding between layers and dilation at the interface. The highest interface friction angles are developed between layers where a significant amount of dilation occurs.
8.5 The type and surface roughness of the flexible membrane liner have the greatest impact on the interface friction angle between FMLs and geotextiles and FMLs and synthetic drainage materials.
8.6 The type, tensile strength and elastic modulus of the geotextile have the greatest impact on the interface friction angle between geotextiles and drainage nets.
8.7 The interface coefficient of friction and adhesion are required to evaluate the shear stress mobilized at the boundary between two geosynthetic layers.
8.8 Soil should be compacted on both sides of the layers of geosynthetics in order to properly model field conditions.
8.9 For displacement rates between 0.003 and 0.3 mm/min, the shear strength is independent of the rate of displacement.
8.10 The shear stress mobilized on the boundary between layers of geosynthetics is directly proportional to the normal stress.
8.11 Due to the large strains required to mobilize the dynamic coefficient of friction (typically greater than 2 cm), the minimum width of the contact area in the direct shear device should be at least 20 cm.

The friction parameters generated in the analyses are believed to be accurate for the materials tested. Extreme care should be exercised in extrapolating the results of these analyses for geosynthetics produced by other manufacturers. In addition, the analyses were all performed against one type of soil. Friction analyses currently in progress indicate that type, compressibility and shear strength of the soil compacted adjacent to a geotextile may have a large impact on the interface friction values between geotextiles and drainage nets.

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Structural Integrity of Geosynthetic Lining and Cover Systems for Solid Waste Landfills

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ABBREVIATIONS

CCOEF – Composite Column on Elastic Foundation
COEF – Column on Elastic Foundation
EPA – Environmental Protection Agency
FEM – finite element method
GCL – geosynthetic clay liner
GG – geogrid
GM – geomembrane
GN – geonet
GT – geotextile
H:V – grade of slope (ratio of horizontal to vertical)
HAZ – hazardous waste
HDPE – high density polyethylene
IEPA – Illinois Environmental Protection Agency
LE – limit equilibrium methods
MSW – municipal solid waste
OSWR – Office of Solid Waste Research
PET – polyester
SCC – simple composite column model

SYMBOLS

α – interface adhesion (intercept for Mohr–Coulomb plot of interface strength)
β – slope angle
γ_c – unit weight of soil cover
δ – interface friction angle (slope for Mohr–Coulomb plot of interface strength)
δ – slope of the line of rupture
δ_L – interface friction angle for lower surface of structural component
δ_U – interface friction angle for upper surface of structural component
λ – COEF parameter comparing interface and axial stiffness
λL – non–dimensional COEF factor
σ_n – normal stress
τ – interface shear–stress
$\tau_{geometry}$ – shear stress applied by the weight of the waste
τ_{max} – maximum load able to be transferred from the waste to the lining system
τ_{soil} – maximum shear stress that can be transferred through the soil
ϕ – slope for Mohr–Coulomb plot of soil strength
ϕ_c – drained strength parameter for cover soil
ϕ_i – slope interface strength parameter
A_c – cross–sectional area in compression

A_t – cross-sectional area in tension
c – intercept value for Mohr-Coulomb plot of soil strength
C_1 – integration constant for solving differential equation
C_2 – integration constant for solving differential equation
E_c – elastic modulus in compression
E_t – elastic modulus in tension
f – uniformly distributed shear load
F_1 – shear resistance on block 1
f_n – net distributed shear load (equal to $f - f_s$)
f_s – distributed interface shear resistance
H – bench height
H_{max} – maximum height of cover
i – represents the node of interest
K_c – axial stiffness in compression ($= E_cA_c$)
k_s – interface stiffness
K_t – axial stiffness in tension ($= E_tA_t$)
L – length of slope
L_c – length of the column in compression
L_t – length of the column in tension
L_{TOT} – total length of column
L_{TOT} – total length of the column
m – number of layers
n – number of nodes per layer
N – normal load
P – axial load in slope column
P_1 – magnitude of buttress force on wedge 1
P_1 – the resisting buttress force
P_{max} – magnitude of P for $H = H_{max}$
P_p – compressive strength of the soil column
S – shear load
S – total shear load (equal to $f \cdot L_{TOT}$)
S_i – external shear force per unit width applied to node i
S_i – mobilized interface resistance along the bottom slope of the potential sliding surface
$S_{L\ strength}$ – the lower interface strength
$S_{L\ mobilized}$ – shear force mobilized along the lower surface of a structural component
$S_{L\ applied}$ – shear force applied along the lower surface of a structural component
S_L – shear force along the lower surface of a structural component
S_n – net shear load on column
S_s – mobilized soil resistance at the toe
$S_{U\ strength}$ – the upper interface strength
$S_{U\ applied}$ – shear force applied along the upper surface of a structural component
$S_{U\ mobilized}$ – shear force mobilized along the upper surface of a structural component
S_U – shear force along the upper surface of a structural component
T_c – cover thickness
T_{gs} – tensile load in the geosynthetic

u_i - displacement
W_1 - weight of block 1
z - interface shear displacement
[C] - a NxN matrix of coefficients
[S] - a 1xN matrix of external shear forces for each node
[u] - a 1xN matrix of displacements for each node

ACKNOWLEDGMENTS

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The contribution and cooperation of all those involved with this project is greatly appreciated.

ABSTRACT

Composite lining and cover systems containing geosynthetic and soil components are used extensively in waste containment facilities. The structural integrity of these systems must be maintained throughout the lifetime of the facility in order to provide for adequate performance. The state of practice for predicting the stresses generated within composite systems considers only limit state equilibrium. A finite difference method which considers both force equilibrium and load-displacement compatibility for predicting these stresses is presented and compared with the state of practice. The proposed finite difference method, GEOSTRES, accounts for the stress-strain behavior both within individual components and at the interfaces between components.

A database of axial load and interface shear test results for geomembranes, geotextiles, and geonets is described and used to define baseline interface properties. Parametric studies are conducted with the model to determine stress and strain levels in the geosynthetic components. Six cover system configurations and six lining system configurations are presented and analyzed to identify critical behavior and to evaluate the sensitivity to specific loading conditions.

Long, J.H., J.J. Daly, and R.B. Gilbert.
STRUCTURAL INTEGRITY OF GEOSYNTHETIC LINING AND COVER SYSTEMS FOR SOLID WASTE LANDFILLS
Final Project Report, Office of Solid Waste Research, University of Illinois, July, 1993, Urbana, Illinois, 284pp.

CHAPTER ONE
INTRODUCTION

While methods to reduce and recycle waste are important to identify and develop, landfills still receive the major portion of solid waste generated in Illinois, and will probably continue to do so for many years. In the state of Illinois alone, 93 solid waste landfills have applied for permits to initiate closure in 1992. Obviously, new landfills will continue to be sited and built, and old landfills will need to be covered.

Considerable regulatory effort has been directed toward ensuring that cover and lining systems for landfills are as impermeable and reliable as possible. Geosynthetic lining systems are often specified, to provide the reliability required by regulatory agencies. However, the soil/geosynthetic lining systems that perform so well to isolate the waste, can cause serious engineering concerns with respect to stability of the slopes within a landfill, and stresses within the soil and geosynthetics.

Better tools to predict stresses in landfill covers and linings are necessary to design safer, more economical landfills, and to assess advantages and disadvantages of changing landfill geometry, construction methods, or materials used in the lining or cover systems. For example, new products are being introduced into the landfill construction industry which provide greater interface strength. Higher interface strength may allow lining or cover slopes to be steeper, and provide greater economy for a landfill. Other new products include thin layers (e.g. geosynthetic clay liners, GCL) which may be used as a replacement for thick clay layers. However, the compressive and tensile strength, as well as the interface strength properties for GCL's are quite different from the materials they replace. A method is needed to assess effects of material properties and landfill geometry on stresses within landfill lining and cover systems.

Presented herein are the results of a study sponsored by the Office of Solid Waste Research (OSWR) to:

- develop a structural model for analyzing and predicting tensile stresses and behavior of lining and cover system,
- develop baseline configurations, geometries, and material properties, and
- use the structural model to assess and understand the behavior of lining and cover systems.

A numerical model called GEOSTRES was developed which predicts the behavior for a cover or lining system, and the stresses in each of the individual components that comprise the soil/geosynthetic composite. The formulation of GEOSTRES provides a major advantage over other, currently used methods. GEOSTRES maintains both equilibrium and strain compatibility to compute stresses and deformations within individual components (soil and geosynthetics) in the lining or cover system.

This report includes eleven chapters and eleven appendices. Chapter Two provides general background information and a statement of need. Baseline configurations, geometries and properties for a cover and lining system are established in Chapter Three. Identifying baseline values provides a basis for example cover and lining systems used throughout this report. Chapter Four describes a current analytical method that uses limit equilibrium to assess stability of cover and lining systems. The ability of the limit equilibrium method to predict tensile loads is demonstrated for a soil/geosynthetic cover system. Chapter Five introduces a limit method used for determining how stresses are distributed within a multiple layered cover or lining system.

Chapter Six introduces three simple numerical models for a cover or lining system using equilibrium and strain compatibility. However, the simple models assume either linear behavior, or limit behavior. A more general approach is discussed in Chapter Seven. The computer program GEOSTRES is introduced and described. The GEOSTRES model is a Column on Elastic Foundation (COEF) model which allows for multiple columns and interfaces. Each column may exhibit non-linear axial load deflection behavior. The interfaces may also exhibit non-linear behavior between shear load and shear displacement. These features are important because, as demonstrated in this report, both non-linear load-deflection behavior, and interface shear stress-displacement behavior, are commonly exhibited by geosynthetic and soil materials. Results are given for example cases to identify the differences between predictions made with a model that observes strain compatibility and equilibrium with models that observe equilibrium only.

Chapters Seven and Eight present the results of GEOSTRES analyses for several cover and lining systems, respectively. The baseline cover and lining systems for municipal and hazardous wastes, developed in Chapter Three, are analyzed. Variations from the baseline systems are also examined to illustrate the effects of substituting a rough geomembrane for a smooth geomembrane, and for including a stiff geogrid in the uppermost soil layer. Finally, conclusions are summarized in Chapter 10.

Several Appendices (A - K) are included to provide details on properties for interfaces used in this study. Appendix J provides the details for the solution for a column on an elastic foundation. Appendix K includes both the axial and interface mechanical properties for profiles 1-12. The appendices reflect a significant effort to collect, collate, and present data relevant to the material and interface properties of soil and geosynthetic materials.

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Landfill Design, Performance and CQA

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Structural Integrity of Composite Geosynthetic Lining and Cover Systems

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ABSTRACT

Composite lining and cover systems containing geosynthetic and soil components are used extensively in waste containment facilities. The structural integrity of these systems must be maintained throughout the lifetime of the facility in order to provide for adequate performance. The state of practice for predicting the stresses generated within composite systems considers only limit state equilibrium. A finite difference method which considers both force equilibrium and load-displacement compatibility for predicting these stresses is presented and compared with the state of practice.

INTRODUCTION

Composite lining and cover systems containing geosynthetic and soil components are used extensively in waste containment facilities. These systems serve as barriers to control the migration of contaminants to the environment. Stresses are generated within lining and cover systems during construction, waste placement and waste settlement. Rupture of any of the geosynthetic components within a composite system due to high stresses may compromise its ability to serve as an effective barrier. In addition, the expected loads carried by geosynthetic components are needed to design geosynthetic anchorage systems (e.g. anchor trenches at slope crests) properly. Finally, the long term performance of a geosynthetic (e.g. creep, stress cracking, etc.) is a function of the stress level in the geosynthetic. Therefore, accurate predictions of stresses within the geosynthetic components of lining and cover systems under different loading conditions can be of considerable importance.

The current state of practice for evaluating stresses within lining and cover systems is to consider limit state equilibrium (e.g. Mitchell, et. al. 1990; Koerner, 1990; and Giroud and Beech, 1989). The tensile force in each geosynthetic layer required to maintain equilibrium within the system is calculated by assuming that the maximum shear stress is mobilized at each interface within the system. There are two major limitations to this approach. First, the mobilized interface shear resistance is a function of relative displacement between adjacent layers. For relative displacements less than that required to mobilize peak interface strength, interface

shear resistance is roughly proportional to displacement. As the relative displacement increases beyond that required to mobilize peak strength, the shear resistance decreases from peak to a residual or large displacement strength. Therefore, a range of mobilized shear resistances is available for each interface, and selection of a single resistance value for use in the limit analysis is unclear. Second, limit state equilibrium methods ignore the axial stress-strain relationship within each layer. The initial axial stiffness may be significantly different among various geosynthetic and soil materials. As a lining or cover system is loaded, the stiffer components within the system tend to support more load than the less stiff components. Therefore, consideration of axial load behavior in addition to interface behavior is necessary to predict stresses accurately.

To account for the limit equilibrium limitations discussed above, Wilson-Fahmy and Koerner (1993) adopted a two-dimensional finite element approach. The finite element method provides a powerful computational tool for investigating two-dimensional effects while considering both shear resistance-displacement compatibility at interfaces and axial stress-strain compatibility within components. However, finite element tools typically require a level of effort in discretizing the geometry and performing numerical calculations that is more suitable for checking a final design configuration. Finite element methods generally are too cumbersome for analyzing multiple configurations required for design optimization and extensive parametric studies.

A finite difference method is presented herein to analyze composite lining and cover systems. This approach offers the advantage of simplicity in formulating the problem and in performing the numerical calculations while considering shear resistance-displacement and axial stress-strain compatibility. An example using a typical cover system is presented to compare results of the approach proposed with the currently accepted limit state approach and to evaluate factors which influence the distribution of stresses in composite systems.

PROPOSED APPROACH

The approach proposed for evaluating stresses within lining and cover systems uses inelastic, non-linear springs to model the shear resistance-displacement behavior at each interface and to model the axial load-displacement behavior within each component. Let m represent the number of components or layers within the system and let L represent the total length of the system. Each layer within the system is modeled one-dimensionally, and it is divided into a series of n nodes along the layer as shown on Fig. 1. Nodes are numbered successively down a column starting in the upper left corner of the system (Fig. 1), yielding N total nodes where N is equal to the number of nodes per layer (n) multiplied by the number of layers (m). Shear springs with stiffness k_s are located between nodes of adjacent components (layers). k_s is related to the interfacial shear modulus by the following relationship

$$k_s = \frac{\tau (L/n+1) W}{\Delta} \tag{1}$$

where τ represents the shear stress, Δ represents the relative displacement at the interface, L/(n+1) represents the distance between adjacent nodes in a layer, and W represents width. In this analysis, a unit width is assumed. In addition, axial springs with stiffness k_a are located between adjacent nodes within a given layer.

k_a is related to the axial modulus of the material by the following relationship

$$k_a = \frac{E_a A}{L/(n+1)} \tag{2}$$

where E_a represents the secant modulus, A represents the cross-sectional area for a unit width, and L/(n+1) again represents the distance between adjacent nodes. The boundaries at the perimeter of the system are assumed to be fixed.

The tensile load in each geosynthetic layer is evaluated by satisfying both system equilibrium and load-displacement compatibility. Consider an individual node as shown on Fig. 1 where i represents the node of interest. By evaluating force equilibrium in the x direction, the following equation is obtained

$$\begin{aligned}\sum F_x = S_i - k_{a(i,i-m)}(\delta_i - \delta_{i-m}) + k_{a(i+m,i)}(\delta_{i+m} - \delta_i) \\ - k_{s(i+1,i)}(\delta_i - \delta_{i+1}) + k_{s(i,i-1)}(\delta_{i-1} - \delta_i) = 0\end{aligned} \tag{3}$$

where δ represents displacement and S_i represents the external shear force per unit width that is applied to node i.

In considering force equilibrium for each node within the system, a system of N equations with N unknowns (i.e. each δ_i) is obtained. This system of equations is represented by the following in matrix form

$$[C]\cdot[\delta] = [S] \tag{4}$$

where [C] represents a NxN matrix of coefficients, [δ] is a 1xN matrix of displacements for each node, and [S] represents a 1xN matrix of external shear forces for each node. For a given row i in [C], the non-zero coefficients are given by the following

$$C_{i,i-m} = -k_{a(i,i-m)} \tag{5}$$

$$C_{i,i-1} = -k_{s(i,i-1)} \tag{6}$$

$$C_{i,i} = k_{a(i,i-m)} + k_{s(i,i-1)} + k_{a(i+m,i)} + k_{s(i+1,i)} \tag{7}$$

$$C_{i,i+1} = -k_{s(i+1,i)} \tag{8}$$

$$C_{i,i+m} = -k_{a(i+m,i)} \tag{9}$$

The system of N equations can be solved using standard algorithms to obtain the

displacement matrix for a given set of coefficients.

Since relationships between axial load and displacement and between shear load and displacement are nonlinear for most geosynthetic and soil components, an iterative approach is used to solve for displacements. The first iteration assumes initial values for each k_a and k_s and an initial set of displacements is obtained. The next iteration is based on displacements of the previous analysis. Values of k_a and k_s are re-evaluated, the analysis is performed, and a second set of displacements is obtained. The analysis is repeated until convergence in the displacements is achieved. A graphical representation of the iterative approach is shown on Fig. 2. An interactive computer program, GEOSTRES, has been developed to perform these calculations.

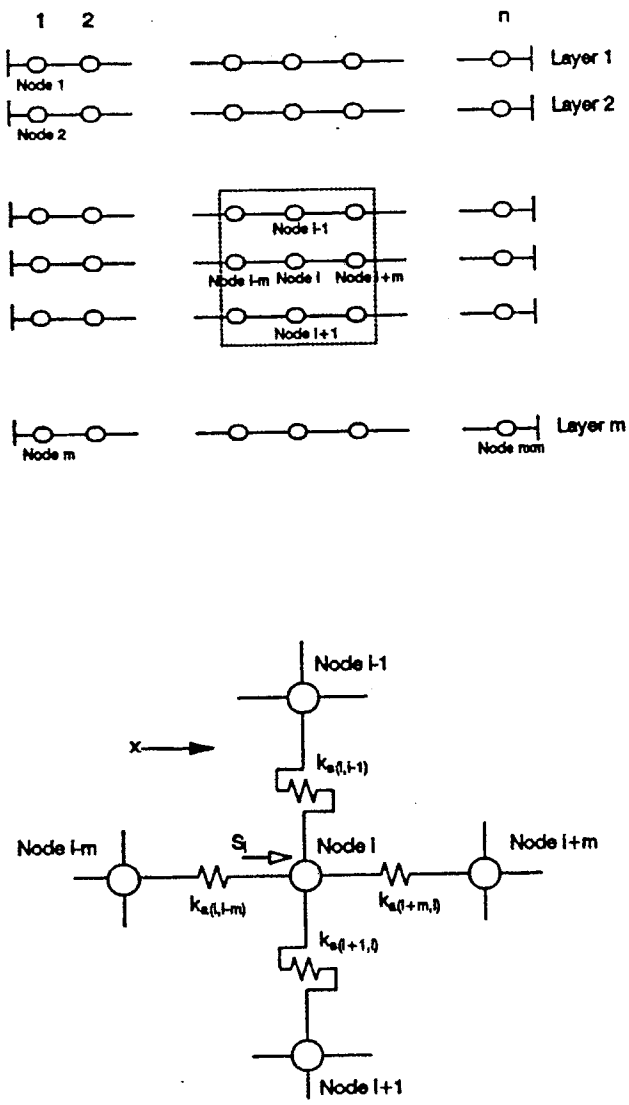

Figure 1. Model of lining or cover system.

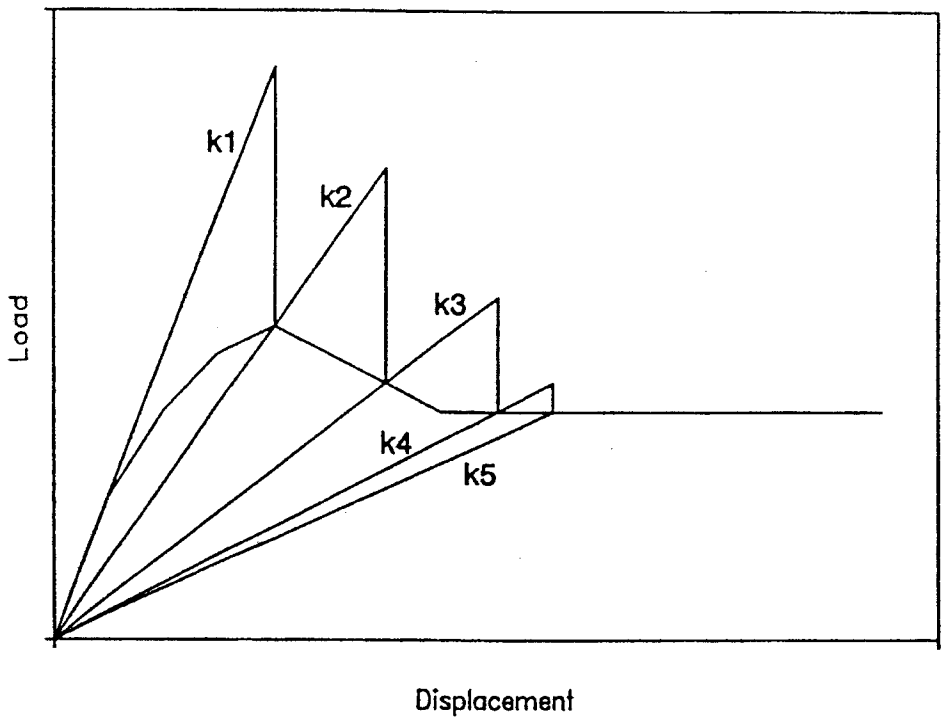

Figure 2. Graphical representation of iterative approach to obtain load-displacement compatibility.

ILLUSTRATIVE EXAMPLE

An example problem is presented to demonstrate the approach proposed for evaluating stresses in lining and cover systems. A typical composite cover system as shown on Fig. 3 is used for this purpose. A constant slope height of 8 m will be assumed in all calculations, while the grade of the slope will be varied. The soil cover is 0.92 m thick with a unit weight of 18.7 kN/m^3. The shear resistance parameters are presented in Table 1 for the various interfaces, and the load-displacement relationships used as input to GEOSTRES are shown on Fig. 4 for a given normal stress of 16.7 kN/m^2 (i.e. the normal stress at the bottom of 0.93 m of cover soil at 25 percent grade). The shear resistance versus displacement relationships are based on both a review of published data and unpublished large scale direct shear testing results. The sample sizes for these large scale tests are typically on the order of 300 mm by 300 mm. The axial load versus strain relationships for the various components are shown on Fig. 5. These relationships are typical of results from tensile tests on wide width specimens (Koerner, 1990), and represent short-term strengths (i.e. creep is not considered). As shown on Figs. 4 and 5, point-wise linear load-deformation relationships have been assumed for simplicity; however, the finite difference approach is general and non-linear relationships can readily be accommodated.

Limit Equilibrium Approaches Stresses within the cover system are predicted using the limit equilibrium approach proposed by Giroud and Beech (1989). Two slight

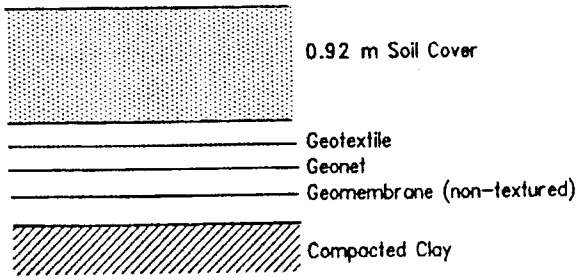

Figure 3. Composite system configuration for example problem.

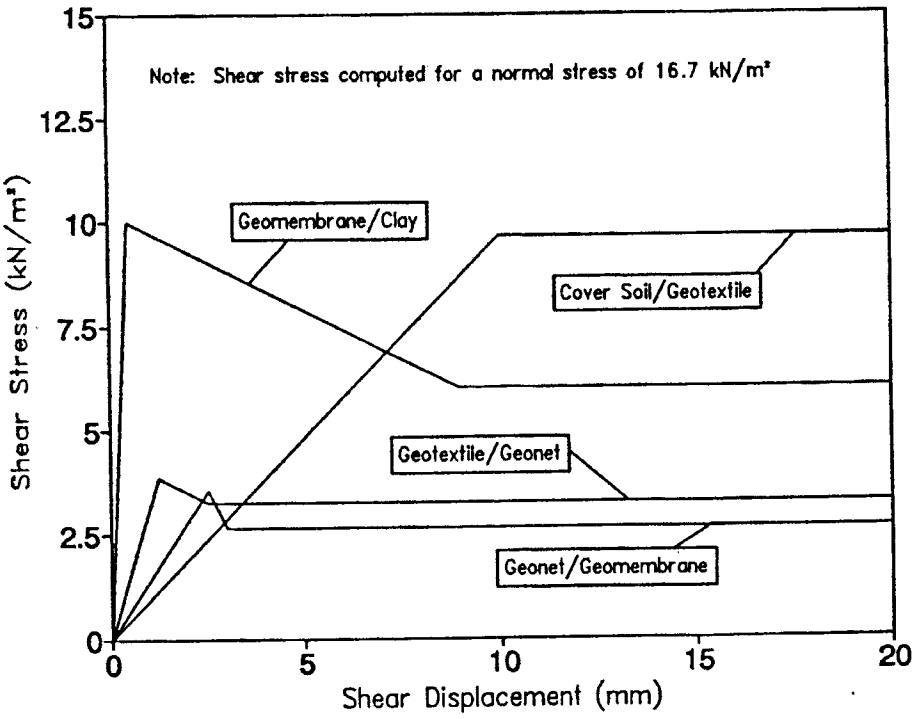

Figure 4. Shear resistance versus displacement for interfaces in example problem.

Table 1. Interface shear resistance versus displacement relationships for example problem.

	Peak			Large Displacement		
Interface	Friction Angle (°)	Intercept (kN/m)	Displace-ment (mm)	Friction Angle (°)	Intercept (kN/m)	Displace-ment (mm)
Cover Soil/ Geotextile	30	0	10	30	0	10
Geotextile/ Geonet	13	0	1.25	11	0	2.5
Geonet/ Geomembrane	12	0	2.5	9	0	3
Geomembrane/ Clay	0	10	0.5	0	6	9

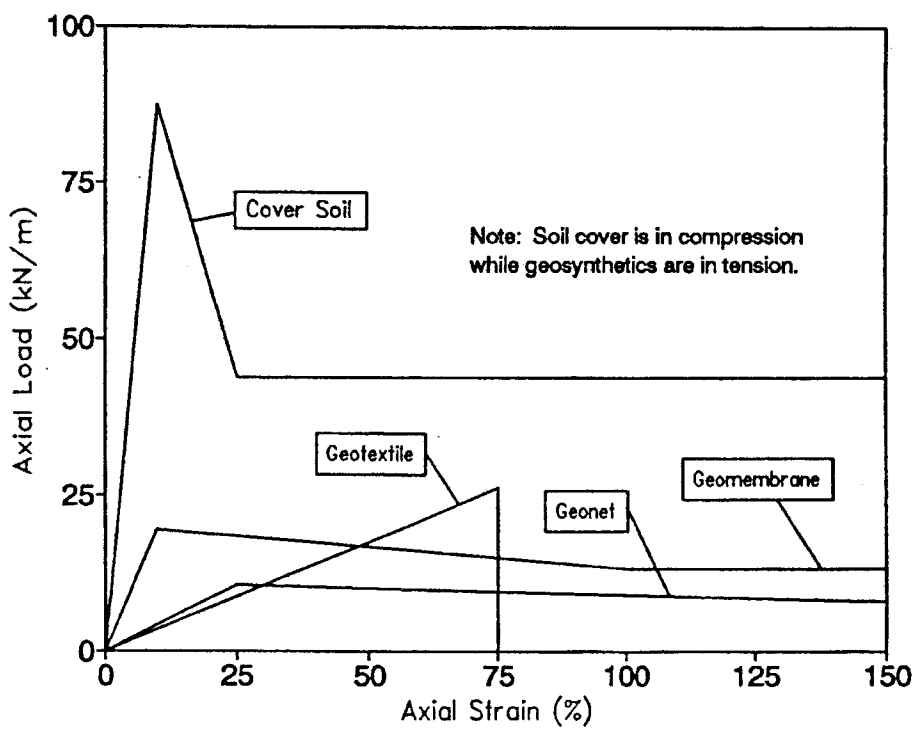

Figure 5. Axial load versus stress for components in example problem.

variations of the Giroud and Beech method are also used to illustrate the effect of buttress load on axial loads in the components. The limit equilibrium approach assumes a mobilized soil resistance at the toe (buttress) and a mobilized interface strength along the slope of the potential sliding surface. Any unbalanced forces are carried by tensile loads in the components above the potential failure surface. Results from three limit equilibrium methods are compared. Each of the methods are distinguished by assumptions that affect mobilization of the buttress load.

The first limit approach is consistent with the Giroud and Beech (1989) method. The method requires that all driving forces be resisted first by interface shear and compressive load in the soil. Tensile loads are carried by the geosynthetics only after driving forces exceed resisting forces provided by shear and the soil compressive load. The approach follows a rationale that the soil cover is the stiffest element (Fig. 5) in the cover system; therefore, a significant portion of the total driving force should be resisted by compressive loads in the soil. For limit analysis, the total driving force less the axial capacity of the soil is mobilized at the soil/geotextile interface while the large displacement resistance is mobilized at the other interfaces. The total tensile load that must be carried by the geosynthetic components (i.e. the summation of the individual geosynthetic loads) to satisfy equilibrium is plotted as a function of slope grade in Fig. 6 as the curve labeled "Maximum Soil Buttress." Since the geonet/geomembrane interface provides the minimum shear resistance (Table 1), the system will slip along this interface and the total tensile load in the geosynthetics is carried by only the geotextile and the geonet. The geosynthetics carry no tensile load until the soil reaches its axial capacity at a slope grade of approximately 44 percent (23.7°). This approach provides a lower bound estimate of the tensile stresses in the geosynthetics.

A conservative approach would assume the buttress carries no axial load; therefore, the driving forces are resisted only by a combination of tensile load in the geosynthetics and shear between the underlying geosynthetics. Analysis is conducted by applying the total driving force along the soil/geotextile interface and using the large displacement strengths at all other interfaces. Results are plotted on the curve labeled "No Soil Buttress" in Fig. 6. For slope grades less than 16 percent (9.0°), the geosynthetics carry no load since the interface friction angle for the geonet/geomembrane is 9°. The tensile load indicated by the "No Soil Buttress" curve in Fig. 6 exceeds the combined tensile strength of the geotextile and geonet for slope grades greater than about 20 percent (11.3°). This approach yields very high values for loads in the geosynthetics and is considered to provide an upper-bound estimate for geosynthetic load. Therefore, the results of limit analyses using the "No Soil Buttress" and the "Maximum Soil Buttress" approach provide upper bound and lower bound estimates of load expected in the geosynthetic components, respectively.

A limit approach that yields results between the two extremes assumes that the axial load in the soil and the shear resistance at the soil/geotextile interface are both at the same proportion of their limits. Therefore, at a slope grade just prior to failure of the soil cover, the axial load and the shear resistance at the soil/geotextile interface will both be at their respective limits. Large displacement resistances are assumed for all of the interfaces below the soil/geotextile interface. The curve labeled "Proportioned Soil Buttress" in Fig. 6 represents the results obtained for this case. The magnitude of the geosynthetic tensile load is between the extreme cases of "No Soil Buttress" and "Maximum Soil buttress." Axial loads for each component are shown in Fig. 7 as a percentage of their maximum strength. The axial load in the soil cover increases gradually as the slope angle increases. The geonet carries load for a slope grade exceeding 18 percent (10°). This identifies where

sliding initiates along the geonet/geomembrane interface. Similarly, the geotextile begins to carry load when the slope grade exceeds 22 percent (12.4°). Once slippage occurs along the geotextile/geonet interface, the geotextile load increases significantly with increasing slope. The combined tensile load of the geonet and geotextile is equal to the geosynthetic load shown in Fig. 6 as "Proportioned Soil Buttress." The geomembrane carries no tensile load.

Proposed Method - GEOSTRES Results from limit analyses (Figs. 6 and 7) demonstrate that a wide range of loads in the soil and geosynthetic components can be predicted. Unfortunately, the limit methods provide no guidance to assess which assumptions result in reasonable predictions for load. To evaluate the consequence of these assumptions, a consideration of shear resistance-displacement and axial stress-strain compatibility is necessary. The example problem has been analyzed using GEOSTRES by dividing each layer into 100 nodes. Results are shown on Figs. 6 and 8. The total load carried by the geosynthetics (Fig. 6) falls between the limit state assumptions of "Proportioned Soil Buttress" and "Maximum Soil Buttress," indicating that to rely on the soil to provide the full resistance is unconservative, and to assume no buttressing at the toe of the soil cover is overly conservative. For slope grades less than about 21 percent (i.e. a slope angle of 12° which is the minimum peak friction angle at any interface), the geosynthetics carry small loads; however, for slope grades greater than 21 percent, the load carried by the geosynthetics increases rapidly over a small increase in the slope grade. The sharp jump in the axial load carried by the geosynthetics results from the post peak behavior of the interface resistance between the geonet and the geomembrane (Fig. 4). Once peak resistance is mobilized, additional deformation in the system causes the shear resistance to decrease to its large displacement value and the load in the geosynthetic components correspondingly increases. After the initial jump, the tensile load increases gradually as the slope grade steepens.

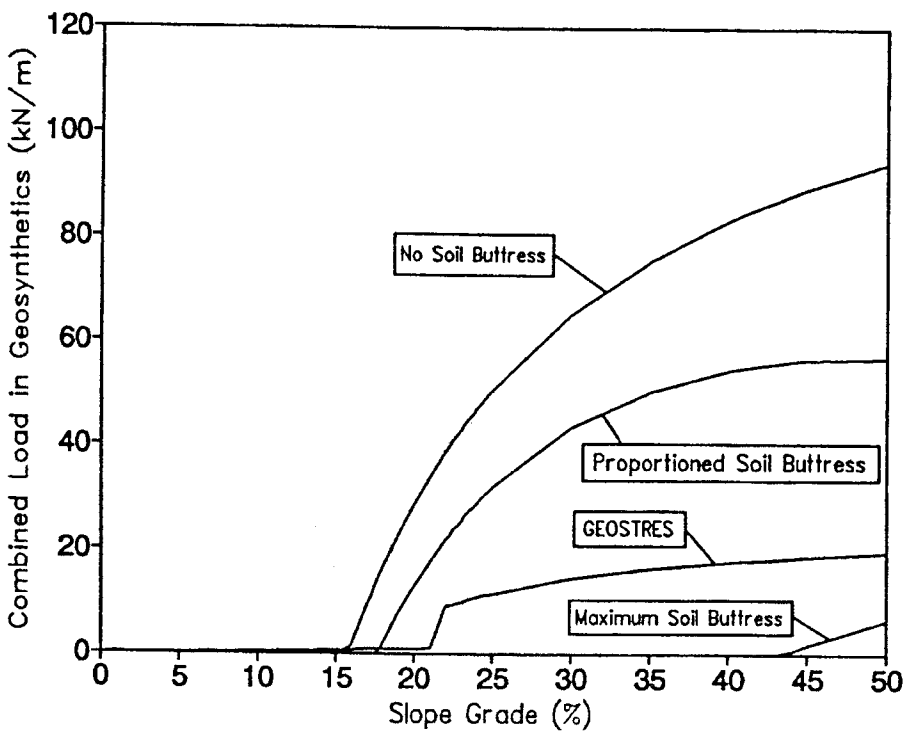

Figure 6. Geosynthetic load versus slope grade for example problem.

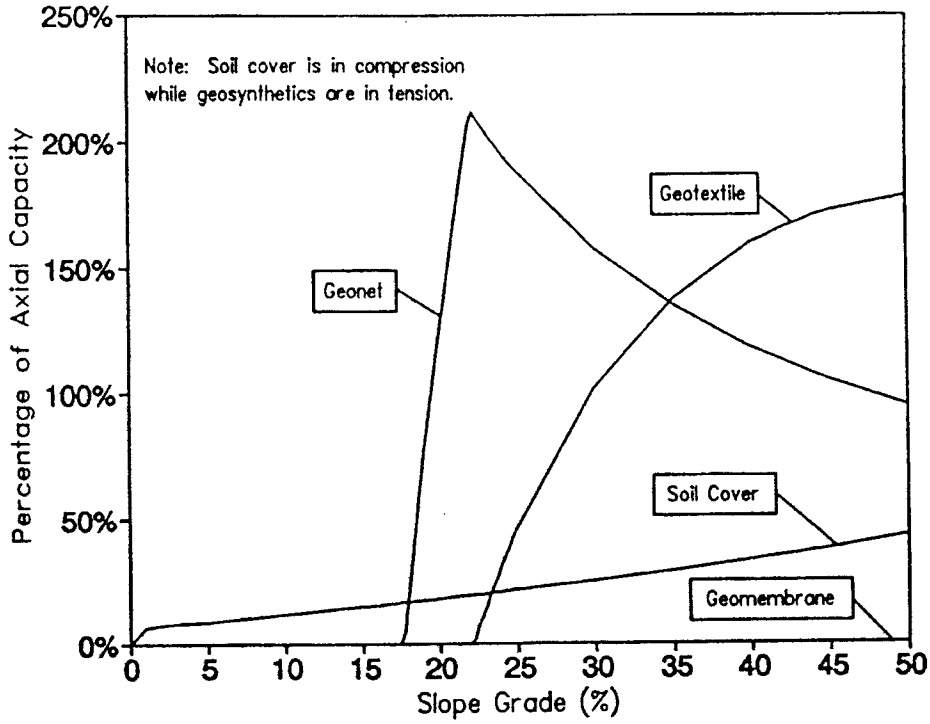

Figure 7. Predicted axial loads example problem using limit state analysis with proportioned soil buttress.

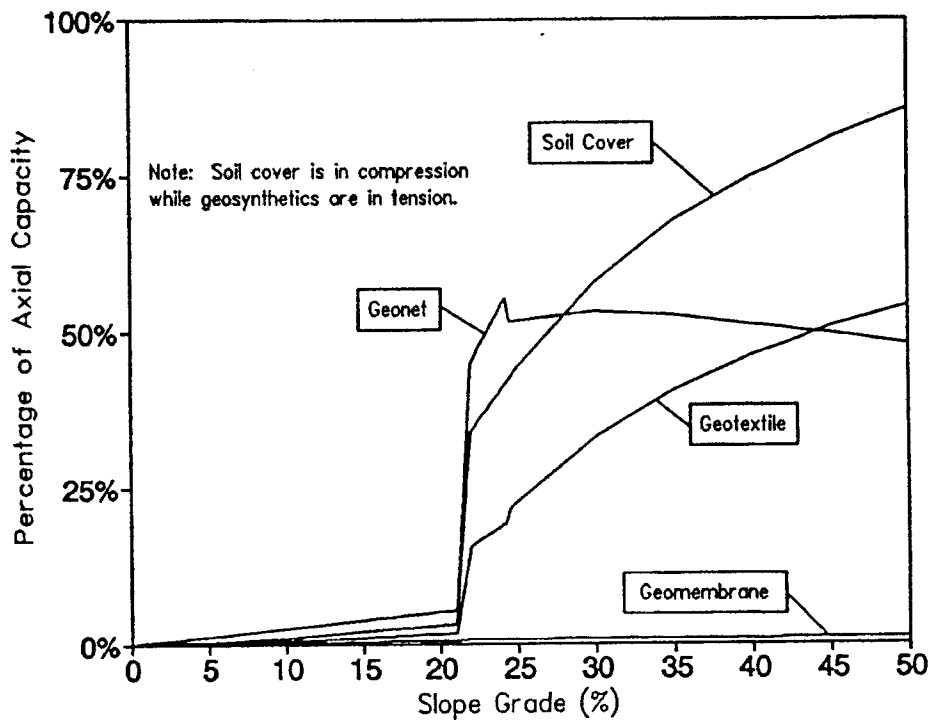

Figure 8. Predicted axial loads for example problem using GEOSTRES.

Predictions for loads in the cover system components using GEOSTRES are shown on Fig. 8. A comparison with Fig. 7 demonstrates that very different loads result from methods that consider equilibrium only. The loads in the geonet and the geotextile predicted by GEOSTRES are generally much smaller than those predicted using the limit approach due to greater resistance provided by the cover soil. Additionally, insight into the complex behavior of the system due to interactions between components can be obtained from Figure 8. For example, the tensile load in the geonet climbs sharply after initial slippage along its lower interface with the geomembrane [i.e. at a slope grade of 21 percent or 12° which is the peak friction angle of the geonet/geomembrane interface (Fig. 4)] and peaks at approximately 25 percent where slippage initiates along its upper interface with the geotextile. Here, the resistance at the geotextile/geonet interface decreases rapidly to its large displacement strength and less load can be transferred to the underlying geonet; thus, the load in the geonet decreases. Also, the load in the geotextile increases in response to the decrease in interface resistance at its base. As the slope grade increases further, the axial loads in the soil cover and geotextile continue to increase to resist the larger driving forces while the axial load in the geonet decreases slightly. Finally, it is interesting to note that some loads are predicted in the components even at slope angles less than 12° (i.e. the peak friction angle for the interface providing the minimum resistance). A limit approach would predict no loads at slope angles lower than 12°; however, the finite difference method predicts mobilization of axial loads due to deformations within the system to accommodate even small driving forces. This example demonstrates that it is important to have a complete understanding of how components interact within the system to design the system effectively, especially considering that the most complex behavior occurs at slope grades commonly used in practice (i.e. 10 to 50 percent).

The influence of strength and deformation parameters on component stress levels is demonstrated using results from a limited sensitivity study for the illustrative example. GEOSTRES was used to conduct the study. The following parameters were considered: the initial axial stiffness of the cover soil, the peak shear displacement at the geomembrane/clay interface, and the available shear resistance at the geomembrane/clay interface. A slope grade of 25 percent was selected for the study, and the results are presented in Table 2.

Since the cover soil carries a significant portion of the total resistance as a compressive load, the axial stiffness of the soil may have a significant influence on the results. The initial soil stiffness was increased and decreased by a factor of 2, and the results are presented as Cases 1 and 2, respectively, in Table 2. Increasing the soil stiffness by a factor of 2 results in greater load carried by the soil and less carried by the geosynthetic components, while the opposite result occurs when the soil stiffness decreases by a factor of 2. However, the magnitude of load change is less than a factor of 2. For example, increasing soil stiffness by 2 results in approximately 25 percent less load in both the geotextile and geonet. Decreasing the soil stiffness results in 29 percent greater load in both components. Therefore, the axial stiffness of a single component within the system can substantially influence the loads developed within that component as well as within other components in the system.

Importance of the interface shear-displacement behavior is illustrated below. The shear resistance versus displacement relationships for interfaces with geosynthetic components are typically obtained in the laboratory; however, these relationships are sensitive to test conditions, test methods, and scale of the test. The peak shear displacement for the non-textured geomembrane/clay interface was increased by a factor of 10 (i.e. the shear displacement at peak was increased from 0.5 mm to 5.0 mm) and is

noted as Case 3 in Table 2. The axial load in the geomembrane increases from 0.13 kN/m to 0.73 kN/m due to the increase in the peak displacement while the axial loads in the other components are essentially unaffected. Since larger displacement is required to mobilize peak shear resistance at the base of the geomembrane, the geomembrane carries more load axially as axial strains are developed. However, the increase in axial load within the geomembrane is insignificant.

Table 2. Sensitivity study results for example problem with 25 percent slope grade.

	Cover Soil		Geotextile		Geonet		Geomembrane	
	Axial Load (kN/m)	Ratio with Base	Axial Load (kN/m)	Ratio with Base	Axial Load (kN/m)	Ratio with Base	Axial Load (kN/m)	Ratio with Base
Base Case	-39	1.00	6.0	1.00	5.4	1.00	0.13	1.00
Case 1	-42	1.09	4.5	0.75	4.1	0.76	0.13	1.00
Case 2	-36	0.92	7.8	1.29	7.0	1.29	0.13	1.00
Case 3	-39	1.00	6.0	1.00	5.4	1.00	0.73	5.5
Case 4	-42	1.08	6.1	1.01	5.5	1.01	6.7	51

Notes:

Case 1 - Increase axial stiffness of cover soil by 2.
Case 2 - Decrease axial stiffness of cover soil by 2.
Case 3 - Increase peak shear displacement at geomembrane/clay interface by 10.
Case 4 - Replace compacted clay with bentonite panels.
Tension is indicated by positive values.
Compression is indicated by negative values.

Replacement of the compacted clay with bentonite panels is considered as Case 4. A typical shear resistance versus displacement relationship for the bentonite panel interface with a non-textured geomembrane is given as follows: a peak friction angle of 9° mobilized at 5 mm and a residual friction angle of 8° mobilized at 13 mm. The resulting geosynthetic loads for the 25 percent slope grade with the bentonite panels are presented as Case 4 in Table 2. The axial load in the geomembrane increases to 6.7 kN/m due to the smaller available shear resistance at the base of the geomembrane, while the axial loads in the other components remain essentially unchanged. Therefore, the available shear resistance at the interfaces has a significant influence on the axial loads developed within the geosynthetics.

CONCLUSIONS

A finite difference method for evaluating stresses in soil and geosynthetic components of lining and cover systems is developed in this paper. The approach satisfies force equilibrium and load-deformation compatibility at interfaces and within components. A typical cover system is analyzed to compare the results of the proposed approach with results of the currently accepted limit state approach. While limit state methods can identify upper and lower bounds for component loads, the difference between the two bounds is too great to assess a realistic load in the geosynthetics. Compared to the approach proposed, the limit state approach can significantly over-

predict, or under-predict loads in components. Furthermore, the limit state approach predicts tensile load in a geosynthetic layer only when slippage occurs on its lower interface. The proposed approach also shows that a geosynthetic can develop tensile load from slippage on its lower interface, but in addition, can develop load when slippage occurs along other underlying interfaces.

The importance of strength and deformation parameters are illustrated with results of a limited sensitivity study. The results demonstrate that changes in strength and deformation parameters of individual components can be significant, or insignificant, depending on the type of parameter varied, and how the other components in the system interact. Methods that consider only limit state equilibrium predict no effect of deformation parameters.

Results from analyses of an example cover system and a sensitivity study demonstrate that stresses within lining and cover systems are affected significantly by both stiffness and strength of the soil and geosynthetic layers. Therefore, both shear resistance-displacement and axial stress-strain compatibility must be satisfied to evaluate stresses accurately within geosynthetic components of lining or cover systems.

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Results of unpublished laboratory tests, such as large-scale direct shear tests, on soil/geosynthetic and geosynthetic/geosynthetic interfaces, and on bentonite panels, were provided by Golder Associates Inc. Their contribution and cooperation is greatly appreciated.

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Analysis of a Large Database of GCL Internal Shear Strength Results

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Abstract: A database of 414 large-scale direct shear test results was assembled to evaluate variables governing geosynthetic clay liner (GCL) internal shear strength. The tests were conducted by a single independent laboratory over 12 years using procedures consistent with current testing standards. A wide range of GCL types, normal stresses, and shear displacement rates allowed investigation of the effect of reinforcement, pore water pressure generation, and sources of shear strength variability. Reinforced GCLs showed higher strength than unreinforced GCLs, with needle-punched GCLs performing better than stitch-bonded GCLs. Thermal locking of needle-punched GCLs was found to be effective at high normal stress, but hydration using low hydration normal stress was found to decrease the effectiveness of thermal locking. Shear-induced pore water pressures were indirectly evaluated using shear strength results from tests conducted using normal stresses above and below that corresponding to the GCL swell pressure. The peak shear strength was found to increase with decreasing shear displacement rates for high normal stresses, while the opposite trend was observed for low normal stresses. Shear strength envelopes showed a bilinear response, with a break at normal stresses consistent with the GCL swell pressure. Good repeatability of test results was obtained using the same-manufacturing-lot GCL specimens, while comparatively high variability was obtained using different-lot specimens. Peak shear strength variability was found to increase linearly with normal stress, but to be insensitive to specimen conditioning procedures. Evaluation of reinforced and unreinforced GCL test results indicates that, in addition to reinforcement variability, bentonite variability contributes to the shear strength variability of reinforced GCLs. Peel strength was found not to be a good indicator of the contribution of fibers to the GCL peak shear strength.

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CE Database subject headings: Databases; Shear strength; Data analysis; Shear tests; Geosynthetic; Clay liners.

Introduction

Geosynthetic clay liners (GCLs) are prefabricated geocomposite materials used in hydraulic barriers as an alternative to compacted clay liners. They consist of sodium bentonite clay bonded to one or two layers of geosynthetic backing materials (carrier geosynthetics). Advantages of GCLs include their limited thickness, good compliance with differential settlements of underlying soil or waste, easy installation, and low cost. Stability is a major concern for side slopes in bottom liner or cover systems that include GCLs because of the very low shear strength of hydrated sodium bentonite (Mesri and Olson 1970). Proper shear strength characterization is needed for the different materials and interfaces in hydraulic barriers. In particular, the failure surface of a liner system may develop internally (within the GCL), either through its bentonite core or along the bentonite/carrier geosynthetic interface. The internal shear strength of GCLs is the focus of the study presented in this paper.

Several investigators have evaluated the GCL internal shear strength using direct shear and ring shear tests (Gilbert et al. 1996, 1997; Stark et al. 1996; Eid and Stark 1997; Fox et al. 1998; Eid et al. 1999). These experimental studies have provided invaluable insight into the significance of parameters that govern the shear behavior of GCLs. However, available information on GCL internal shear strength is still limited to specific ranges of normal stresses, GCL types, and test conditions. There are three primary reasons why a comprehensive evaluation of GCL internal shear strength is still needed. First, the use of tests from different laboratories may have masked sources of variability, as was the case in a shear strength database assembled by Stoewahse et al. (2002) using results from European laboratories. Second, the current standard for internal and interface GCL shear strength testing (ASTM D6243) has only been available since 1998 (ASTM 1998), so tests conducted before the approval of this standard may have not been consistent with current procedures. Third, significant costs (large-scale direct shear devices, long time for conditioning and testing) have limited the number of available test results and precluded evaluations of variability.

A database of 414 large-scale direct shear tests conducted by a single laboratory was assembled and evaluated in this study to identify and quantify the variables governing the internal shear strength of GCLs. This database, referred to as the GCL shear strength (GCLSS) database, is used to define upper and lower bounds on peak and large-displacement GCL internal shear strength. In addition, an analysis of the results in the GCLSS database allows evaluation of: (1) The performance of GCLs

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Table 1. Summary of GCLs in the GCLSS Database

GCL label	GCL product	Description[a]	No. of tests reaching τ_p	No. of tests reaching τ_{ld}
A	Bentomat ST	Needle-punched W-NW	270	203
B	Claymax 500SP	Stitch-bonded W-W	48	5
C	Bentofix NS	Thermal-locked, needle-punched W-NW	26	26
D	Bentofix NW	Thermal-locked, needle-punched NW-NW	16	13
E	Bentofix NWL	GCL *D* with lower mass of sodium bentonite per unit area	8	8
F	Claymax 200R	Unreinforced W-W	13	13
G	Not Marketed	GCL *A* with additives to the sodium bentonite	3	0
H	Bentomat DN	Needle-punched NW-NW	18	6
I	Not Marketed	GCL *A* with adhesive strengthened reinforcements	8	0
J	Geobent	Needle-punched W-NW	4	4

[a]W=Woven carrier geotextile, NW=Nonwoven carrier geotexile.

manufactured using different types of reinforcement, (2) pore water pressures during shearing (indirect evaluation), and (3) the GCL internal shear strength variability.

Database

Data Source

The large-scale direct shear tests in the GCLSS database were performed between 1992 and 2003 by the Soil-Geosynthetic Interaction laboratory of GeoSyntec Consultants, currently operated by SGI Testing Services (SGI). SGI is an accredited testing facility with significant consistency in its testing procedures. It should be noted that procedures used for GCL direct shear tests conducted by SGI over the period 1992 to 2003 are consistent with ASTM D6243 (ASTM 1998), even though this standard was only approved in 1998. Most tests in the GCLSS database were conducted for commercial purposes and, consequently, the testing characteristics and scope was defined by project-specific requirements. A few additional tests were conducted specifically for this investigation in order to complement tests conducted using different shear displacement rates and to incorporate peel strength results in variability analyses. Test conditions reported for each series in the GCLSS database include specimen preparation and conditioning procedures, hydration time (t_h), consolidation time (t_c), normal stress during hydration (σ_h), normal stress during shearing (σ_n), and shear displacement rate (SDR).

Materials

Direct shear tests in the GCLSS database were conducted using ten commercial GCL products (nine reinforced, one unreinforced). Table 1 provides the designation of the GCLs used in this study (GCL *A* to *J*), the product name, and a short description of the reinforcement characteristics and carrier geotextiles. An important objective of this study is the comparison of shear strength results among different types of GCLs. Unreinforced GCLs are used in applications where high shear strength is not required, while reinforced GCLs (e.g., stitch-bonded needle-punched GCLs) are used otherwise. The unreinforced GCL investigated in this study (GCL *F*) consists of an adhesive-bonded bentonite layer held between two woven polypropylene geotextiles. The stitch-bonded GCL investigated in this study (GCL *B*) consists of a bentonite layer stitched using synthetic yarns between two woven polypropylene carrier geotextiles. The needle-punched GCLs investigated in this study (GCLs *A*, *C*, *D*, *E*, *G*, *H*, *I*, and *J*) consist of a bentonite layer between two (woven or nonwoven) carrier geotextiles that is reinforced by pulling fibers through using a needling board. The fiber reinforcements are typically left entangled on the surface of the top carrier geotextile. Since pullout of the needle-punched fibers from the top carrier geotextile may occur during shearing (Gilbert et al. 1996), some needle-punched GCL products (GCLs *C*, *D*, and *E*) were thermal locked to minimize fiber pullout. Thermal locking involves heating the GCL surface to induce bonding between individual reinforcing fibers as well as between the fibers and the carrier geotextiles (Lake and Rowe 2000). For simplicity, thermal-locked needle-punched GCLs will be referred to simply as thermal-locked GCLs in this paper.

Testing Equipment and Procedures

The large-scale direct shear tests conducted in this study used large direct shear devices each containing a top and bottom shear box. Typically, the top shear box measured 305 mm by 305 mm in plan and 75 mm in depth. The bottom shear box measured 305 mm by 355 mm in plan and 75 mm in depth. For the GCL internal direct shear tests, the bottom shear box was sectioned down to plan dimensions of 305 mm by 305 mm. A constant SDR was applied to the bottom shear box using a mechanical screw drive system and the resultant shear load was measured on the top shear box using a load cell. The direct shear devices used in this study were capable of applying normal stresses from 2.4 to 3,000 kPa during shearing. Dead weights were placed above the GCL in tests conducted under low normal stresses, while an air bladder or a hydraulic cylinder were used to exert a normal force between the GCL and a reaction frame in tests conducted under relatively high normal stress. A load cell was used to measure the normal load. The accuracy of the normal stress application device and calibration of the load cells were verified at least every year as a part of a laboratory accreditation program.

A detail of the specimen configuration for GCL internal shear strength testing is shown in Fig. 1(a). A water bath may be used for testing GCLs under submerged conditions, although most tests in the GCLSS database were conducted without a water bath. For each test, a fresh GCL specimen was trimmed from the bulk GCL sample. The internal strength testing of the GCL specimen involved constraining the GCL specimen so that shearing could only occur within the bentonite component of the GCL. The specimen was constrained by bonding the two carrier geotextiles to porous rigid substrates using textured steel gripping surfaces. Extensions of each carrier geotextile were secured using a second

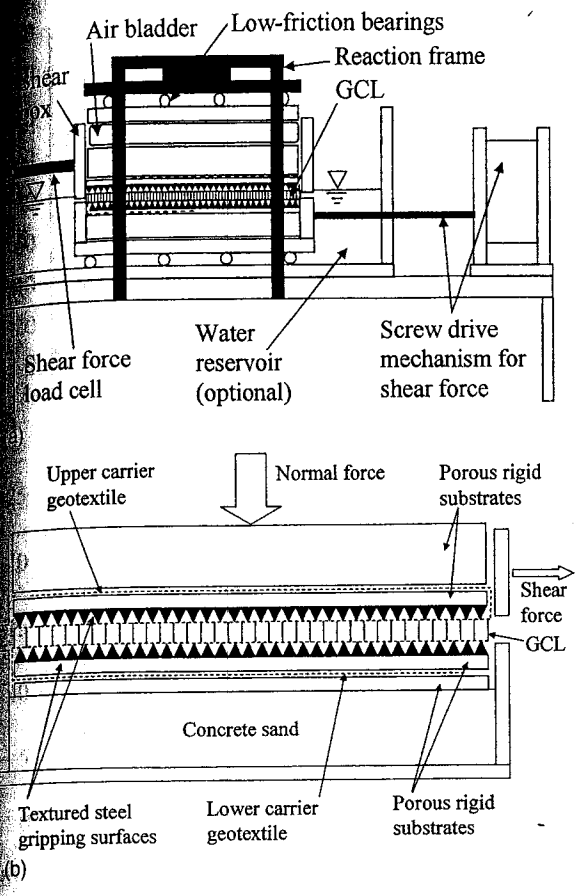

Fig. 1. Direct shear device: (a) Load application configuration; and (b) specimen detail

porous rigid substrate as shown in Fig. 1(b). The textured steel gripping surfaces were employed to minimize slippage between each carrier geotextile and the porous rigid substrate. Post-test examination of the sheared GCLs indicated that slippage did not occur between the GCL and the grips, suggesting a uniform shear stress transfer onto the GCL specimens.

Conditioning of specimens plays an important role in GCL internal shear strength testing as moisture interactions should simulate correctly those anticipated in the field. GCL conditioning involves hydration and (in some cases) subsequent consolidation of the sodium bentonite. Pore water pressures in the sodium bentonite of the GCLs tested in this study are negative for typical initial (as received) moisture conditions. Hydration of the sodium bentonite leads to reduction of the negative pore water pressures and vertical swelling. Changes in pore pressures and vertical deformations were not measured during GCL conditioning or shearing. Although this is consistent with the current state of the practice and ASTM (1998), measurements of vertical deformation during specimen conditioning and shearing would have allowed assessment of bentonite hydration by using conventional methods to estimate the degree of consolidation (Gilbert et al. 1997). Consequently, hydration of the bentonite was only assessed in this study by the reported hydration time. Although hydration times as high as 250 hs may be required to reach full hydration, hydration times beyond 72 hs have been reported not to significantly increase the GCL water content, especially under high σ_n (Stark and 1996). The hydration process used in this study involved typically a two-stage procedure similar to that reported by Fox et al. (1998). The specimen and rigid substrates were placed under a specified σ_h outside the direct shear device and soaked in tap water during the specified t_h. This assembly was then transferred to the direct shear device. σ_h was often specified to equal the shearing normal stress (σ_n). However, if σ_h was less than σ_n (e.g., to simulate field conditions representative of bottom liners), the normal stress was slowly ramped up to σ_n, and pore pressures were allowed to dissipate during a consolidation period (t_c).

Shearing was conducted after GCL conditioning by applying the shear load under a constant SDR. The shear force was recorded for increasing shear displacement. The maximum shear stress was identified as the peak shear strength (τ_p), and the shear stress at the end of testing was identified as the large-displacement shear strength (τ_{ld}). Table 1 shows the number of tests used to define τ_p and τ_{ld} of each GCL. τ_{ld} was reported only when the post-peak shear stress reached an approximately constant value within the maximum displacement of the test device (75 mm). In some cases, shearing was discontinued after reaching the peak value because the test, conducted for commercial purposes, did not require post-peak assessment. In other cases, a peak shear strength value was reached, but partial separation of the reinforcements from the carrier geotextiles after reaching the peak led to an unrealistically high τ_{ld}, especially at low normal stress. As will be discussed below, the particular mode of shear failure of stitch-bonded GCL *B* generally did not allow shearing beyond the peak value.

SDR in the field is anticipated to occur slowly, which is consistent with drained conditions (Gilbert et al. 1997). The SDR used for most tests in the GCLSS database is 1.0 mm/min. While relatively fast for guaranteeing drained conditions, a SDR of 1.0 mm/min is typically used in engineering practice because of time and cost considerations. Additional tests were sheared using slower rates (as low as 0.0015 mm/min). Shearing was typically terminated when a displacement of 75 mm, or an approximately constant τ_{ld} value, was reached. Consistent with observations reported by Gilbert et al. (1996) and Fox et al. (1998), dismantling of the needle-punched thermal-bonded and unreinforced GCL specimens indicated that failure occurred typically through the interface between the bentonite and the carrier geotextile. The carrier geotextiles were always found to contain extruded bentonite. In the stitch-bonded GCL *B* specimens, the continuous fibers stretched during initial shearing. However, once the continuous fibers became fully stretched, continued shear displacement often led to rupture of the fibers or tearing of the carrier geotextiles at the threaded connections. Despite the particular arrangement of fiber reinforcements in stitch-bonded GCLs, observation of the specimens after testing did not show slippage of the woven geotextiles at the interface with the gripping system.

Analysis of Results from Different GCL Materials

A total of 32 failure envelopes (FEs) were defined considering the different GCL types and test conditions used in this investigation. A total of 385 of the 414 test results were used, while 29 test results did not have similar conditioning procedures to any of the 32 defined failure envelopes. Table 2 summarizes the test conditions, the approximate range of σ_n, and the friction angle and cohesion intercept defining the τ_p and τ_{ld} envelopes. In some cases, the internal shear strength was also characterized using a bilinear FE. The square root of the mean-squared error of the linear regression, which is considered the standard deviation of the linear regression (Helsel and Hirsh 1991), was calculated as a measure of the spread of data around the best-fit lines:

Table 2. Summary of Failure Envelopes in the Geosynthetic Clay Liner Shear Strength (GCLSS) Database

			Test conditions					Peak				Large-displacement[b]			
Failure envelope[a]	GCL label	Number of tests	SDR (mm/min)	σ_h (kPa)[c]	t_h (hs)	t_c (hs)	σ_n range (kPa)	c_p (kPa)	ϕ_p (Degrees)	R^2	s (kPa)	c_{ld} (kPa)	ϕ_{ld} (Degrees)	R^2	s (kPa)
FE 1	*A*	27	1.0	σ_n	24	0	3.4–72	13.5	46.6	0.987	3.11	2.1	8.6	0.842	1.25
FE 2	*A*	2	1.0	4.8	24	0	14–24	10.7	37.1	1.000	N/A	3.3	4.0	1.000	N/A
FE 3	*A*	12	0.5	σ_n	24	0	48–386	42.8	24.6	0.975	11.00	9.4	9.8	0.968	4.78
FE 4	*A*	40	1.0	σ_n	48	0	2.4–2759	42.4	14.0	0.966	25.36	16.2	6.3	0.983	12.49
FE 4 (Low σ_n)	*A*	31	1.0	σ_n	48	0	2.4–97	14.4	35.4	0.948	13.04	N/A	N/A	N/A	N/A
FE 4 (High σ_n)	*A*	9	1.0	σ_n	48	0	97–2759	102.4	11.9	0.987	52.78	N/A	N/A	N/A	N/A
FE 5	*A*	5	1.0	4.8	48	0	14–276	35.9	29.9	0.991	6.79	2.0	4.4	0.996	N/A
FE 6	*A*	8	1.0	σ_n	72	0	2.4–103	17.4	34.7	0.840	10.80	2.8	8.5	0.943	1.93
FE 7	*A*	141	0.1	20.7	168	48	35–310	20.6	25.2	0.999	23.88	15.5	9.4	0.999	10.65
FE 8a	*A*	1	0.0015	8.0	144	1,476	248								
FE 8b	*A*	1	0.0015	63.0	48	540	520	74.3	21.9	0.988	23.38	35.0	5.8	0.991	5.22
FE 8c	*A*	1	0.0015	8.0	144	2,328	993								
FE 9	*A*	3	1.0	68.9	24	12	138–552	37.9	22.7	0.998	5.53	2.8	11.2	0.918	17.69
FE 10	*A*	3	1.0	6.9	60	24	4.8–29	12.4	50.1	0.991	1.98	N/A	N/A	N/A	N/A
FE 11	*A*	7	1.0	0.0	0	0	2.4–35	12.9	60.1	0.921	4.64	N/A	N/A	N/A	N/A
FE 12	*B*	7	1.0	σ_n	24	0	24–690	53.4	7.3	0.818	16.93	N/A	N/A	N/A	N/A
FE 13	*B*	25	1.0	4.8	48	0	2.4–982	24.3	4.4	0.949	3.87	N/A	N/A	N/A	N/A
FE 14	*B*	10	1.0	7.2	96	0	10–1000	24.1	4.6	0.976	5.21	N/A	N/A	N/A	N/A
FE 15	*B*	3	0.1	20.7	168	48	35–310	32.4	7.3	0.994	1.95	N/A	N/A	N/A	N/A
FE 16	*C*	13	0.5	σ_n	24	0	7.2–575	23.3	23.8	0.959	13.08	12.3	9.8	0.951	11.12
FE 16 (Low σ_n)	*C*	6	0.5	σ_n	24	0	7.2–103	17.2	28.3	0.999	12.07	N/A	N/A	N/A	N/A
FE 16 (High σ_n)	*C*	7	0.5	σ_n	24	0	103–575	9.7	14.9	0.950	14.41	N/A	N/A	N/A	N/A
FE 17	*C*	10	0.2	55.2	24	0	10–290	22.0	29.3	0.993	5.17	8.0	12.0	0.975	3.86
FE 18	*C*	3	0.1	20.7	168	48	35–310	22.3	16.6	1.000	0.21	0.9	8.3	0.974	4.67
FE 19	*D*	6	1.0	σ_n	72	0	6.9–552	5.7	18.6	1.000	5.28	0.1	8.4	0.985	5.21
FE 20	*D*	3	0.5	σ_n	24	0	98–380	75.3	25.1	0.997	0.20	21.3	9.6	0.982	5.43
FE 21	*D*	6	0.1	3.4	24	24	6.9–690	40.9	27.1	0.972	27.40	15.5	8.0	1.000	0.18
FE 21 (Low σ_n)	*D*	3	0.1	3.4	24	24	6.9–28	22.4	38.9	0.972	2.03	N/A	N/A	N/A	N/A
FE 21 (High σ_n)	*D*	3	0.1	3.4	24	24	172–690	101.0	21.6	1.000	2.03	N/A	N/A	N/A	N/A
FE 22	*E*	4	1.0	σ_n	336	0	14–58	32.7	31.8	0.993	1.20	7.3	11.3	0.994	0.37
FE 23	*E*	4	1.0	σ_n	48	0	14–58	30.6	38.9	0.993	1.57	6.8	13.7	0.993	0.46
FE 24	*F*	3	1.0	σ_n	168	0	14–55	1.7	12.3	0.999	0.18	2.1	8.5	1.000	0.00
FE 25	*F*	3	1.0	0.0	0	0	69–483	16.1	3.7	1.000	0.28	10.1	4.0	1.000	1.13
FE 26	*G*	4	1.0	σ_n	24	0	2.4–19	4.8	30.4	1.000	0.11	N/A	N/A	N/A	N/A
FE 27	*H*	6	1.0	σ_n	24	0	4.8–483	19.7	33.8	0.997	8.29	23.8	5.3	0.997	1.56
FE 27 (Low σ_n)	*H*	4	1.0	σ_n	24	0	4.8–48	5.3	47.0	0.998	1.60	N/A	N/A	N/A	N/A
FE 27 (High σ_n)	*H*	2	1.0	σ_n	24	0	241–483	8.5	31.7	1.000	N/A	N/A	N/A	N/A	N/A
FE 28	*H*	6	1.0	3.4	24	24	6.9–690	33.0	32.1	0.988	21.12	29.9	8.5	0.996	3.46
FE 28 (Low σ_n)	*H*	3	1.0	3.4	24	24	6.9–28	16.5	45.0	0.971	2.58	N/A	N/A	N/A	N/A
FE 28 (High σ_n)	*H*	3	1.0	3.4	24	24	172–690	78.9	28.4	1.000	3.13	N/A	N/A	N/A	N/A
FE 29	*H*	6	0.25	0.0	96	24	4.8–10	12.1	46.3	1.000	1.63	N/A	N/A	N/A	N/A
FE 30	*I*	4	1.0	0.0	0	0	2.4–24	19.3	58.2	0.988	5.01	N/A	N/A	N/A	N/A
FE 31	*I*	4	1.0	2.4	72	0	2.4–24	21.9	51.1	0.932	1.56	N/A	N/A	N/A	N/A
FE 32	*J*	4	1.0	σ_n	24	0	24–193	5.5	9.1	1.000	0.31	0.4	6.9	0.982	1.51

[a]FE 4, 16, 21, 27, and 28 represented using both linear and bilinear envelopes.

[b]N/A=Not applicable.

[c]$\sigma_h=\sigma_n$ means that the normal stress used during hydration is the same as the normal stress used during shearing.

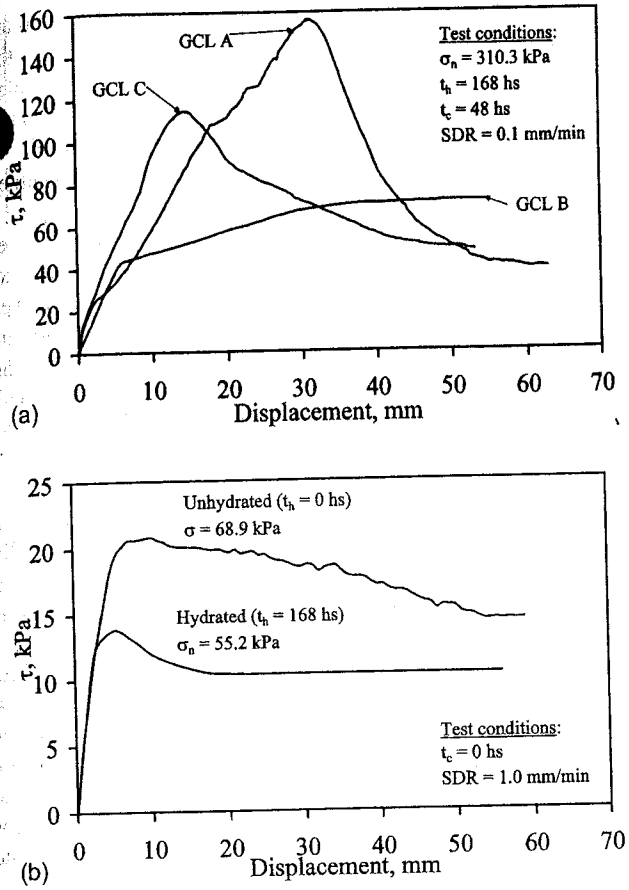

Fig. 2. Shear stress-displacement curves for different GCLs: (a) GCLs *A* (needle punched), *B* (stitch bonded), and *C* (thermally locked); and (b) GCL *F* (unreinforced)

$$s = \sqrt{\frac{\sum_{i=1}^{n} e_i^2}{n-2}} \tag{1}$$

where s=standard deviation of the linear regression; e_i=difference between the shear strength value and the value on the best-fit line at the same normal stress; and n=number of data points in the regression. Since the data summarized in Table 2 follow approximately a normal distribution around the FEs, a bound of one standard deviation contains 84% of the likely shear strength values (Helsel and Hirsh 1991).

The effect on the GCL internal shear strength of the type of internal reinforcements is investigated in this section in order to provide: (1) An evaluation of the shear stress-displacement behavior of the different GCL types, (2) a preliminary overview of GCL internal shear strength, and (3) a comparison of GCLs tested under similar conditioning procedures.

Shear Stress-Displacement Behavior

Fig. 2(a) shows shear stress-displacement curves for GCLs *A* (needle punched), *B* (stitch bonded), and *C* (thermal locked). The three GCL types were tested using the same σ_n (310.3 kPa), same t_h (168 h), same t_c (48 h), and same SDR (0.1 mm/min.). GCL *A* shows a well-defined τ_p and a marked post-peak shear strength loss. Unlike GCL *A*, GCL *B* shows a rapid initial mobilization of shear strength until reaching a "yield" stress level, beyond which a less pronounced hardening takes place until reaching τ_p. The

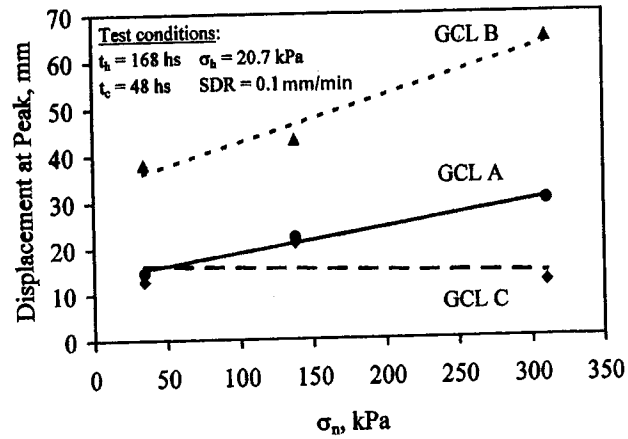

Fig. 3. Displacement at peak shear strength as a function of σ_n for GCLs *A*, *B*, and *C*

displacement at peak for GCL *B* is significantly larger than that observed for GCL *A*. The post-peak behavior of GCL *B* could not be evaluated since this GCL did not reach a steady large-displacement strength value at the maximum displacement of the device. Thermal-locked GCL *C* shows a behavior similar to that of needle-punched GCL *A*, although the τ_p value is below that obtained for GCL *A*. GCLs *A* and *C* were reinforced using similar needle-punching techniques and have the same specified peel strength (6.5 N/m). Consequently, differences in their behavior are attributed to the effect of thermal locking. Comparison of the response of the two GCLs, tested under identical conditions, suggests that thermal locking did not lead to the expected increase in shear strength.

Fig. 2(b) shows shear stress-displacement curves for GCL *F* (unreinforced) tested under hydrated and unhydrated conditions. Although a direct comparison of τ_p is not possible as the specimens were tested using different σ_n, the results indicate that the hydrated GCL has lower τ_p and τ_{ld} than the unhydrated GCL. Both specimens, however, show a significantly lower τ_p than that obtained for reinforced GCLs. The displacement at peak of unreinforced GCLs is consistent with displacement at the yield stress observed for GCL *B*. However, the displacement at peak of unreinforced GCLs is significantly lower than the one obtained for the reinforced GCLs. While both hydrated and unhydrated unreinforced GCLs show post-peak shear strength loss, the hydrated GCL appears to reach residual conditions at lower shear displacement than the unhydrated GCL.

Fig. 3 summarizes the displacement at peak for the three tests shown in Fig. 2(a) along with results from additional tests conducted under two additional σ_n values (34.5 and 137.9 kPa). GCLs *A* and *B* show increasing displacement at peak with increasing σ_n, while the displacement at peak for GCL *C* is apparently insensitive to σ_n. GCL *B* shows significantly larger displacement at peak than the other GCL types, which may be particularly relevant for displacement-based stability analyses (e.g., for seismic design). For example, if the design criterion requires a maximum shear displacement of 50 mm for a σ_n=310.3 kPa, the results in Fig. 2(a) indicate that τ_p would govern the design if GCL *B* is selected, but τ_{ld} would need to be considered if GCLs *A* or *C* are used.

Overall Internal Strength Assessment

Fig. 4(a) shows the τ_p data for all GCLs in the GCLSS database, illustrating the wide range of normal stresses at which the GCLs

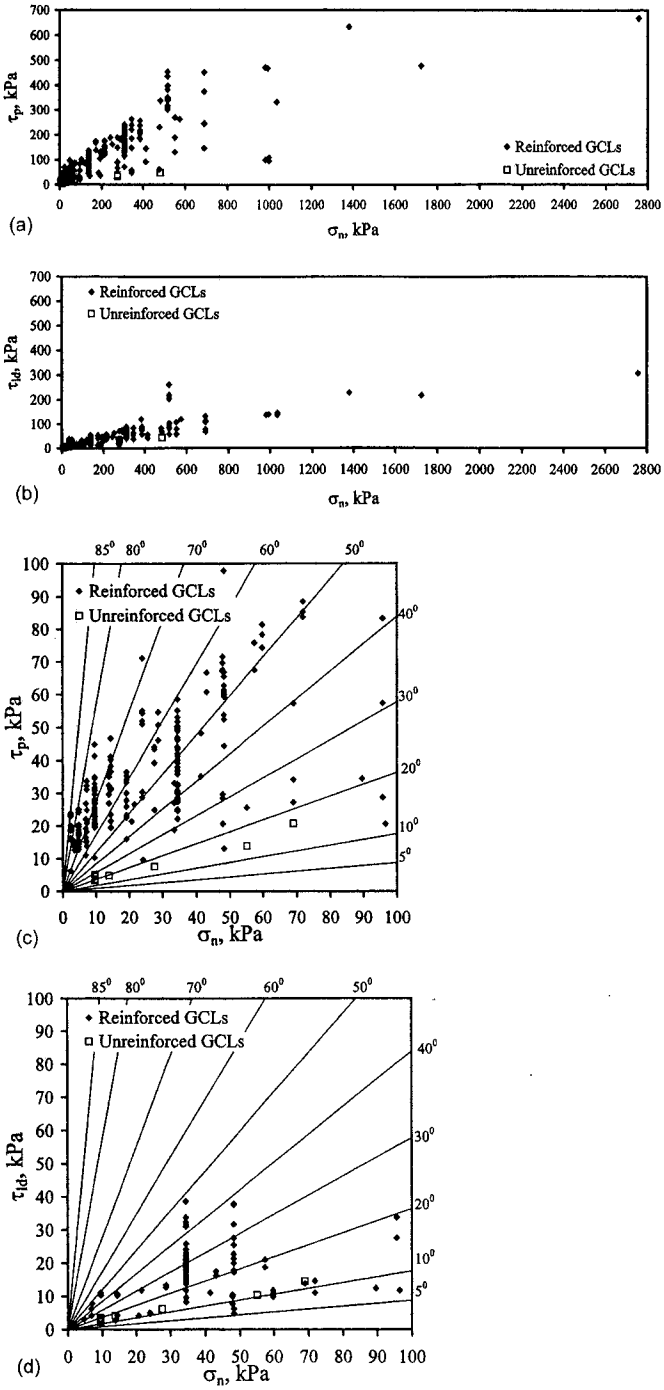

Fig. 4. Shear strength results for all geosynthetic clay liners: (a) peak shear strength values; (b) large-displacement shear strength values; (c) peak shear strength (scaled); and (d) large-displacement shear strength (scaled)

were tested and the significant scatter in the data. Similarly, Fig. 4(b) shows the τ_{ld} data for all GCLs in the GCLSS database, illustrating that the range of τ_{ld} values is significantly narrower than the range of τ_p values. As most data points shown in Figs. 4(a and b) correspond to comparatively low σ_n, Figs. 4(c and d) show a detail for σ_n values below 100 kPa. The results shown in Fig. 4(c) reflect the relevance of using a cohesion intercept to characterize τ_p at low σ_n. Inspection of the standard deviation s values in Table 2 indicates that the $s(\tau_p)$ for unreinforced GCLs (FE 24 and 25) is less than that for reinforced GCLs. Fig. 4(d) shows that the trend in τ_{ld} for low σ_n is consistent with the trend observed for higher σ_n. Inspection of the results in Figs. 4(b and d), as well as the information presented in Table 2 indicates that large-displacement shear strength is approximately independent of the GCL type. Reinforced GCLs tend to show a higher large-displacement shear strength value than the unreinforced GCLs, with stitch-bonded GCLs having the lowest large-displacement shear strength among all reinforced GCLs.

The test results for all GCLs were grouped into ten data sets based on reinforcement type. Table 3 summarizes the information for each data set, and provides the parameters for the shear strength envelopes (c, ϕ) of each data set. The GCL data sets are used only for preliminary database analysis, as they do not account for the effect of specimen conditioning on shear strength. Comparisons of τ_p values among the ten GCL data sets is aided by defining the shear strength values calculated using the GCL data set envelopes at given reference normal stresses. Table 3 includes the values of τ_{50} and τ_{300} for each data set, which are the average shear strength values at $\sigma_n = 50$ and 300 kPa, respectively. These reference normal stresses are representative of normal stress values for landfill cover and liner systems, respectively. In order to quantify the variability of the shear strength for each GCL data set, the range of shear strength values was defined for each reference normal stress. Specifically, the lowest and highest shear strength values were defined using the individual failure envelopes (FE in Table 2) of each data set. Additional information is provided by McCartney et al. (2002). Inspection of the τ_{50} and τ_{300} values shown in Table 3, leads to the following observations regarding the internal peak shear strength of GCLs under low and high normal stresses:

- The peak internal shear strength of all GCLs in the database (Set SS1) can be characterized by a cohesion intercept of 38.9 kPa and a friction angle of 18.0°. However, there is a significant scatter in the results both under comparatively low normal stresses (τ_{50} ranges from 13 to 71 kPa) and comparatively high normal stresses (τ_{300} ranges from 36 to 241 kPa). The most frequently tested GCL in the GCLSS database is GCL *A* (Set SS2, 270 tests), which has peak internal shear strength that can be characterized by a cohesion intercept of 46.6 kPa and a friction angle of 18.7°. Less scatter is observed in the shear strength of GCL *A* than that observed for all GCLs both under comparatively low normal stresses (τ_{50} ranges from 48 to 66 kPa) and high normal stresses (τ_{300} ranges from 117 to 195 kPa).
- As expected, the peak internal shear strength of reinforced GCLs (Set SS3) in consistently higher than that of unreinforced GCLs (Set SS4) both under low normal stresses [τ_{50}(Set SS3) = 57 kPa and τ_{50}(Set SS4) = 10 kPa] and high normal stresses [τ_{300}(Set SS3) = 139 kPa and τ_{300}(Set SS4) = 35 kPa].
- The peak internal shear strength of needle-punched GCLs (Set SS5) is consistently higher than that of stitch-bonded GCLs (Set SS6) both under low normal stresses [τ_{50}(Set SS5) = 58 kPa and τ_{50}(Set SS6) = 33 kPa] and high normal stresses [τ_{300}(Set SS5) = 149 kPa and τ_{300}(Set SS6) = 58 kPa]. The difference is less significant under low normal stresses because stitch-bonded GCLs show some cohesion (c_p = 28.5 kPa), but is more significant under high normal stresses due to the low friction angle (ϕ_p = 5.6°).
- The peak internal shear strength of needle-punched GCLs with woven-nonwoven (W-NW) carrier geotextile configurations (Set SS7) is similar to that of needle-punched GCLs with NW-NW carrier geotextiles (Set SS8) under low normal stresses [τ_{50}(Set SS7) = 58 kPa and τ_{50}(Set SS8) = 58 kPa].

Table 3. Geosynthetic Clay Liner (GCL) Data Sets for Overall Shear Strength Assessment

GCL data set	GCL set description[a]	GCL label	Peak envelope		Peak strength at σ_n=50 kPa	Peak strength at σ_n=300 kPa	Large-displacement envelope[b]	
			c_p (kpa)	ϕ_p (Degrees)	τ_{50}[range][b,c] (kpa)	τ_{300}[range][b,c] (kPa)	c_{ld} (kPa)	ϕ_{ld} (Degrees)
SS1	All GCLs	*A-J*	38.9	18.0	55[13(FE24) to 71(FE23)]	137[36(FE25) to 241(FE28)]	17.2	7.8
SS2	GCL *A*	*A*	46.6	18.7	63[48(FE7) to 66(FE1)]	148[117(FE4) to 195(FE8]	17.2	7.6
SS3	All reinforced GCLs	*A-E,G-J*	40.9	18.0	57[14(FE32) to 71(FE23)]	139[48(FE13) to 241(FE28)]	18.2	7.8
SS4	Unreinforced GCLs	F	5.0	5.7	10[13(FE24) to 13(FE24)]	35[36(FE25) to 36(FE25)]	3.5	5.3
SS5	Needle-punched GCLs	*A,C-E,G-J*	19.9	39.7	58[14(FE32) to 71(FE23)]	149[107(FE19) to 241(FE28)]	18.3	7.9
SS6	Stitch-bonded GCLs	*B*	28.5	5.6	33[28(FE14) to 60(FE12)]	58[48(FE13) to 92(FE12)]	N/A	N/A
SS7	W-NW needle-punched GCLs	*A,C,G,I,J*	19.1	40.9	58[14(FE32) to 66(FE1)]	145[111(FE18) to 195(FE8)]	19.1	7.8
SS8	NW-NW needle-punched GCLs	*D,E,H*	35.0	24.5	58[23(FE19) to 71(FE23)]	172[107(FE19) to 241(FE28)]	11.3	8.7
SS9	Needle-punched GCLs without thermal-locking	*A,G-J*	40.5	19.5	61[14(FE32) to 66(FE1)]	149[117(FE4) to 195(FE8)]	19.7	7.7
SS10	Needle-punched GCLs with thermal-locking	*C-E*	33.2	22.7	54[23(FE19) to 71(FE23)]	159[107(FE19) to 220(FE21)]	11.8	9.0

[a]GCL sets do not consider the effect of specimen conditioning or SDR.
[b]The range includes the lowest shear strength and corresponding FE as well as the highest shear strength and corresponding FE.
[c]Upper and lower FE envelopes at the reference normal stresses were defined using the parameters presented in Table 2.

However, needle-punched GCLs with W-NW carrier geotextiles showed a lower peak shear strength than those with NW-NW carrier geotextile configurations under high normal stresses [τ_{300}(Set SS7)=145 kPa and τ_{300}(Set SS8)=172 kPa].

- Needle-punched GCLs that were not thermal-locked (Set SS9) showed higher peak internal shear strength under low normal stresses than those that were thermal-locked (Set SS10) [τ_{50}(Set SS9)=58 kPa and τ_{50}(Set SS10)=54 kPa]. However, the opposite trend is observed under high normal stress [τ_{300}(Set SS9)=146 kPa and τ_{300}(Set SS10)=159 kPa]. This finding suggests that thermal locking of the fiber reinforcements is more effective under high normal stresses.

Unlike comparisons of τ_p values, comparisons of τ_{ld} values among the 10 data sets can be conducted by direct comparison of the large-displacement friction angles. This is because the cohesion intercept of large-displacement shear strength envelopes is negligible (less than 20 kPa). Inspection of ϕ_{ld} values shown in Table 3 leads to the following observations regarding the internal large-displacement shear strength of GCLs:

- The large-displacement shear strength of unreinforced GCLs is consistently lower than that of reinforced GCLs [ϕ_{ld}(Set SS4)=5.3° and ϕ_{ld}(Set SS3)=7.8°].
- The range of large-displacement shear strength for the reinforced GCLs data sets in Table 3 is narrow (ϕ_{ld} ranging from 7.6° to 9.0°). However, the wider range of large-displacement shear strength observed for the individual failure envelopes of reinforced GCLs in Table 2 (ϕ_{ld} ranging from 4.0° to 13.7°) indicates that the variability in large-displacement shear strength should be considered.

Assessment of Shear Strength of GCLs Tested under the Same Conditioning Procedures

The assessments using τ_{50} and τ_{300} allow direct comparison among the shear strength values of different GCL types under representative normal stresses. However, shear strength characterization for design purposes requires the definition of shear strength envelopes that account for the potential effect of GCL conditioning. Comparisons between GCLs tested under similar conditions are discussed below. Additional analyses are provided by McCartney et al. (2002).

Fig. 5(a) shows the τ_p envelopes for GCLs *A* (needle-punched), *B* (stitch-bonded), and *C* (thermal-locked) tested under the same σ_n (34.5, 137.9, 310.3 kPa), t_h (168 hs), t_c (24 hs), and SDR (0.1 mm/min). Typical shear stress-displacement curves for some of these tests are shown in Fig. 2(a). Contrary to the observations made in the overall shear strength analysis, the needle-punched GCL *A* shows higher τ_p than the thermal-locked needle-punched GCL *C* for the full range of normal stresses (34.5 to 310.3 kPa). The thermal-locked GCL *C* appears to have been detrimentally affected by the long hydration time (t_h=168 hs) under the low hydration normal stress of (σ_h=20.7 kPa). Pullout of fibers may have occurred from the woven geotextile of GCL *C* during both hydration and shearing. The fibers in GCL *A* are typically left entangled on the surface of the woven geotextile, so significant swelling or shear displacement is required for pullout of the fibers from the carrier geotextile. On the other hand, the fibers in GCL *C* are melted together at the surfaces of the carrier geotextiles. This is consistent with the results reported by Lake and Rowe (2000), who observed that the melted fibers still pull out of the woven carrier geotextile despite thermal treatment during hydration and shearing. Consistent with trends observed using the overall shear strength assessment, the stitch-bonded GCL *B*

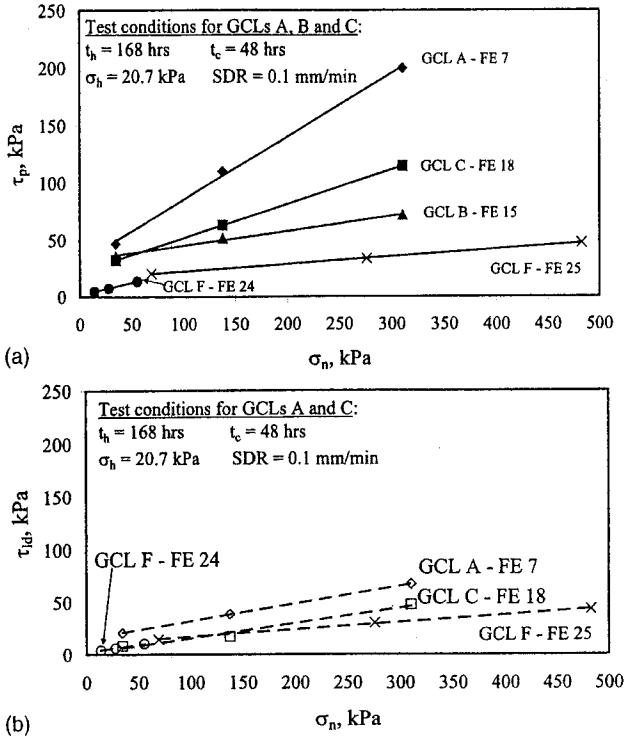

Fig. 5. Comparison of failure envelopes for needle-punched (GCL *A*), stitch-bonded (GCL *B*), thermal-locked (GCL *C*), and unreinforced (GCL *F*) GCLs: (a) peak shear strength; and (b) large-displacement shear strength. Note: When multiple shear strength results are available for a given σ_n, the data points in the figure correspond to the average shear strength value.

shows the lowest τ_p among the different reinforced GCLs. Further, consistent with observations reported by Fox et al. (1998), the continuous fiber reinforcements in GCL *B* did not break during shearing. Instead, the continuous fiber stitches tore the woven carrier geotextile while reaching comparatively large (post-peak) shear displacements. The relatively low reinforcement density (only three lines of stitching in a 305 mm wide specimen) as well as the transfer of shear stress from the stitches to the carrier geotextile during shearing probably contributed to the low τ_p of GCL *B*. Fig. 5(b) shows the τ_{ld} envelopes for the same cases. Similar to the observations for τ_p, the needle-punched GCL *A* has higher τ_{ld} than the thermal-locked GCL *C*.

Also included in Figs. 5(a and b) are the τ_p and τ_{ld} envelopes for unreinforced GCL *F*. The hydration conditioning for tests conducted under comparatively low and high σ_n (below and above approximately 60 kPa) are different. The GCL tested under low σ_n is hydrated, but shows a higher friction angle than the unhydrated GCL tested under higher σ_n. Despite the differences in GCL conditioning between the tests on unreinforced specimens, both τ_p and τ_{ld} for GCL *F* are significantly below those obtained for reinforced GCLs.

Indirect Evaluation of Pore Water Pressures Generated during Shearing

Direct measurement of pore water pressures generated during shearing poses significant experimental challenges and has not been successfully accomplished to date (Fox et al. 1998). While direct measurement of pore water pressures was beyond the scope of the commercial tests in the GCLSS database, some results provide indirect insight into the shear-induced pore water pressures. Such insight is provided by evaluation of direct shear tests conducted using different SDRs and of shear strength envelopes obtained for a wide range of σ_n. Although the behavior of GCLs under comparatively low σ_n has been reported in the technical literature, the response of GCLs under comparatively high σ_n has not been thoroughly investigated so far, probably due to experimental difficulties. Of particular interest in this study is the comparison between the behavior of GCLs tested under σ_n below and above the swell pressure of the GCL. The swell pressure has been defined as the normal stress at which the sodium bentonite in the GCL does not swell beyond its initial thickness (Petrov et al. 1997). Petrov et al. (1997) reported swell pressures ranging from 100 to 160 kPa for thermal-locked GCLs, while lower values were reported by Stark (1997) for one test conducted using a needle-punched GCL. Pore water pressures generated during shearing are indirectly investigated herein by comparing the response of tests conducted under comparatively low and high σ_n.

Evaluation of the Effect of Shear Displacement Rate

The effect of SDR on τ_p and τ_{ld} has been reported by Stark and Eid (1996), Gilbert et al. (1997), Eid and Stark (1997), Fox et al. (1998), and Eid et al. (1999). These studies, which primarily focused on the response of tests conducted under relatively low σ_n, reported an increasing τ_p with increasing SDR. The GCLSS database allows analysis of the effect of SDR on internal shear strength using tests conducted under σ_n values beyond those reported in previous studies. Fig. 6(a) shows the results of tests on GCL *A* conducted under comparatively low σ_n (50 kPa) using the same test conditions (t_h=24 hs, σ_h=σ_n, t_c=0 hs), but varying SDRs (0.01, 0.5, 1.0 mm/min). Consistent with the trend reported in past studies for tests conducted under low σ_n, the results show an increasing τ_p with increasing SDR. Fig. 6(b) shows the results of tests on GCL *A* conducted under high σ_n (520 kPa) using the same test conditions (t_h=312 hs, σ_h=496.8 kPa, t_c=48 hs), but varying SDRs (0.0015, 0.01, 0.1, 1.0 mm/min). Unlike the trend shown in Fig. 6(a) for tests conducted under low σ_n, the results in Fig. 6(b) show a decreasing τ_p with increasing SDR. The results in Figs. 6(a and b) suggest that the large-displacement shear strength appears to approach residual conditions toward the end of the test conducted with high SDR (1.0 mm/min) test while the tests conducted at lower SDRs have not reached this condition at the end of testing.

Fig. 6(c) summarizes the peak shear strength results from Figs. 6(a and b), and includes additional tests conducted to verify the repeatability of results. The value of τ_p decreases at a rate of approximately 15 kPa per log cycle of SDR for tests conducted at σ_n=520 kPa, while it increases at a rate of approximately 12 kPa per log cycle of SDR for tests conducted at σ_n=50 kPa. Varying SDR appears to have a similar effect on τ_p for the σ_n values shown in the figure (e.g., 10 to 15 kPa per log cycle). However, it should be noted that this corresponds to significant changes in τ_p for GCLs tested at σ_n=50 kPa (approximately 40% decrease per log cycle of SDR while it corresponds to smaller changes in τ_p for GCLs tested at σ_n=520 kPa (approximately 10% increase in shear strength per log cycle of SDR). Based on these observations, if design is governed by τ_p, test specification involving comparatively high are acceptable if the σ_n of interest is relatively high, as the test will lead to conservative (i.e., lower) shear

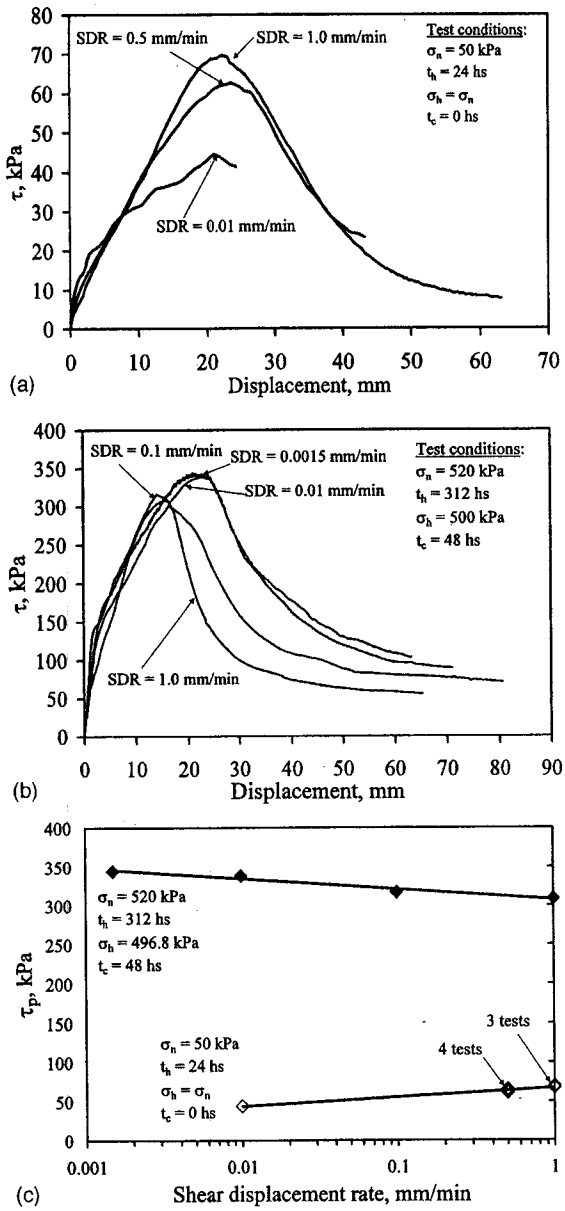

Fig. 6. Effect of shear displacement rate (SDR) on peak shear strength of needle-punched GCL *A*: (a) shear stress-displacement curves for tests under low σ_n (50 kPa); (b) shear stress-displacement curves for tests under high σ_n (520 kPa); and (c) summary trends of peak shear strength as a function of SDR

strength values. However, tests should still be specified with sufficiently low SDR (e.g., 0.1 mm/min) if the σ_n of interest is relatively low.

Explanations proposed to justify the trend of increasing τ_p with increasing SDR observed in previous studies, conducted under relatively low σ_n, have included shear-induced pore water pressures, secondary creep, undrained frictional resistance of bentonite at low water content, and SDR-dependent pullout behavior of fibers during shearing. However, the results obtained from tests conducted under both low and high σ_n suggest that the observed trends are consistent with the generation of shear-induced pore water pressures. Shear-induced pore water pressures are expected to be negative in tests conducted under low σ_n (i.e., below the swell pressure of GCLs). Consequently, increasing SDR will lead to increasingly negative pore water pressures and thus higher τ_p. This trend was also observed for tests conducted on unreinforced

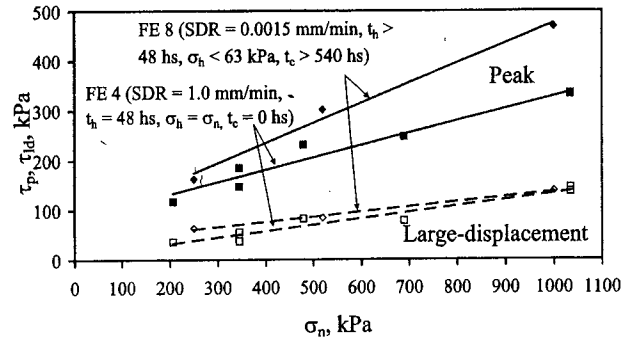

Fig. 7. Effect of shear displacement rate on the peak and large-displacement shear strength of needle-punched GCL *A*

GCLs (Gilbert et al. 1997). On the other hand, shear-induced pore water pressures are expected to be positive in tests conducted under high σ_n (i.e., above the swell pressure of GCLs). In this case, increasing SDR will lead to increasingly positive pore water pressures and thus lower τ_p.

Since no shear-induced pore water pressures are expected (positive or negative) for constant volume conditions, the same residual shear strength is anticipated for different SDRs. Eid and Stark (1999) reported that residual shear strength results were insensitive to SDRs, while Fox et al. (1998) found a slightly increasing strength with increasing SDR for a normal stress of 72.2 kPa. Although residual shear strength was not achieved for the tests reported in Figs. 6(a and b), the tests conducted using higher SDR showed post-peak shear strength loss at comparatively smaller shear displacement values. A consequence of this observation is that, if design is governed by large-displacement shear strength, direct shear tests conducted using high SDR should be adequate for preliminary internal shear strength characterization.

Indirect Evaluation of Pore Water Pressures from Shear Strength Envelopes

Fig. 7 shows FE 8, which includes three tests that were hydrated under a constant low σ_h for more than 48 hs. The normal stress was subsequently increased in stages from σ_h to σ_n during a period of over 540 hs. The specimens were finally sheared using a SDR of 0.0015 mm/min. Determination of the three data points for FE 8 required approximately one year of direct shear testing. For comparison, Fig. 7 also includes data from tests conducted using a SDR of 1.0 mm/min (FE 4). The results in this figure allow investigation of the cumulative effect of conditioning and SDR on the internal shear strength of GCL *A*. For instance, despite the different hydration and consolidation procedures of the three tests in FE 8, a well-defined linear failure envelope was obtained (R^2=0.988). Also, for the range of σ_n shown in this figure (above the swell pressure of GCLs), the trends are consistent with those observed in Fig. 6. That is, the differences in τ_p between FE 4 (SDR=1.0 mm/min) and FE 8 (SDR=0.0015 mm/min) are more significant at higher σ_n because of higher positive pore water pressures induced in FE 4. The direct shear tests corresponding to FE 4 and FE 8 appear to be approaching residual conditions toward the end of the test. The τ_{ld} envelopes suggest that the residual shear strength is approximately insensitive to the different conditioning procedures and different SDRs.

Additional insight on shear-induced pore water pressures can

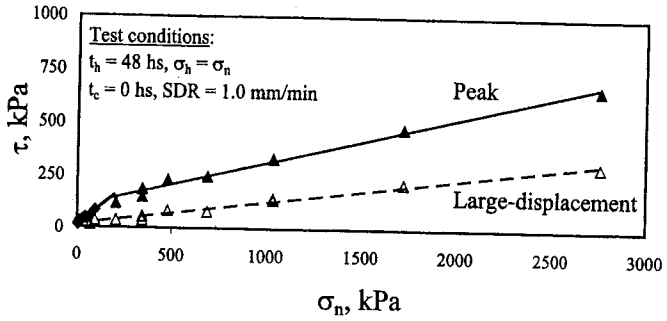

Fig. 8. Typical shear strength envelopes for needle-punched GCL *A* obtained using a wide range of σ_n

be obtained from evaluating shear strength envelopes in the GCLSS database that include tests conducted using σ_n ranging from values below to values above the swell pressure of GCLs. Fig. 8 shows τ_p and τ_{ld} results for tests on GCL *A* (FE 4) conducted using t_h=48 hs, $\sigma_h=\sigma_n$, t_c=0 hs, and SDR=1.0 mm/min. The internal shear strength envelope shown in the figure was defined using 40 direct shear tests. Some tests were conducted using σ_n as high as 2,759 kPa, which corresponds to stresses expected in bottom liners of high landfills or heap leach pads. Tests on GCLs under such high σ_n have not been reported in previous investigations. A linear envelope does not provide a good representation of τ_p over the wide range of σ_n encompassing the swell pressure of the GCL, which is consistent with nonlinear envelopes reported for GCLs (Gilbert et al. 1996; Fox et al. 1998), and for sodium montmorillonite (Mesri and Olson 1970). The GCL and unreinforced sodium bentonite are expected to be influenced by the same mechanisms when tested at normal stresses above and below the swell pressure. As shown in the figure, a bilinear FE provides a good representation of the τ_p data. Linear envelopes fit the τ_p data well for σ_n below approximately 100 kPa (c=14.4 kPa, ϕ=35.4°) and for σ_n above approximately 200 kPa (c=102.4 kPa, ϕ=11.9°). A transition zone appears to take place for σ_n ranging from 100 to 200 kPa, which is within the reported range of GCL swell pressure. The bilinear trend is not caused by a change in fiber failure mechanisms (from pullout to breakage), as the normal stress needed to induce breakage of the polypropylene fibers is well above that of typical geotechnical projects (Zornberg 2002). The τ_{ld} envelope is well represented by a linear envelope characterized by a friction angle of 6.3° and negligible cohesion intercept (c_p=16.2 kPa). Other GCLs in the database, tested under a wide range of σ_n (e.g., FE 16 and 21), show a similar bilinear τ_p response.

Consistent with the results obtained for varying SDR, the break in the bilinear trend in τ_p is in agreement with the generation of negative and positive excess pore water pressures in tests conducted using σ_n below and above the swell pressure of GCLs, respectively. The linear trend obtained for τ_{ld} a wide range of σ_n is also in agreement with the negligible pore water pressures expected under large-displacement conditions.

Variability

The number of test results in the GCLSS database is large enough to provide a basis for assessment of internal shear strength variability. Considering the composite nature of GCLs, the analyses presented herein allow both identification and quantification of different sources of shear strength variability. This information may prove relevant for reliability-based limit equilibrium analyses (McCartney et al. 2004). Potential sources of GCL internal shear strength variability include: (1) Differences in material types (type of GCL reinforcement, carrier geosynthetic), (2) variation in test results from the same laboratory (repeatability), and (3) overall material variability. In turn, the overall material variability includes more specific sources such as: (3-*a*) Inherent variability of fiber reinforcements, and (3-*b*) inherent variability of sodium bentonite. The source of variability (1) listed above is not addressed in this study since only the variability of individual GCL types is evaluated. The sources of variability (2) and (3) are assessed in this study using data presented in Table 4. This table presents a total of seven sets identified for assessment of shear strength variability. Each data set includes tests conducted using the same GCL type, same conditioning procedures, and same σ_n.

Repeatability of Test Results Obtained from the Same Laboratory

The source of variability (2) can be assessed by evaluating Sets V1 and V2 in Table 4, which includes the results of tests conducted by a single laboratory using specimens collected from a single manufacturing lot tested with the same conditioning procedures and same σ_n. Although the size of manufacturing lots is not standardized, it typically involves a set of rolls produced in a shift, day, or even week. Fig. 9 shows shear stress-displacement curves for GCL *A* specimens obtained from rolls of the same lot, which were tested by the same laboratory using the same σ_n. Although the number of tests is small, these results illustrate that good repeatability can be achieved in the stress-strain-strength response when tests are conducted in the same laboratory using same-lot specimens. As indicated by Table 4, the maximum relative difference between these tests is less than 6%, which is significantly smaller than the relative difference associated with different-lot GCLs presented in the next section.

Overall Material Variability

The source of variability (3) may be assessed by evaluating Sets V3 through V7 in Table 4. Unlike the results for Sets V1 and V2 shown in Fig. 9, the GCL specimens in Sets V3 through V7 were obtained from different manufacturing lots. For each set, Table 4 indicates the mean values for τ_p and τ_{ld} [$E(\tau_p)$ and $E(\tau_{ld})$], their standard deviations [$s(\tau_p)$ and $s(\tau_{ld})$], their coefficient of variation c.o.v. values [$s(\tau)/E(\tau)$], and the maximum relative difference. Subsets of data sets V3, V4, and V5 (V3*a* though V3*e*, V4*a* through V4*e*, and V5*a* through V5*e*), in Table 4 include the shear strength variability data corresponding to the manufacturing year of each of the GCL specimens. The maximum relative differences for Sets V3 through V7 (approximately 55%) are significantly higher than those obtained for tests using same-lot GCL specimens (6%). Sets V3, V4, and V5 include data from 141 internal shear strength tests on GCL *A* conducted using the same test conditions (t_h=168 hs, t_c=48 hs, SDR=0.1 mm/min) and three different normal stresses (σ_n=34.5, 137.9, 310.3 kPa). Evaluation of statistical information on the τ_p results for these three sets shows an increasing $s(\tau_p)$ and a relatively constant c.o.v. with increasing σ_n, which indicates that peak shear strength variability increases linearly with σ_n. The c.o.v. and maximum relative difference values are approximately 0.25 and 55%, which are significantly high values for engineering materials. Fig. 10(a) shows the τ_p envelope defined using the mean values of the 141 direct shear test results (Sets V3, V4, and V5 in Table 4). This figure

Table 4. [illegible]synthetic Clay Liner (GCL) Data Sets for Assessment of Shear Strength Variability[illegible]

GCL data set	GCL label	Test conditions: t_h (hs)	t_c (hs)	SDR (mm/min)	σ_n (kPa)	Year GCL manufactured	Number of tests	Peak shear strength: $E(\tau_p)$ (kPa)	$s(\tau_p)$ (kPa)	c.o.v.	Max. rel. difference[a] (%)	Large-displacement shea[illegible]ength: $E(\tau_{ld})$ (kPa)	$s(\tau_{ld})$ (kPa)	c.o.v.	Max. rel. difference[a] (%)
V1	A	24	0	0.5	48.3	1998	3	63.2	2.1	0.03	6	20.7	2.5	0.12	21
V2	A	24	0	0.5	386.1	1998	3	210.7	6.4	0.03	6	79.3	4.8	0.06	11
V3	A	168	48	0.1	34.5	1997–2003	47	35.6	10.4	0.29	64	20.6	6.27	0.30	79
V3a	*A*	*168*	*48*	*0.1*	*34.5*	*1997*	*2*	*52.1*	*4.4*	*0.08*	*11*	*8.3*	*0.0*	*0.00*	*0*
V3b	*A*	*168*	*48*	*0.1*	*34.5*	*1998*	*8*	*44.6*	*3.6*	*0.08*	*24*	*16.5*	*3.1*	*0.19*	*45*
V3c	*A*	*168*	*48*	*0.1*	*34.5*	*1999*	*9*	*47.9*	*6.1*	*0.13*	*33*	*26.0*	*9.9*	*0.38*	*75*
V3d	*A*	*168*	*48*	*0.1*	*34.5*	*2002*	*15*	*28.5*	*2.9*	*0.10*	*32*	*19.9*	*2.9*	*0.15*	*41*
V3e	*A*	*168*	*48*	*0.1*	*34.5*	*2003*	*13*	*27.3*	*5.1*	*0.19*	*42*	*21.2*	*4.5*	*0.21*	*54*
V4	A	168	48	0.1	137.9	1997–2003	47	87.4	22.2	0.25	57	39.3	8.09	0.21	75
V4a	*A*	*168*	*48*	*0.1*	*137.9*	*1997*	*2*	*114.1*	*32.7*	*0.29*	*34*	*13.8*	*0.00*	*0.00*	*0*
V4b	*A*	*168*	*48*	*0.1*	*137.9*	*1998*	*8*	*106.8*	*14.9*	*0.14*	*40*	*34.4*	*6.43*	*0.19*	*43*
V4c	*A*	*168*	*48*	*0.1*	*137.9*	*1999*	*9*	*112.7*	*15.8*	*0.14*	*34*	*43.6*	*9.16*	*0.21*	*48*
V4d	*A*	*168*	*48*	*0.1*	*137.9*	*2002*	*15*	*74.5*	*5.3*	*0.07*	*27*	*37.2*	*4.98*	*0.13*	*33*
V4e	*A*	*168*	*48*	*0.1*	*137.9*	*2003*	*13*	*68.7*	*6.0*	*0.09*	*25*	*43.9*	*4.82*	*0.11*	*29*
V5	A	168	48	0.1	310.3	1997–2003	47	166.0	33.4	0.20	51	66.6	11.75	0.18	56
V5a	*A*	*168*	*48*	*0.1*	*310.3*	*1997*	*2*	*198.9*	*60.0*	*0.30*	*35*	*39.3*	*0.00*	*0.00*	*0*
V5b	*A*	*168*	*48*	*0.1*	*310.3*	*1998*	*8*	*203.0*	*21.0*	*0.10*	*27*	*63.9*	*10.06*	*0.16*	*43*
V5c	*A*	*168*	*48*	*0.1*	*310.3*	*1999*	*9*	*197.2*	*23.2*	*0.12*	*33*	*67.8*	*15.94*	*0.24*	*53*
V5d	*A*	*168*	*48*	*0.1*	*310.3*	*2002*	*15*	*146.5*	*12.8*	*0.09*	*29*	*61.5*	*7.99*	*0.13*	*34*
V5e	*A*	*168*	*48*	*0.1*	*310.3*	*2003*	*13*	*138.9*	*8.8*	*0.06*	*23*	*75.3*	*5.70*	*0.08*	*18*
V6	A	48	0	1.0	9.6	1997	18	31.1	5.8	0.19	55	N/A	N/A	N/A	N/A
V7	F	24	0	1.0	9.6	1999	6	3.9	0.7	0.19	35	3.0	0.5	0.15	35

[a]Maximum relative difference $=[(\max \tau_p - \min \tau_p)/\max \tau_p] \times 100\%$.

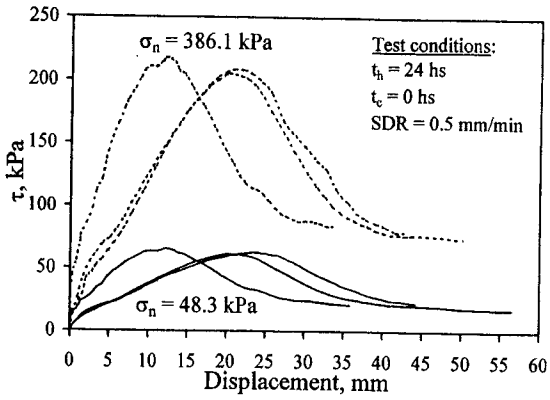

Fig. 9. Repeatability of test results on needle-punched GCL *A* specimens from rolls taken from the same lot

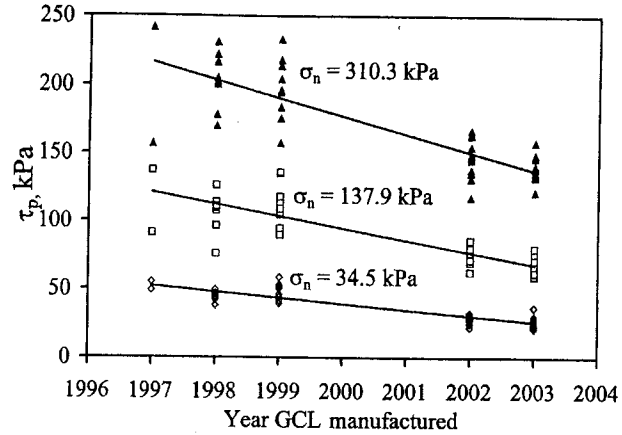

Fig. 11. Peak shear strength of GCL *A* for different manufacturing years

illustrates the significant scatter of results from tests conducted using the same GCL type and test conditions, but using specimens from different GCL *A* lots. Fig. 10(b) shows idealized normal probability density distributions for τ_p at each σ_n, obtained using the mean and standard deviation for the shear strength data of Sets V3, V4, and V5. These probability distributions quantify statistical information on τ_p, which is useful for reliability-based design. Table 4 also includes statistical information regarding τ_{ld}. Although τ_{ld} may not be fully representative of the residual shear strength, the c.o.v. of τ_{ld} is relatively high (up to 0.30), which indicates that the variability in large-displacement shear strength is not less significant than that of peak shear strength.

The 141 GCL specimens in Sets V3 through V5 were received between January 1997 and May 2003. The c.o.v. and maximum relative difference for each of the subsets of Sets V3 to V5 are typically lower each year than for the overall multiyear data sets. For example, the overall c.o.v. for Set V3 is 0.29 while the c.o.v. values for Subsets V3*a* through V3*d* range from 0.08 to 0.19. Fig. 11 shows the shear strength variability for each manufacturing year. A slight decreasing trend in the mean value of the peak shear strength is observed with each subsequent GCL manufacturing year. However, a decreasing trend in the standard deviation value of the peak shear strength is also observed with each subsequent GCL manufacturing year for high normal stresses (e.g., σ_n =137.9 and 310.3 kPa), which may reflect an improvement over time of manufacturing quality assurance programs.

Set V6 in Table 4 includes variability data from a set of 19 direct shear tests conducted using the same GCL tested in Sets V3 through V5 (GCL *A*, manufactured in 1997), but different test conditions (t_h=48 hs, t_c=0 hs, SDR=1.0 mm/min, σ_n=9.6 kPa). The c.o.v. and maximum relative difference for Set V6 are similar to those for Sets V3 through V5 despite the shorter time allowed for conditioning (t_h=24 hs). This suggests that specimen conditioning is not a major source of inherent material variability.

Inherent Variability of Fiber Reinforcements

Peel strength results have been reported to provide an index of the density (and possibly the contribution) of fiber reinforcements in needle-punched GCLs (Heerten et al. 1995, Eid and Stark 1997). Consequently, an assessment is made herein of the usefulness of peel strength as an indicator of the fiber contribution to GCL internal shear strength. If useful, the peel strength variability would be an indicator of the contribution of fibers to the variability of GCL shear strength [source of variability (3-*a*)]. The peel strength test (ASTM 1999) involves clamping the carrier geotextiles of a 100 mm wide unhydrated GCL specimen, and applying a force normal to the GCL plane until separating (or peeling) the geotextiles. It should be noted that the peel strength test mobilizes the fibers in a manner that may not be representative of the conditions in which the fibers are mobilized during shearing.

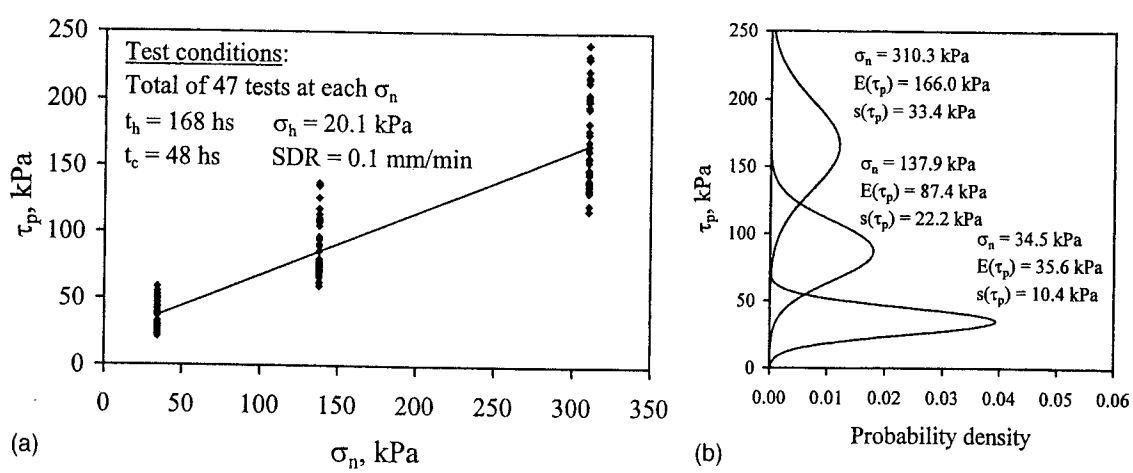

Fig. 10. Variability of peak shear strength results obtained using needle-punched GCL *A* specimens from different lots, tested using same conditioning procedures and σ_n: (a) τ_p envelope; and (b) normal distributions for τ_p at each σ_n

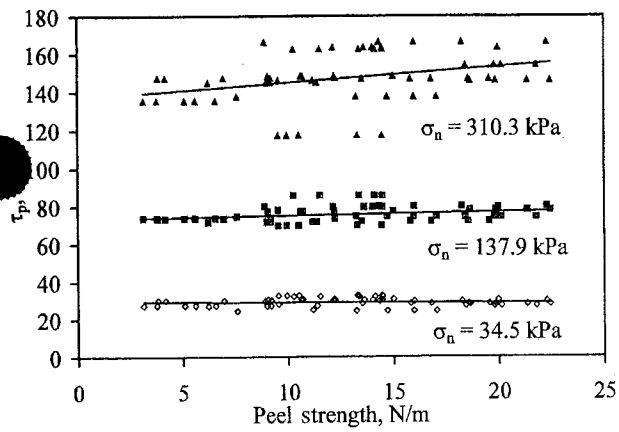

Fig. 12. Relationship between peel strength and τ_p for needle-punched GCL *A*

A total of 75 peel strength tests were conducted using GCL *A* specimens manufactured in 2002. Specifically, five tests were conducted using GCL *A* specimens from 15 rolls (different lots) manufactured in 2002 used for the test results presented in Fig. 10 (Sets V3 through V5 in Table 4). The peel strength specified by the GCL *A* manufacturer is 6.5 N/m. However, peel strength results varied significantly (from 4.3 to 22.5 N/m), with a mean of 12.5 N/m and a standard deviation of 5.51 N/m. The relationship between peel strength and τ_p obtained using GCL specimens collected from these 15 rolls is shown in Fig. 12. Although a slightly increasing trend of peel strength with increasing τ_p can be observed at high σ_n, the results suggest that τ_p is not very sensitive to the peel strength. This is consistent with results reported by Richardson (1997). Consequently, no conclusion can be drawn regarding the effect of the inherent variability of peel strength on the variability of the fiber contribution to GCL internal shear strength [source of variability (3-*a*)]. Instead, these results suggest that mobilization of fiber reinforcement in peel strength tests may not be representative of the mobilization of fibers in shear tests. Accordingly, the peel strength appears not to be a good indicator of the contribution of fibers to τ_p.

Inherent Variability of Sodium Bentonite

The source of variability (3-*b*) may be assessed by evaluating the internal shear strength variability of unreinforced GCLs. Set V7 (Table 4) includes variability data from six direct shear tests conducted using an unreinforced GCL (GCL *F*). The tests were conducted using a relatively low σ_n (9.6 kPa) and the same test conditions (t_h=24 hs, t_c=48 hs, SDR=1.0 mm/min). The variability of direct shear test results for unreinforced GCLs is useful to assess the variability of the bentonite shear strength contribution to the shear strength of reinforced GCLs. It should be noted that adhesives are mixed with the sodium bentonite, but they have been reported to have little effect on the GCL internal shear strength once hydrated (Eid and Stark 1997). The c.o.v. and maximum relative difference of the τ_p obtained for Set V7 using unreinforced GCLs is similar to that obtained for Sets V3 through V6 using reinforced GCLs (c.o.v. of approximately 0.20). In particular, the reinforced GCLs (GCL *A*) in Set V6 were tested under the same σ_n and similar conditioning procedures as the unreinforced GCLs in Set V7. Even though the internal shear strength variability has been attributed mainly to the fibers, the similar magnitude of variability observed in the unreinforced GCLs suggests that the variability of the sodium bentonite [source of variability (3-*b*)] is also relevant.

Conclusions

A database of 414 GCL internal shear strength tests was analyzed in this study. The data were obtained from large-scale (305 mm by 305 mm) direct shear tests conducted by a single laboratory over a period of 12 years using procedures consistent with current testing standards. Shear strength parameters were defined to evaluate the effect of GCL type, indirectly quantify the effect of pore water pressures, and assess sources of internal shear strength variability. The following conclusions can be drawn from this study:

1. Comparisons were made between shear strength values obtained for normal stresses representative of cover and bottom liners (50 and 300 kPa, respectively). This evaluation indicates a high scatter in peak internal GCL shear strength. Reinforced GCLs were observed to have significantly higher peak shear strength than unreinforced GCLs. Stitch-bonded GCLs were observed to have lower peak shear strength than needle-punched GCLs. Needle-punched GCLs with NW-NW GCL carrier geotextile configurations were observed to have higher peak shear strength than those with W-NW GCL carrier geotextiles. Needle-punched GCLs without thermal locking were observed to have higher peak shear strength at low normal stresses than those with thermal locking, but the opposite trend was observed at high normal stresses.
2. Unreinforced GCLs were observed to have lower large-displacement shear strength than reinforced GCLs.
3. Stitch-bonded GCLs showed a higher displacement at peak than the other reinforced GCLs.
4. Thermal locking of needle-punched GCLs was detrimentally affected by long hydration periods under low hydration normal stresses. Thermal locking was observed to be effective at high normal stresses.
5. The peak shear strength of reinforced GCLs was observed to increase with increasing SDR for tests conducted under low σ_n, while the opposite trend was observed under high σ_n. This behavior is consistent with the generation of negative shear-induced pore water pressures under low σ_n (below the swell pressure) and of positive pore water pressures under high σ_n. Consequently, if design is governed by τ_p, test specification involving comparatively high SDR are acceptable if the σ_n of interest is relatively high, as the test will lead to conservative (i.e., lower) shear strength values. However, tests should still be specified with sufficiently low SDR (e.g., 0.1 mm/min) if the σ_n of interest is relatively low.
6. Large-displacement shear strength was achieved at smaller shear displacements in tests conducted using comparatively large SDRs. consequently, tests with high SDR should be adequate if design is governed by τ_{ld}.
7. Peak shear strength results obtained over a wide range of σ_n (up to 2,759 kPa) defined bilinear failure envelopes in which a break was defined for normal stresses consistent with the swell pressure of GCLs.
8. Good repeatability of results was observed for tests conducted by the same laboratory using GCL specimens from the same manufacturing lot. However, significant variability was observed for tests conducted using GCL specimens obtained from different lots over a period of 7 years. Nonetheless, the variability among GCLs manufactured in a single year is less than that observed over the 7 year period.
9. The shear strength variability, quantified by the c.o.v. and maximum relative difference, was observed to increase lin-

early with σ_n, but was found to be insensitive to specimen conditioning procedures.

10. Peel strength results showed a relatively high variability. However, the τ_p was found not to correlate well with the peel strength. Consequently, no conclusions can be drawn regarding the effect of the variability of peel strength on the variability of GCL internal shear strength.
11. The c.o.v. of unreinforced GCLs was observed to be similar to that of reinforced GCLs, indicating that the inherent variability of sodium bentonite is a relevant source of reinforced GCL shear strength variability.

Acknowledgments

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ATTACHMENT 12

Revised Perimeter Berm Slope Stability Analysis

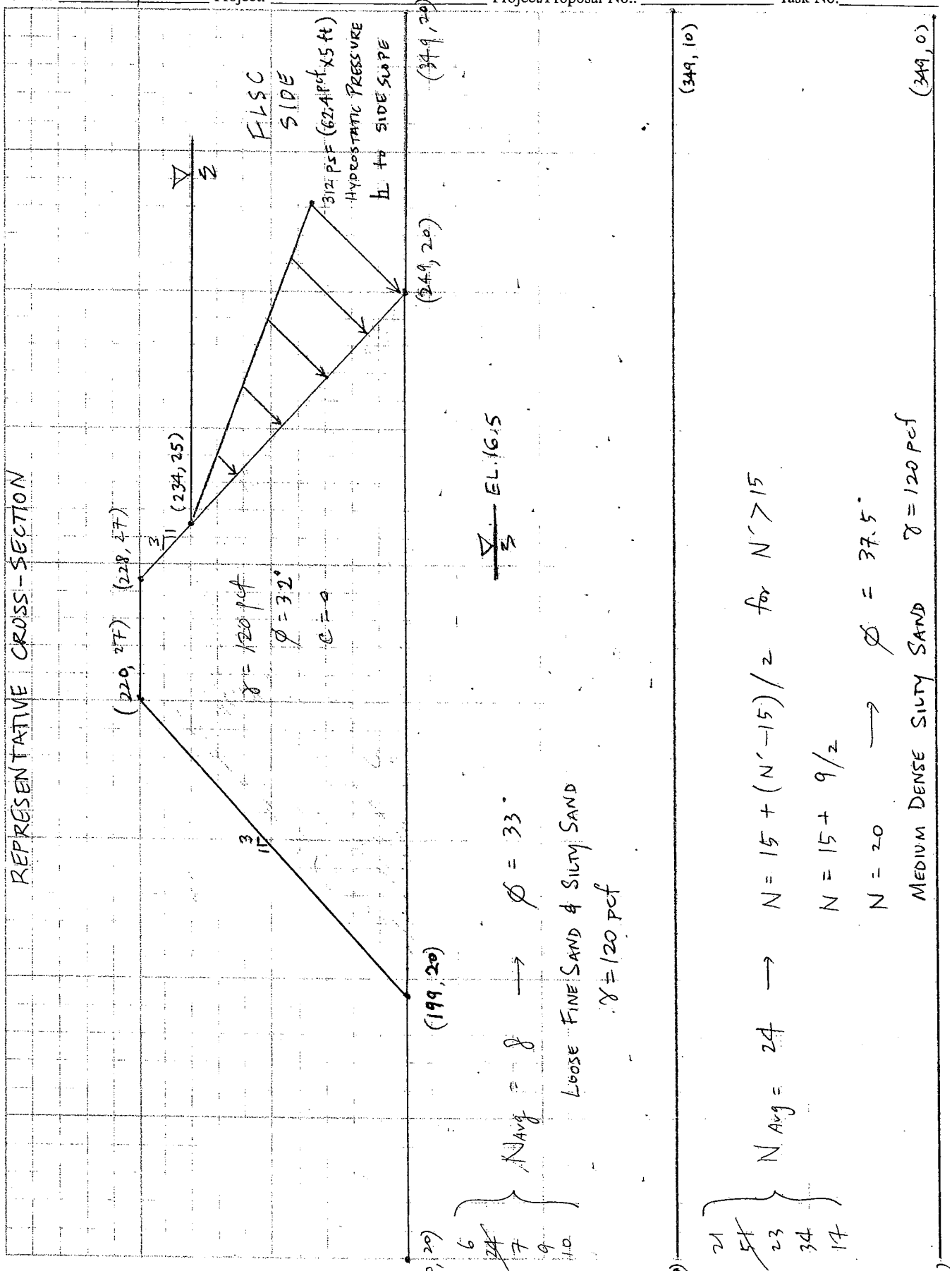
A Gupta
3/29/07

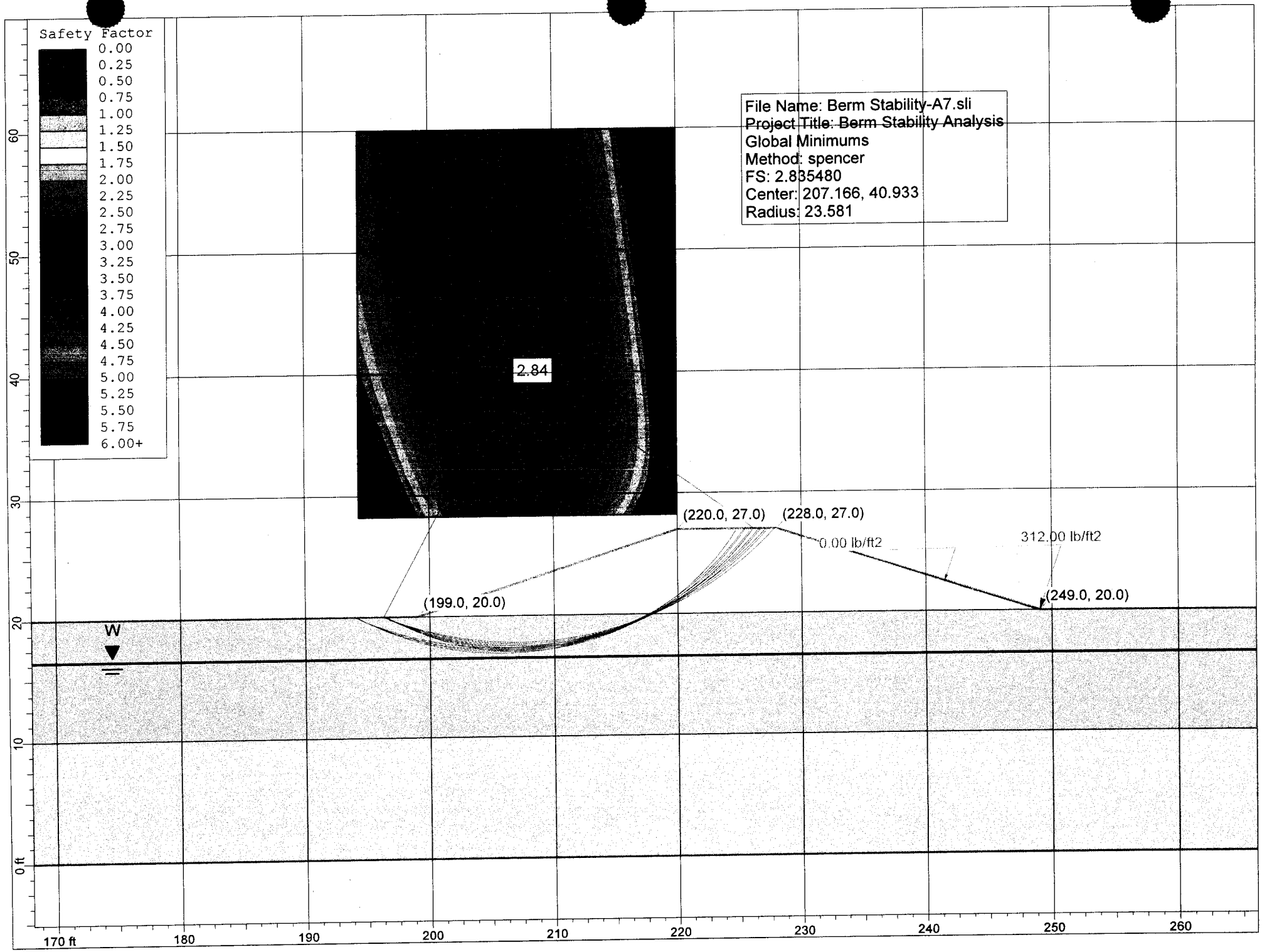
GEOSYNTEC CONSULTANTS

Page _____ of _____

Written by: JUAN QUIROZ Date: 07/03/23 (YY/MM/DD) Reviewed by: _____ Date: ___/___/___ (YY/MM/DD)

Client: Sarasota Project: FLSC Project/Proposal No.: FL1109 Task No: _____

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4.50
4.75
5.00
5.25
5.50
5.75
6.00+
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Project Title: Berm Stability Analysis
Global Minimums
Method: spencer
FS: 2.835480
Center: 207.166, 40.933
Radius: 23.581
2.84
(220.0, 27.0)
(228.0, 27.0)
0.00 lb/ft2
312.00 lb/ft2
(249.0, 20.0)
(199.0, 20.0)
W
60
50
40
30
20
10
0 ft
170 ft
180
190
200
210
220
230
240
250
260

Slide Analysis Information

Document Name

File Name: Berm Stability-A7.sli

Project Settings

Project Title: Berm Stability Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 50
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Circular
Search Method: Grid Search
Radius increment: 20
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: 0
Minimum Depth: Not Defined

Loading

1 Distributed Load present:
Distributed Load Triangular Distribution, Orientation: Normal to boundary, Magnitudes 1,2: 312 and 0 lb/ft2

Material Properties

Material: Berm
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Loose Fine Sand & Silty Sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 33 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Med. Dense Silty Sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 37.5 degrees
Water Surface: Water Table
Custom Hu value: 1

Global Minimums

Method: spencer
FS: 2.835480
Center: 207.166, 40.933
Radius: 23.581
Left Slip Surface Endpoint: 196.309, 20.000
Right Slip Surface Endpoint: 226.191, 27.000
Resisting Moment=245054 lb-ft
Driving Moment=86423.9 lb-ft
Resisting Horizontal Force=9674.63 lb
Driving Horizontal Force=3411.98 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 8833
Number of Invalid Surfaces: 428
Error Codes:
Error Code -102 reported for 6 surfaces
Error Code -103 reported for 419 surfaces
Error Code -111 reported for 3 surfaces

Error Codes

The following errors were encountered during the computation:

-102 = Two surface / slope intersections,
but resulting arc is actually outside soil region.

-103 = Two surface / slope intersections,
but one or more surface / nonslope external polygon
intersections lie between them. This usually occurs
when the slip surface extends past the bottom of the
soil region, but may also occur on a benched
slope model with two sets of Slope Limits.

-111 = safety factor equation did not converge

List of All Coordinates

Water Table

0.0	16.5
349.0	16.5

Search Grid

194.3	28.2
220.0	28.2
220.0	60.0
194.3	60.0

Material Boundary

199.0	20.0
249.0	20.0

Material Boundary

0.0	10.0
349.0	10.0

External Boundary

349.0	0.0
349.0	10.0
349.0	20.0
249.0	20.0
228.0	27.0
220.0	27.0
199.0	20.0
0.0	20.0
0.0	10.0
0.0	0.0

Distributed Load

249.0	20.0
234.0	25.0

ATTACHMENT 13

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

A Water Resources Technical Publication

Second Edition
1973
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1977

C. REQUIREMENTS FOR STABILITY

173. General.—A concrete gravity dam must be designed to resist, with ample factor of safety, these three tendencies to destruction: (1) overturning, (2) sliding, and (3) overstressing.

174. Overturning.—There is a tendency for a gravity dam to overturn about the downstream toe at the foundation or about the downstream edge of any horizontal section. If the vertical stress at the upstream edge of any horizontal section computed without uplift exceeds the uplift pressure at that point, the dam is considered safe against overturning. The most critical condition for inducing overturning is when, at the upstream face, the uplift pressure exceeds the vertical stress at any horizontal section and the combined pressure diagram of figure 223(D) is assumed. Under this condition, if B5 in figure 223(D) is less than the allowable stress for the concrete in any horizontal section or less than the allowable stress in the concrete and foundation for a horizontal section at the foundation, the dam is considered to be safe against overturning.

175. Sliding.—The horizontal force, ΣV, in figure 223 tends to displace the dam in a horizontal direction. This tendency is resisted by the frictional and shear resistance of the concrete or the foundation.

The shear friction factor [4] is the sliding stability criterion for all large concrete dams and should generally be used for small concrete dams on rock foundations. The shear friction factor is:

$$Q=\frac{CA+(\Sigma W-U)\tan\phi}{\Sigma V}$$

where:

C=cohesion value of concrete or rock
A=area of base considered
$\tan\phi$=coefficient of internal friction

The values of cohesion and internal friction of the rock or rock-concrete contact must generally be determined by special laboratory tests. For certain rock types, free from adverse geologic structures, cohesion and internal friction can be estimated from published test data. Rock with infilled jointing or lamination and other adverse geologic structures require investigation and testing of the properties of the rock surfaces and infilling material.

The acceptable factor of safety is dependent on many conditions. For small storage dams where failure would mean loss of life or other catastrophic occurrences, the minimum shear friction factor for normal loading conditions is 4. Under extreme loading conditions, the shear friction factor should be at least 1.5. A typical normal loading condition would include normal headwater, tailwater, uplift, and silt (if applicable). Extreme loading conditions should include the following:

(1) Normal water surface, drains inoperative, and earthquake, or
(2) Maximum water surface and drains inoperative.

For small dams with minimal storage where loss of life, extensive property damage, or any other catastrophic occurrence are not involved in a failure, the acceptable minimum safety factor for rock foundations is 2 for normal loading conditions and 1.25 for extreme loading conditions.

For concrete structures on noncohesive foundation materials, it is usually not feasible to obtain safety factors equivalent to prescribed safety factors for structures on competent rock. However, these structures are usually low and of minimal storage where failure would not involve loss of life or other catastrophic occurrences. A general guide for acceptable factors of safety is 2.0 for normal loading conditions and 1.25 for extreme loading conditions. These safety factors may have to be reduced further in certain cases. The safety factor for these structures is set by selecting a sliding factor, f, where:

$$f=\frac{\text{Coefficient of static friction}}{\text{Factor of safety selected}},$$

then, when $(\Sigma W-U)f\geqq\Sigma V$, the factor of safety against a sliding failure is equal to or greater than that selected. Exact values of the coefficient of static friction between surfaces of the assumed sliding plane must generally be

determined by tests. Many static friction tests have been made and the results published, but care should be exercised in using them. For certain foundations, such as sand and gravel, published factors [9] can be used as a guide in the selection of the coefficient of static friction; however, for rock foundations consideration must be given to the extent of jointing and the jointing pattern before selecting a published friction coefficient. Determination of shear strength characteristics of shales, silts, and clays usually requires testing.

Concrete cutoff walls are often provided on structures constructed on soil foundations. The cutoff, properly located and designed, engages an additional volume of foundation materials that must be moved before the structure can slide. If a stratum weaker than the overlying strata exists in a foundation of rock or soil, the sliding stability should also be investigated along the top of the weak bed. In this case, however, the weight of the overlying strata and the shear resistance of material downstream from the structure also would be considered in computing the sliding factor.

~~**176. Overstressing.**—The unit stresses in the~~ concrete and the foundation must be kept within prescribed maximum values. Normally, the stresses in the concrete of gravity dams within the scope of this text will be so low that a concrete mix designed as specified in appendix F to meet other requirements such as durability and workability will attain sufficient strength to insure a factor of safety of at least 4 against overstressing.

The foundation should be investigated and the maximum allowable stress established. Engineering properties of foundation materials and accompanying considerations affecting such properties are discussed in chapter V. Local codes of allowable bearing pressures and engineers qualified in evaluating the adequacy of foundation materials should be consulted as far as possible before final design. Suggested allowable bearing values for footings for structures appurtenant to small dams are given in appendix C. These may be used as a guide in designing small concrete dams. If there is any doubt as to the proper classification and adequacy of the foundation materials, laboratory tests should be made to determine the allowable bearing pressures.

~~D. DAMS ON PERVIOUS (SOIL) FOUNDATIONS~~

177. General.—Small gravity dams constructed on rock present relatively few difficult foundation problems. The design of dams on pervious (soil) foundations, however, involves problems of erosion of the foundation material, settlement, and seepage under the structure. The complexity of these problems varies greatly and depends on the type, stratification, permeability, homogeneity, and other properties of the foundation materials, as well as the size and physical requirements of the structure itself.

The design of concrete gravity storage dams and diversion dams more than 30 feet high on pervious (soil) foundations usually requires extensive field and laboratory investigations. Such structures are beyond the scope of this text, which for pervious (soil) foundations is limited to gravity dams whose maximum net head (headwater to tailwater) is not appreciably greater than 20 feet.

The control of erosion, seepage, and uplift forces under dams constructed on pervious foundations often requires the use of some, all, or various combinations of the following devices:

(1) Upstream apron, usually with cutoffs at the upstream end.

(2) Downstream apron, with scour cutoffs at the downstream end, and with or without filters and drains under the apron.

(3) Cutoffs at the upstream or downstream end or at both ends of the overflow section, with or without filters or drains under the section.

ATTACHMENT 14

Water Parameters

For each material in the **Define Material Properties** dialog, *Water Parameters* can be defined, which determine the pore pressure calculation for each material. The *Water Parameters* depend on the *Groundwater Method* chosen in the Project Settings dialog.

NOTE: For materials using one of the following Strength Type models:

- *Undrained*
- *No Strength*
- *Infinite Strength*

Water Parameters are not applicable, and are disabled.

Water Surfaces

If the *Groundwater Method* in **Project Settings** is *Water Surfaces*, then the following *Water Parameters* will apply:

Water Surface

The user must choose the *Water Surface* (Water Table or Piezo Line) which corresponds to the material type (soil region) they are defining. Only existing Water Surfaces will appear in the list. Piezo Lines are identified by an ID number. The user may also choose *None* (this is the default selection), if no *Water Surface* is associated with a given material (pore pressure will be zero for a material, if *Water Surface* = *None*).

Hu Coefficient

The Hu Coefficient, as defined in SLIDE, is simply a factor between 0 and 1, by which the VERTICAL distance from a point in the soil (e.g. the center of a slice base) to a Water Surface (either a Water Table or Piezo Line) is multiplied to obtain the pressure head. The Hu Coefficient is used to calculate the pore pressure as follows:

$$u = \gamma_w h H_u$$

where:

u = pore pressure

γ_w = the *Pore Fluid Unit Weight* (entered in the Project Settings dialog)

h = the vertical distance from the base of a slice to a Water Surface

H_u = the Hu coefficient for the soil type (either user defined or *Auto*, see below)

NOTE:

- If the distance *h* is negative, (i.e. Water Surface is below the base of a slice) then the pore pressure is set to zero.

- If a Water Surface is not defined above a given slice, then the safety factor calculation for that particular slip surface will not proceed, and an error message will be written to the file. It is up to the user to ensure that Water Surfaces span all of the required soil regions.

There are two ways of defining the Hu Coefficient - *Auto* or *Custom*.

Custom Hu

With the *Custom* option, the user can enter their own value for Hu. A value between 0 and 1 must be specified. For example:

- Hu = 1 would indicate hydrostatic conditions. This can be used where the Water Surface is horizontal. Where the Water Surface is inclined, setting Hu = 1 will provide a conservative (low) estimate of the safety factor, since in general this will overestimate the true pore pressure. In most cases, the user will simply set Hu = 1, because this represents the worst case scenario (maximum pore pressure).

- Hu = 0 would indicate a dry soil. Pore pressure will be zero. Setting Hu = 0 can be used to turn "off" the pore pressure for a material, although this can also be achieved by setting *Water Surface* = *None*.

- Intermediate values of Hu can be used to simulate head loss due to seepage. This would be applicable where the Water Surface is inclined. The user could create a separate material region for each segment of the Water Surface which is inclined, and enter Hu values less than 1. However, the *Auto* Hu option, described below, can be used to automatically account for the inclination of the Water Surface.

NOTE: if you are using Piezometric Lines, you should, strictly speaking, use the *Custom* Hu option, with Hu = 1. This is because a Piezometric Line is usually a direct representation of the pressure head, for a specific slip surface. However, in SLIDE, the user may decide how to apply the Hu Coefficient, for any type of Water Surface (Water Table or Piezometric Lines).

Auto Hu

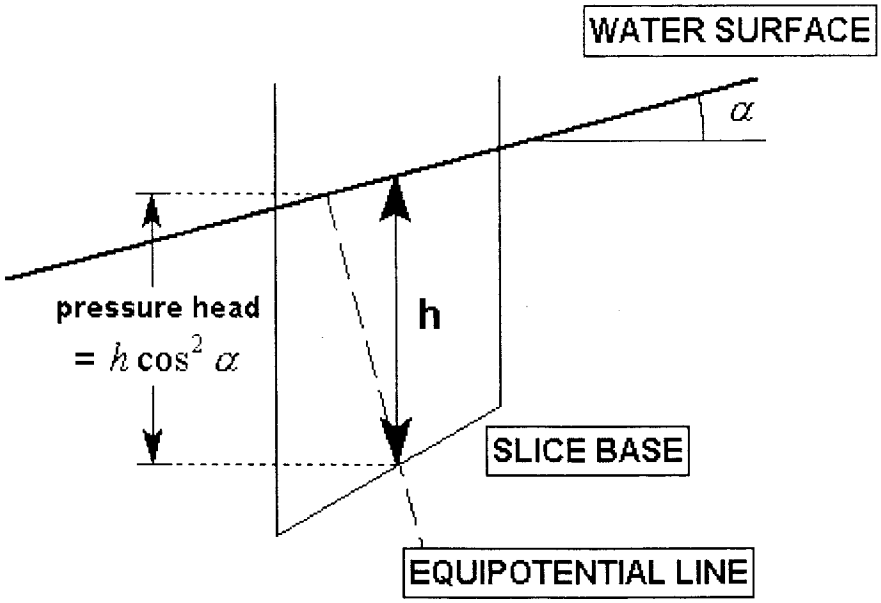
With the *Auto* Hu option, SLIDE will automatically calculate a value of Hu, based on the inclination (angle) of the Water Surface, above any given point. This is based on the assumption that the equipotential line which passes through the center of a slice base, is a straight line, between the slice base and the Water Surface (strictly applicable for an infinite slope case). This is illustrated below.

- α = the inclination of the Water Surface (above a given point)

- h = VERTICAL distance, from center of slice base, to Water Surface

Simple geometry can be used to show that the pressure head, as illustrated in the diagram below, is equal to $h\cos^2\alpha$. The automatically calculated Hu coefficient, is therefore equal to $\cos^2\alpha$. For

a horizontal Water Surface, α = 0, and Hu = $\cos^2\alpha$ = 1.



Automatic Calculation of Hu coefficient

The *Auto* Hu option is a useful method of estimating pore pressures, based on the inclination of a Water Surface. In the absence of more accurate data (e.g. Seepage Analysis results), this is a simple but useful method of approximating head loss due to seepage.

Ru Coefficient

If the *Groundwater Method = Ru Coefficient* in **Project Settings**, then the following Water Parameters will apply:

Ru Value

An Ru coefficient between 0 and 1 must be specified. The Ru coefficient used in SLIDE is the one widely used, which simply models the pore pressure as a fraction of the vertical earth pressure for each slice.

NOTE: the calculation of the vertical earth pressure includes the weight of ponded water, if a material is submerged under water. However, the vertical pressure does NOT include forces due to external loads (e.g. line, distributed or seismic loads).

If one soil type has regions of differing Ru values, then a different material will have to be defined for *each different Ru value*. Appropriate Material Boundaries will have to be added to the model, in order to define the different soil regions.

Using Ru with Water Surfaces or Grids

It is possible to use the Ru method of pore pressure calculation, in conjunction with either Water Surfaces or Water Pressure grids. To enable this feature, you must select the *Allow Ru with Water Surfaces or Grids* checkbox, in **Project Settings**.

ATTACHMENT 15

COPY OF REFERENCE SOURCE:

Giroud, J.P., Soderman, K.L., Khire, M.V. and Badu-Tweneboah, K., "New Developments in Landfill Liner Leakage Evaluation," *Proceedings, Sixth International Conference on Geosynthetics*, International Geosynthetics Society, 1998, pp. 261-268.

New Developments in Landfill Liner Leakage Evaluation

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ABSTRACT: This paper presents: (i) a new equation for calculating the rate of leakage through a composite liner due to geomembrane defects; (ii) a new equation that gives the rate of leakage through defects in a geomembrane placed on a semi-permeable medium; (iii) a new equation that gives the rate of leakage through a defect in a geomembrane liner taking into account the fact that the leachate collection material overlying the geomembrane hinders the flow of leachate toward the defect; and (iv) new equations for the design of leakage collection layers. Then, the paper presents a methodology based on these equations to select the optimal configuration of a double liner system.

KEYWORDS: Landfills, Liners, Geomembranes, Leachate, Leakage.

1 INTRODUCTION

The purpose of this paper is to provide information on new equations for the evaluation of the rate of leakage due to advective flow through defects in geomembranes included in liner systems and for the design of leakage collection layers. These equations were recently developed and published; references are made to the original publications for more details.

2 RATE OF LEAKAGE THROUGH COMPOSITE LINERS DUE TO GEOMEMBRANE DEFECTS

2.1 Presentation of the New Equation

In the context of this paper: (i) a composite liner consists of a synthetic component (a geomembrane) and a mineral component (a low-permeability soil or a GCL); and (ii) the mineral component is located beneath the geomembrane and is designated herein as "the low-permeability medium underlying the geomembrane". Semi-empirical equations are available to calculate the rate of leakage through a composite liner, due to geomembrane defects, when the leachate head on top of the liner is small compared to the thickness of the low-permeability medium underlying the geomembrane, whether the defect is small (Giroud et al. 1989) or large (Giroud et al. 1992). Equations are also available for the case where the leachate head on top of the liner is large compared to the thickness of the low-permeability medium underlying the geomembrane (Giroud et al. 1992, 1994); however, in such a case, graphs are necessary to obtain the value of one of the terms of the equations, which is cumbersome. Giroud (1997) has shown that this term can be expressed analytically, which leads to entirely analytical expressions for the equations that give the rate of leakage through a composite liner, whether the leachate head on top of the liner is smaller or greater than the thickness of the low-permeability medium underlying the geomembrane. The equation in the case of a circular or quasi-circular defect is (Giroud 1997):

$$Q = C_{qo}\left[1+0.1\left(h/t_{UM}\right)^{0.95}\right]a^{0.1}\,h^{0.9}\,k_{UM}^{0.74} \qquad (1)$$

hence, for a circular defect:

$$Q = 0.976\,C_{qo}\left[1+0.1\left(h/t_{UM}\right)^{0.95}\right]d^{0.2}\,h^{0.9}\,k_{UM}^{0.74} \qquad (2)$$

where: Q = leakage rate; a = defect area; d = defect diameter; h = leachate head on top of the liner; t_{UM} = thickness of the low-permeability medium underlying the geomembrane; k_{UM} = hydraulic conductivity of the low-permeability medium underlying the geomembrane; and C_{qo} = dimensionless coefficient that characterizes the quality of contact between the geomembrane and the underlying medium.

Equations 1 and 2 must be used with the following units: Q (m^3/s), a (m^2), d (m), h (m), t_{UM} (m), and k_{UM} (m/s). It should be noted that, when the leachate head on top of the liner is smaller than the thickness of the low-permeability medium underlying the geomembrane, the term in brackets in Equations 1 and 2 is approximately equal to 1. This term

is greater than 1 when the leachate head on top of the liner is greater than the thickness of the low-permeability medium underlying the geomembrane, which is often the case when this medium is a GCL.

Two typical values of C_{qo} are considered: C_{qogood}, the value of C_{qo} in the case of good contact; and C_{qopoor}, the value of C_{qo} in the case of poor contact. Definitions of good and poor contact are given by Giroud (1997). The following values were established by Giroud et al. (1989):

$$C_{qogood} = 0.21 \quad (3)$$

$$C_{qopoor} = 1.15 \quad (4)$$

Equations 1 and 2, and similar equations for rectangular defects and infinitely long defects given by Giroud (1997), supersede equations previously published by Giroud et al. (1992, 1994).

2.2 Limits of Validity of the Equations

The limits of validity of Equations 1 and 2 result from considerations such as: the experimental data supporting Equation 1, the restrictions to flow imposed by surface tension, and the range of applicability of Bernoulli's equation for free flow through an orifice. These limits can be summarized as follows (Giroud et al. 1997c):

- If the defect is circular, the defect diameter should be no less than 0.5 mm and not greater than 25 mm.
- The liquid head on top of the geomembrane should be equal to or less than 3 m.
- The hydraulic conductivity of the low-permeability medium underlying the geomembrane, k_{UM}, should be equal to or less than a certain value k_G. Giroud et al. (1997c) propose the following value for k_G in the case where the geomembrane defect is circular:

$$k_G = \left\{ 0.3891 d^{1.8} \Big/ \left[C_{qo} \left(1 + 0.1 \left(h/t_{UM} \right)^{0.95} \right) h^{0.4} \right] \right\}^{1/0.74} \quad (5)$$

Values of k_G calculated using Equation 5 are given in Table 1.

3 RATE OF LEAKAGE THROUGH DEFECTS IN A GEOMEMBRANE ON A SEMI-PERMEABLE MEDIUM

3.1 Presentation of the New Equation

When a geomembrane is overlain and underlain by infinitely permeable media, the rate of leakage through a geomembrane defect is given by the classical Bernoulli's equation for free flow through an orifice:

$$Q = 0.6a\sqrt{2gh} = 0.15\pi d^2 \sqrt{2gh} = Q_B \quad (6)$$

As shown by Giroud et al. (1997c), Bernoulli's equation is valid if the hydraulic conductivity of the medium underlying the geomembrane is greater than:

$$k_B = 10^5 d^2 \text{ with } k_B\,(\text{m/s}) \text{ and } d\,(\text{m}) \quad (7)$$

Values of k_B calculated using Equation 7 are given in Table 2. A comparison of Tables 1 and 2 reveals that k_G is always smaller than k_B. To evaluate the rate of leakage through defects in geomembranes underlain by a semi-permeable medium, i.e. when the hydraulic conductivity, k_{UM}, of the medium underlying the geomembrane is between k_G and k_B, Giroud et al. (1997c) have developed the interpolation method described below.

Interpolation between Equation 6 for flow through a defect in a geomembrane underlain by an infinitely permeable medium, and Equation 1 or 2 for flow through a defect in a geomembrane underlain by a low-permeability medium gives the following equation for the rate of leakage through defects in a geomembrane placed on a semi-permeable medium (Giroud et al. 1997c):

$$\log\left(Q_B/Q\right) = 0.74 \left[\frac{\log\left(k_B/k_{UM}\right)}{\log\left(k_B/k_G\right)} \right]^{\log(k_B/k_G)} \quad (8)$$

where Q_B is defined by Equation 6, k_B by Equation 7, and k_G by Equation 5.

Table 1. Maximum value, k_G (m/s), of the hydraulic conductivity of the medium underlying the geomembrane for Equations 1 and 2 to be valid in the case where $C_{qo} = 0.21$ (good contact) and $t_{UM} = 0.6$ m (from Giroud et al. 1997c).

Leachate head on top of the geomembrane, h (m)	Geomembrane defect diameter, d (mm)						
	0.5	1	2	3	5	10	11.284
0.01	2.6×10^{-7}	1.4×10^{-6}	7.5×10^{-6}	2.0×10^{-5}	7.0×10^{-5}	3.8×10^{-4}	5.1×10^{-4}
0.03	1.4×10^{-7}	7.7×10^{-7}	4.1×10^{-6}	1.1×10^{-5}	3.8×10^{-5}	2.1×10^{-4}	2.8×10^{-4}
0.1	7.3×10^{-8}	3.9×10^{-7}	2.1×10^{-6}	5.7×10^{-6}	2.0×10^{-5}	1.1×10^{-4}	1.4×10^{-4}
0.3	3.8×10^{-8}	2.1×10^{-7}	1.1×10^{-6}	3.0×10^{-6}	1.0×10^{-5}	5.6×10^{-5}	7.5×10^{-5}
1	1.8×10^{-8}	9.5×10^{-8}	5.1×10^{-7}	1.4×10^{-6}	4.7×10^{-6}	2.6×10^{-5}	3.4×10^{-5}
3	7.1×10^{-9}	3.8×10^{-8}	2.1×10^{-7}	5.6×10^{-7}	1.9×10^{-6}	1.0×10^{-5}	1.4×10^{-5}

Table 2. Hydraulic conductivity of the medium underlying the geomembrane below which Bernoulli's equation for free flow through an orifice is not theoretically valid, k_B (Equation 7), or not applicable for practical purposes, $k_{UM\ min}$ (Equation 11) (from Giroud et al. 1997c).

Geomembrane defect diameter, d (mm)	0.5	1	2	3	5	10	11.284 (a = 1 cm^2)
Theoretical, k_B (m/s)	2.5×10^{-2}	1.0×10^{-1}	4.0×10^{-1}	9.0×10^{-1}	2.5	10	13
Practical, $k_{UM\ min}$ (m/s)	2.5×10^{-4}	1.0×10^{-3}	4.0×10^{-3}	9.0×10^{-3}	2.5×10^{-2}	1.0×10^{-1}	1.3×10^{-1}

Combining Equations 5, 6, 7 and 8 gives:

$$\log Q = 0.3195 + 2\log d + 0.5\log h - 0.74\left(\frac{5 + 2\log d - \log k_{UM}}{n}\right)^n \quad (9)$$

where:

$$n = 5.5540 - 0.4324\log d + 0.5405\log h + 1.3514\log C_{qo} + 1.3514\log\left[1 + 0.1\left(\frac{h}{t_{UM}}\right)^{0.95}\right] \quad (10)$$

The rate of leakage through a defect in a geomembrane underlain by a semi-permeable medium (whose hydraulic conductivity, k_{UM}, is greater than k_G and smaller than k_B) can be calculated using Equation 9, which is equivalent to Equation 8. The genesis of the equation appears more clearly in Equation 8, whereas numerical calculations may be done more conveniently using Equation 9.

3.2 Example of Use of the Equation

Figure 1 shows a series of curves that represent the rate of leakage through a given geomembrane defect (diameter, d = 2 mm) as a function of the hydraulic conductivity of the medium underlying the geomembrane for various leachate heads. Each curve in Figure 1 comprises three portions: the left-hand portion (straight line) represents Equation 2; the right-hand portion (plateau) represents Equation 6; and the central portion (curve) was interpolated using Equation 9. Both Equations 2 and 9 were used with C_{qo} = 0.21 (Equation 3), i.e. assuming good contact between the geomembrane and the underlying medium. The limit value of the hydraulic conductivity between the left-hand portion and the central portion is k_G given by Equation 5 and Table 1; as shown in Table 1, k_G has a different value for each curve. The limit value of the hydraulic conductivity between the central portion and the right-hand portion is k_B given by Equation 7 and Table 2; as shown in Table 2, k_B has the same value for all curves related to the same value of d; for example, for d = 2 mm, k_B = 0.4 m/s. Similar graphs for other values of d are given by Giroud et al. (1997c).

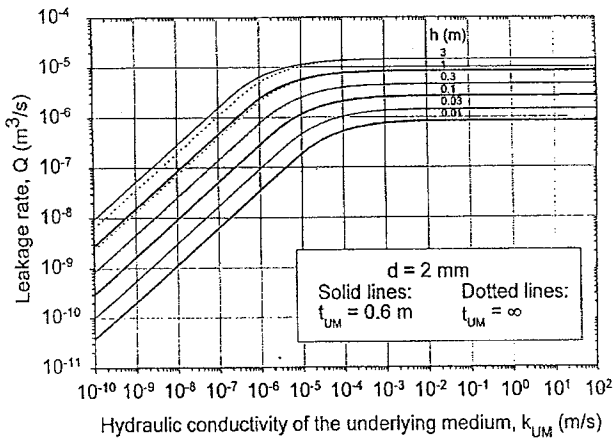

Figure 1. Rate of leakage through a 2 mm diameter defect in a geomembrane underlain by a medium, with a hydraulic conductivity k_{UM} and a thickness t_{UM}, overlain by a medium that is significantly more permeable than the underlying medium, for various values of the leachate head on top of the geomembrane, h (from Giroud et al. 1997c).

3.3 Limit of Applicability of Bernoulli's Equation

As indicated by Giroud et al. (1997c), Bernoulli's equation (Equation 6) provides a value of the leakage rate that is close to the value obtained using the interpolation method presented in Section 3.1, for values of the hydraulic conductivity of the medium underlying the geomembrane that are greater than $k_{UM\ min}$ defined as $k_B/100$. Therefore, the practical limit of applicability of Bernoulli's equation is 100 times smaller than the theoretical limit of validity, k_B, hence, from Equation 7:

$$k_{UM\,min} = 10^3 d^2 \quad \text{with } k_{UM\,min}\,(\text{m/s}) \text{ and } d\,(\text{m}) \quad (11)$$

Values of $k_{UM\ min}$ calculated using Equation 11 are given in Table 2.

It is important to note that, for Bernoulli's equation to be applicable, the geomembrane must be not only underlain by a sufficiently permeable medium, but also overlain by a highly permeable medium. The required minimum hydraulic conductivity of the medium overlying the geomembrane is discussed in Section 4.

4 RATE OF LEAKAGE THROUGH DEFECTS IN A GEOMEMBRANE OVERLAIN BY A PERMEABLE MEDIUM AND UNDERLAIN BY A HIGHLY PERMEABLE MEDIUM

4.1 Presentation of the New Equation

As indicated in Section 3.1, when a geomembrane is overlain and underlain by infinitely permeable media, the rate of leakage through a geomembrane defect is given by the classical Bernoulli's equation for free flow through an orifice (Equation 6). Engineers designing landfills use Bernoulli's equation routinely without questioning its applicability. However, sometimes, absurd results are obtained, such as a calculated rate of leakage through a defect in a geomembrane liner greater than the total rate of liquid supply above the geomembrane.

The absurd results of the type indicated above are caused by an overestimation of the rate of leakage by Bernoulli's equation because this equation is based on the assumption that the hydraulic conductivity of the medium overlying the geomembrane is infinite. In reality, this hydraulic conductivity is not infinite; therefore, the flow of leachate toward the geomembrane defect is hindered and, as a result, the rate of leakage is less than in the ideal case of a geomembrane overlain by an infinitely permeable medium. Taking into account the fact that leachate does not flow freely toward the geomembrane defect, Giroud et al. (1997b) developed the following equation:

$$h = \left\{ \frac{a\, q_i}{2\, k_{OM}\, \pi} + \frac{Q}{2\, k_{OM}\, \pi} \left[\ln\left(\frac{Q}{a\, q_i} \right) - 1 \right] + \frac{1}{4 g^2} \left(\frac{Q}{0.6\, a} \right)^4 \right\}^{1/2} \tag{12}$$

where: q_i = rate of leachate supply on top of the medium overlying the geomembrane; and k_{OM} = hydraulic conductivity of the medium overlying the geomembrane.

It should be noted that, if k_{OM} is infinite, Equation 12 becomes identical to Equation 6, i.e. Bernoulli's equation for free flow through an orifice.

Equation 12 cannot be solved for Q. Therefore, iterations are necessary to determine Q when h, a, k_{OM} and q_i are known. Alternatively, graphical solutions can be used. An example is shown in Figure 2, and a series of similar graphical solutions is provided by Giroud et al. (1997b). Figure 2 shows that, in general, Bernoulli's equation overestimates the leakage rate. However, Figure 2 also shows that, for certain values of the leachate head on top of the geomembrane and the hydraulic conductivity of the medium overlying the geomembrane, Bernoulli's equation provides an excellent approximation of the leakage rate. This is further discussed in Section 4.2.

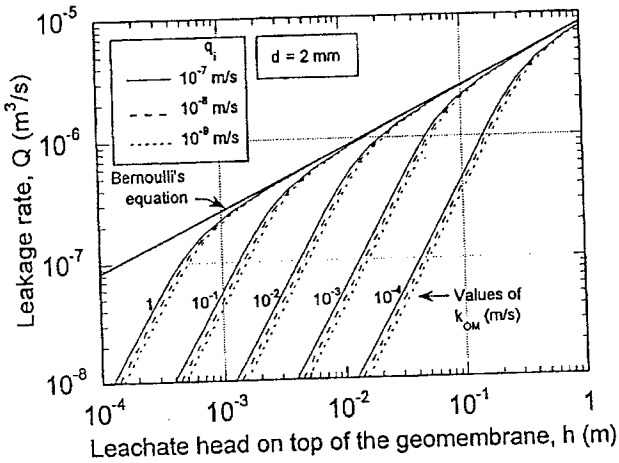

Figure 2. Graphical solution of Equation 12 for a geomembrane defect having a diameter of 2 mm.

4.2 Limit of Applicability of Bernoulli's Equation

Comparing Equations 6 and 12, Giroud et al. (1997b) have shown that Bernoulli's equation gives the rate of leakage through a geomembrane defect with an error less than 5% if the hydraulic conductivity of the medium overlying the geomembrane, k_{OM}, is greater than:

$$k_{OM\,min\,5\%} = \frac{30\, d^2}{h^{3/2}} \tag{13}$$

where the following units should be used: d (m), h (m) and $k_{OM\,min}$ (m/s).

It is interesting to note the consistency between two limits of applicability of Bernoulli's equation: the minimum value of the hydraulic conductivity of the medium *overlying* the geomembrane (given by Equation 13) and the minimum value of the hydraulic conductivity of the medium *underlying* the geomembrane (given by Equation 11). Equations 11 and 13 are consistent for h = 0.1 m, which is remarkable because these two equations were established independently and are related to two different media.

4.3 Relationship Between Liquid Supply and Leakage

For a given permeable medium (such as a leachate collection layer) overlying a geomembrane, the leachate head, h, and the leachate supply rate, q_i, are not independent. The leachate head depends on the leachate supply rate and varies as a function of the distance to the toe of the leachate collection layer slope. As shown by Giroud and Houlihan (1995), in a large number of cases, an excellent approximation of the average leachate head is given by the following equation:

$$h = \frac{q_i L}{2\, k_{OM} \tan\beta} \tag{14}$$

where: β = slope angle of the permeable medium; and L = horizontal projection of the length of the permeable medium in the direction of the flow.

It is then possible to establish a direct relationship between the rate of leachate supply to the permeable medium, q_i , and the rate of leakage through the liner defect, Q. To that end, the leachate head, h, is eliminated by combining Equations 12 and 14, hence:

$$1 = (2/\pi) A\left[1 + C(\ln C - 1)\right] + B^2 (C/0.6)^4 \tag{15}$$

where A, B, and C are dimensionless parameters defined as follows (Giroud et al. 1997b):

$$A = \frac{a\, k_{OM} \tan^2\beta}{q_i L^2} \tag{16}$$

$$B = \frac{q_i\, k_{OM} \tan\beta}{gL} \tag{17}$$

$$C = \frac{Q}{a\, q_i} \tag{18}$$

Equation 15 provides a direct relationship between the rate of leachate supply, q_i, and the rate of leakage, Q. This direct relationship gives a definitive and quantitative answer to the following question often posed when practicing or teaching landfill liner design: is the rate of leakage through geomembrane defects greater if a geomembrane is overlain by a low-permeability leachate collection layer (which slows down the leachate flow toward the defects) or a high-permeability leachate collection layer (which reduces the leachate head over the geomembrane)? The answer to this question can be derived from Figure 3 which provides a graphical solution to Equation 15. Figure 3 shows that the rate of liquid migration through geomembrane defects decreases if A or B increases. From Equations 16 and 17, it appears that both A and B increase when L decreases and β and k_{OM} increase. The influence of L and β was already known through Equation 14: as L decreases or β increases, the leachate head decreases and, consequently, the leakage rate decreases. However, the influence of k_{OM} was not known because k_{OM} is a parameter in both Equations 12 and 14. Therefore, it is important to learn from the above discussion that, for a given situation defined by L, β and q_i, the higher the hydraulic conductivity, k_{OM}, of the leachate collection layer, the lower the leakage rate. Therefore, the answer to the question posed above is that the rate of leakage through geomembrane defects is greater if a geomembrane is overlain by a low-permeability leachate collection layer than by a high-permeability leachate collection layer. It should be noted that this conclusion is based on a demonstration that is limited, as is the scope of Section 4, to the case of geomembranes placed over a highly permeable medium. (However, the same conclusion would be reached if the geomembrane was placed on a low-permeability medium to form a composite liner because the rate of leachate migration through a composite liner is too small to have any significant impact on the leachate head on top of the liner. Therefore, in the case of a composite liner, it is obvious that the rate of leakage through geomembrane defects is greater if the leachate collection layer over the geomembrane has a low hydraulic conductivity than if it has a high hydraulic conductivity.)

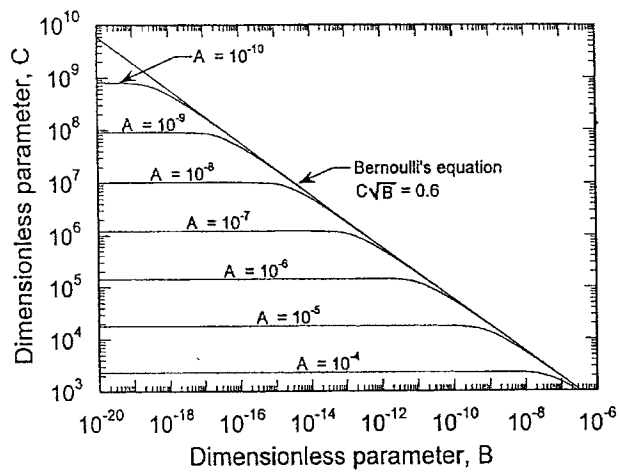

Figure 3. Graphical solution of Equation 15.

5 LEACHATE FLOW IN LEAKAGE COLLECTION LAYERS DUE TO DEFECTS IN GEOMEMBRANE LINERS

5.1 Presentation of the New Equation

Sections 2, 3 and 4 were devoted to the evaluation of the rate of leakage through geomembrane defects, considering several cases of hydraulic conductivities of the media overlying and underlying the geomembrane. Section 5 addresses the flow of leachate in the leakage collection layer located between the two liners in a double liner system. Since only leakage through defects in the primary liner is considered herein, and since the number of defects is generally limited, the leachate generally flows only in portions of the leakage collection layer called the wetted zones. If the defects in the primary liner are sufficiently far apart, the wetted zones related to the various defects do not overlap, and the boundary of the wetted zone related to one defect is approximately a parabola, as shown below.

The leachate flows downslope in the leachate collection layer overlying the primary liner (Figure 4a). A very small fraction of this leachate passes through the primary liner defect, D (Figure 4a). The leachate that has passed through the defect in the primary liner, first flows more or less vertically (DA in Figure 4a) through the leakage collection layer upper part, which is unsaturated. Then, when the leachate reaches (at Point A) the saturated portion of the leakage collection layer, it flows in all directions in the plane of the leakage collection layer (Figure 4a). It is therefore logical to assume that the leachate phreatic surface in the leakage collection layer is a cone with its apex at Point A located vertically beneath the defect in the primary liner (Figure 4a). Furthermore, for leachate to flow in all directions, the hydraulic gradient must be approximately the same in all directions. Since the hydraulic gradient is closely related to the slope of the phreatic surface, it may then be assumed that the slope of the cone generatrices is the same in all directions. The slope of the phreatic surface (i.e. the slope of the cone generatrix) in the downslope direction is approximately known: it is close to the slope angle, β, since the flow thickness is small compared to the length of the leakage collection layer. Therefore, it is assumed that the angle between a horizontal plane and all generatrices of the cone that form the leachate phreatic surface is β (Figure 4a), i.e. the cone axis is vertical.

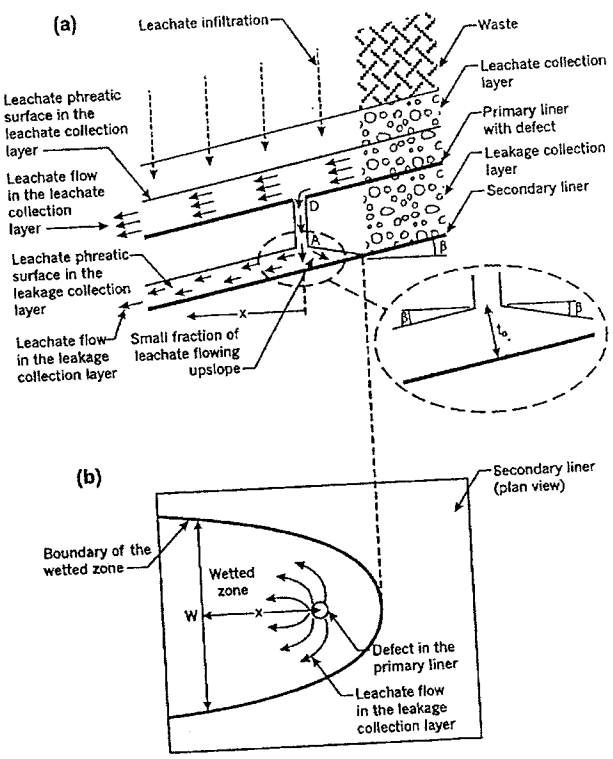

Figure 4. Leachate flow in the leachate collection layer, through a defect in the primary liner, and in the leakage collection layer: (a) cross section; (b) plan view.

From the foregoing discussion, it appears that the wetted zone (Figure 4b) is parabolic since the intersection of a cone and a plane parallel to a generatrix of the cone is a parabola. However, the actual wetted zone is only approximately parabolic because several simplifying assumptions were made, as indicated above.

Giroud et al. (1997a) showed that a consequence of the conical shape of the phreatic surface is the following relationship between the rate of leachate migration through the primary liner defect, Q, the hydraulic conductivity of the leachate collection layer, k, and the thickness of leachate in the leakage collection layer beneath the defect, t_o (Figure 4a):

$$Q = k\, t_o^2 \tag{19}$$

t_o is the maximum thickness of leachate in the leakage collection layer (i.e. the distance between Point A and the secondary liner), hence the condition for the leachate collection layer to not be filled with leachate:

$$t_o = \sqrt{\frac{Q}{k}} \le t_{LCL} \tag{20}$$

Equations 19 and 20 are extremely simple and do not depend on the size of the defect in the primary liner or on the slope of the leakage collection layer.

5.2 Equation of the Boundary of the Wetted Zone

Giroud et al. (1997a) established the equation of the parabola that defines the wetted zone related to a single geomembrane defect. This equation is conveniently expressed as the width of the parabola at the horizontal distance x (Figure 4b) from the geomembrane defect:

$$W = \frac{2\,t_o}{\sin\beta}\sqrt{1+\frac{2x\sin\beta}{t_o}} = 2\,L\mu\sqrt{1+2(x/L)/\mu} \tag{21}$$

where L = horizontal projection of the length of the leakage collection layer in the direction of the slope; and μ is a dimensionless parameter defined as follows:

$$\mu = \frac{t_o}{L\sin\beta} \tag{22}$$

5.3 Wetted Fraction

Typically, there are several defects in a primary liner. The frequency of defects, F, is defined as the ratio of the number of defects in the liner and the surface area of the liner. For example, if there are four defects per hectare, F = 4/10,000 = 4×10^{-4} m^{-2}. The total wetted zone generated by the defects consists of the individual parabolic wetted zones for the various defects. The wetted fraction is defined as the ratio of

the area of the total wetted zone and the surface area of the liner. The individual wetted zones may overlap; the smaller the defect frequency, the smaller the probability for the individual wetted zones to overlap. If the individual wetted zones do not overlap, which is the most frequent case since the defect frequency is generally small, two typical scenarios can be considered: (i) the worst scenario (Figure 5a) where all of the defects are located at the higher end of the primary liner slope, which results in the largest value for the wetted fraction; and (ii) the random scenario (Figure 5b) where the defects are located at random, which results in an average value for the wetted fraction. Using the equation of the parabola (Equation 21), Giroud et al. (1997a) calculated the wetted fraction, $R_{w\ worst}$ in the worst scenario, and $R_{w\ rand}$ in the random scenario:

$$R_{w\ worst} = \lambda_{worst}\, F\, L^2 \tag{23}$$

where λ_{worst} is a dimensionless factor defined as follows:

$$\lambda_{worst} = \frac{2}{3}\mu^2\left[\left(1+\frac{2}{\mu}\right)^{3/2} - 1\right] \tag{24}$$

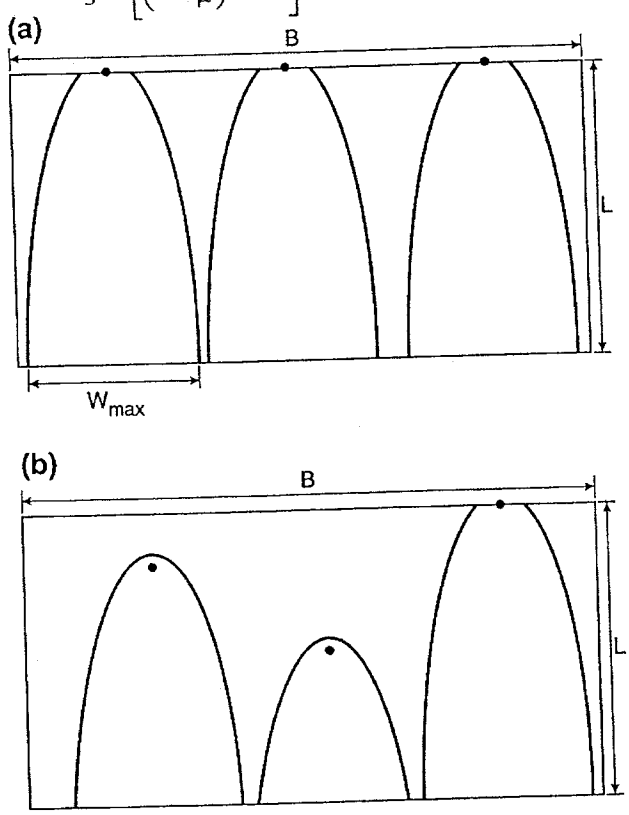

Figure 5. Leakage collection layer zones wetted by leachate migrating through several defects (•) in the primary liner, assuming no overlapping of wetted zones: (a) worst scenario where all of the defects are located at the high end of the primary liner slope; (b) random scenario where the defects are randomly distributed.

$$R_{w\ rand} = \lambda_{rand}\, F\, L^2 \tag{25}$$

where λ_{rand} is a dimensionless factor defined as follows:

$$\lambda_{rand} = \frac{2}{15}\mu^3\left[\left(1+\frac{2}{\mu}\right)^{5/2} - 2\right] \quad (\text{for } \mu \le 2) \tag{26}$$

$$\lambda_{rand} = \frac{2}{15}\mu^3\left[\left(1+\frac{2}{\mu}\right)^{5/2} + \left(1-\frac{2}{\mu}\right)^{5/2} - 2\right] \tag{27}$$

(for $\mu \ge 2$)

5.4 Leachate Head on Top of the Secondary Liner

The leachate head on top of the secondary liner is zero outside the wetted zone. Inside the wetted zone, the leachate head varies from one point to another and an average value, h_{avg}, can be calculated. Based on the conical shape of the phreatic surface (Figure 4a) and using Equations 21 to 27, Giroud et al. (1997a) calculated the average leachate head on top of the secondary liner, $h_{avg\ worst}$ in the worst scenario, and $h_{avg\ rand}$ in the random scenario:

$$\frac{h_{avg\ worst}}{t_o \cos\beta} = \frac{3}{2\mu\left[\left(1+\frac{2}{\mu}\right)^{3/2} - 1\right]} = \frac{\mu}{\lambda_{worst}} \tag{28}$$

$$\frac{h_{avg\ rand}}{t_o \cos\beta} = \frac{(5/3) + [15/(2\mu)]\, x_{rand}/L}{\mu\left[\left(1+\frac{2}{\mu}\right)^{5/2} - 2\right]} \tag{29}$$

It should be noted that $h_{avg\ rand}$ is greater than $h_{avg\ worst}$ because the wetted zone is smaller in the random scenario than in the worst scenario ($R_{w\ rand} < R_{w\ worst}$). However, the total amount of leachate present at a given time in the leakage collection layer is greater in the worst case than in the random case.

As shown by Giroud et al. (1997a), if $R_{w\ worst}$ exceeds 2/3 or if $R_{w\ rand}$ exceeds 4/15, there is a high probability that the individual wetted zones will overlap. In this case, it would be extremely complex to determine the surface area of the wetted zone and, from a practical standpoint, it is preferable to use the approximate approach that consists of assuming that the entire surface area of the secondary liner is wetted (i.e. $R_w = 1$). As shown by Giroud et al. (1997a), the values of the average leachate heads then become:

$$h_{avg\ worst} = \frac{F\, L\, Q}{k \tan\beta} \tag{30}$$

$$h_{avg\ rand} = \frac{F\,L\,Q}{2\,k\,\tan\beta} \qquad (31)$$

5.4 Use of the Equations

To design a leakage collection layer, Equation 20 should be used. This extremely simple equation makes it possible to determine the required thickness of the leakage collection layer, t_{LCL}, as a function of the hydraulic conductivity of the leakage collection layer material, k, to accommodate a given leakage rate, Q.

To calculate the rate of leakage through the secondary liner (i.e. the rate of leakage into the ground), it is necessary to determine the head of leachate on top of the secondary liner. First, Equation 23 or 25 should be used to calculate the wetted fraction. If $R_{w\ worst}$ is less than 2/3 and $R_{w\ rand}$ is less than 4/15, Equations 28 and 29 can be used to calculate the leachate head in the worst and random case, respectively. If $R_{w\ worst}$ exceeds 2/3 or if $R_{w\ rand}$ exceeds 4/15, it should be assumed that the entire surface area of the secondary liner is wetted and Equations 30 and 31 must be used to calculate the average leachate head in the worst and the random case, respectively.

6 USE OF THE EQUATIONS TO SELECT THE OPTIMAL CONFIGURATION OF A DOUBLE LINER SYSTEM

The following methodology based on the equations presented in the preceding sections can be used to calculate the rate of leakage into the ground in the case of a double liner system: (i) calculate the rate of leakage through the primary liner; (ii) calculate the average head of leachate on top of the secondary liner; and (iii) calculate the rate of leakage through the secondary liner. Giroud et al. (1997d) used this methodology to compare two configurations of a double liner system: (i) in the first configuration, the primary liner is a geomembrane and the secondary liner is a geomembrane-GCL composite liner; and (ii) in the second configuration, the same two liners are in the inverse order, i.e. a geomembrane-GCL composite primary liner and a geomembrane secondary liner. They found that the rate of leakage into the ground is much less in the case of the second configuration, thereby showing that, from the viewpoint of minimizing advective flow of leachate, it is preferable to use the composite liner as the primary liner rather than as the secondary liner.

7 CONCLUSIONS

The new equations presented in this paper provide engineers designing landfills or evaluating landfill performance with tools better than previously available. In particular:

- An entirely analytical method to calculate the rate of leakage through defects in the geomembrane component of a composite liner.
- An entirely analytical method to calculate the rate of leakage through defects in a geomembrane placed on a semi-permeable medium.
- An extension of Bernoulli's equation that eliminates the risk of absurd results such as those sometimes obtained with Bernoulli's equation, e.g. calculated leakage rate greater than the leachate supply rate.
- A set of equations that describe the flow of leachate in leakage collection layers and make it possible to design leakage collection layers and to calculate the leachate head on the secondary liner that is needed to calculate the rate of leakage through the secondary liner of a double liner system.

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Evaluation of Landfill Liners

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ABSTRACT: This paper presents equations to evaluate the rate of leakage of liquids such as leachate through liners typically used in landfills including compacted soil liners and several types of geosynthetic liners: geomembrane liners, geoclay liners (panels that consist of a layer of bentonite encapsulated between two geotextiles), and composite liners where a geomembrane is placed on low-permeability compacted soil or geoclay. The results of parametric studies conducted using these equations are tabulated and presented in graphs. These studies show that composite liners are significantly more effective than compacted soil liners or geomembranes placed on permeable media, and that geoclay is a viable alternative to compacted soil in composite liners.

1 INTRODUCTION

A variety of liners constructed with low-permeability soils and/or low-permeability geosynthetics are used in landfills. Low-permeability geosynthetics include geomembranes and geoclays. Geomembranes are either flexible polymeric sheets or geotextiles impregnated with low-permeability compounds. (Herein, only polymeric geomembranes are considered.) Geoclays are panels that consist of a layer of bentonite (a type of clay) encapsulated between two geotextiles, which are generally connected by needlepunching or stitching. Geomembranes and geoclays can be used alone (i.e., on a permeable medium) or can be a component of a composite liner. Typical composite liners consist of a geomembrane on a low-permeability compacted soil layer or a geomembrane on geoclay.

This paper reviews equations that may be used to calculate the rate of leakage through liners and presents comparisons between the various liners based on calculated leakage rates. It should be noted that these comparisons do not include important factors that should be considered for liner selection, such as chemical attenuation as leachate percolates through the liner, time needed by the leachate to percolate through the liner, chemical compatibility between liner material and leachate, mechanical properties of the liner, ease of construction of the liner, cost and availability of liner materials, regulations that affect liner selection, etc. The information presented in this paper may be used by landfill designers to prepare "equivalency demonstrations" that are required by certain regulatory agencies when an alternate liner is proposed instead of a liner prescribed by a regulation.

The comparisons presented in this paper are only applicable to landfills where the head of leachate on top of the liner is small, i.e., typically less than 0.3 m. The relative effectiveness, and even the ranking, of the various liners compared in this paper may be very different under the large liquid heads encountered in liquid impoundments (reservoirs), canals, and dams.

In all the comparisons presented in this paper, soil liners and geoclays are assumed to be in perfect condition, i.e., without preferential flow paths resulting from cracks, zones of high permeability, poor connection between lifts of compacted soil or panels of geoclay, etc. In contrast, geomembranes (which are quasi-impermeable when they are in perfect condition) are assumed to have defects, such as punctures and incomplete seams. Leakage through geomembranes occurs at certain locations only, whereas leakage through soil liners and geoclays occurs over the entire area of the liner due to the permeability of the material.

The paper is organized as follows: Section 2 presents a review of equations for leakage rate evaluation, and Section 3 presents comparisons of the various types of liners based on leakage rates calculated using the equations presented in Section 2.

2 LEAKAGE RATE EVALUATION

2.1 Equations for leakage rate evaluation

Geomembrane liner. As shown by Giroud and Bonaparte (1989a), the rate of leakage through a geomembrane liner due to geomembrane permeability is negligible compared to the rate of leakage through defects in the geomembrane. Consequently, only leakage through defects is considered herein. As proposed by Giroud (1984), Bernoulli's equation for free flow through an orifice can be used to evaluate the rate of leakage through a defect in a geomembrane overlain and underlain by a very permeable medium:

$$Q = 0.6\ a\ \sqrt{2gh} \qquad (1)$$

where: Q = leakage rate; a = defect area; g = acceleration of gravity; and h = hydraulic head on top of the geomembrane.

Equation 1 can only be used if the flow through the geomembrane defect is free, i.e., is not impeded by the materials in contact with the geomembrane. This condition is met if the average opening size, O_{avg}, of the material in contact with the geomembrane is greater than the diameter, d_d, of the geomembrane defect:

$$O_{avg} > d_d \qquad (2)$$

In the case of soils, the following relationship exists:

$$k \approx 10^3 \text{ to } 10^4\ d_{avg}^2 \qquad (3)$$

where: k = hydraulic conductivity of the soil; and d_{avg} = average diameter of soil particles. In Equation 3, often referred to as Hazen's equation, k is in m/s and d_{avg} in m.

In typical soils, the average opening size is approximately one third of the average particle size:

$$O_{avg} \approx d_{avg}/3 \qquad (4)$$

Combining Equations 2, 3 and 4 gives:

$$k > 10^4 \text{ to } 10^5\ d_d^2 \approx 10^4 \text{ to } 10^5\ a \qquad (5)$$

where: a = defect area in m^2.

Although it was demonstrated for soils, Equation 5 is considered to be applicable to any permeable medium with a hydraulic conductivity k. Therefore, free flow conditions are ensured and Equation 1 is valid if the hydraulic conductivity of the media (e.g., soil, geonet) in contact with the geomembrane is greater than 10^{-1} to 1 m/s if a = 0.1 cm^2 (10^{-5} m^2) and greater than 1 to 10 m/s if a = 1 cm^2 (10^{-4} m^2).

Soil liner. The rate of leakage through a soil liner can be evaluated using Darcy's equation (Darcy, 1856):

$$Q/A = ki = k(1 + h/D) \qquad (6)$$

where: A = surface area of the soil liner; k = hydraulic conductivity of the soil; i = hydraulic gradient; h = hydraulic head on top of the liner; and D = thickness of the soil liner. (Note: Hydraulic conductivity is also called "coefficient of permeability" and soils with a small hydraulic conductivity are generally referred to as "low-permeability soils".)

Composite liner. A composite liner is composed of two components: a geomembrane and a layer of low-permeability soil. Herein, the geomembrane is assumed to be on top of the low-permeability soil component, which can be a compacted soil layer or a geoclay.

Based on studies presented by Giroud and Bonaparte (1989b), the following equations were established by Giroud et al. (1989) for the evaluation of the rate of leakage through a defect in the geomembrane component of a composite liner. These equations depend on the quality of contact between the geomembrane and the underlying soil:

$$Q = 0.21\ a^{0.1}\ h^{0.9}\ k^{0.74} \text{ (for good contact)} \qquad (7)$$

$$Q = 1.15\ a^{0.1}\ h^{0.9}\ k^{0.74} \text{ (for poor contact)} \qquad (8)$$

Equations 7 and 8 must be used with the following units: Q (m^3/s), a (m^2), h (m), and k (m/s). These equations are valid if the hydraulic head above the geomembrane is less than the thickness of the soil component of the composite liner (i.e., h < D); therefore, these equations are not applicable to composite liners where the low-permeability soil component is a geoclay (since D, in this case, is very small: typically 6 mm). Also, Equations 7 and 8 are valid only if the hydraulic conductivity, k, of the soil component of the composite liner is less than 10^{-6} m/s, according to Giroud et al. (1989).

In the case where the lower component of the composite liner is a compacted soil layer, good and poor contact conditions were defined by Giroud and Bonaparte (1989b), and described as follows by Bonaparte et al. (1989) and Giroud et al. (1992):

- Good contact conditions correspond to a geomembrane installed, with as few wrinkles as possible, on top of a low-permeability soil layer that has been adequately compacted and has a smooth surface.
- Poor contact conditions correspond to a geomembrane that has been installed with a certain number of wrinkles, and/or placed on a low-permeability soil that has not been well compacted and does not appear smooth.

Good contact conditions are assumed in all the parametric studies presented herein because it is believed that such conditions can be achieved with proper construction and strict quality assurance.

If the head of liquid above the geomembrane is greater than the thickness of the soil component of the composite liner, the following equations established by Giroud et al. (1992) can be used to evaluate the rate of leakage through a geomembrane defect:

$$Q = 0.21\ i_{avg}\ a^{0.1}\ h^{0.9}\ k^{0.74} \text{ (for good contact)} \quad (9)$$

$$Q = 1.15\ i_{avg}\ a^{0.1}\ h^{0.9}\ k^{0.74} \text{ (for poor contact)} \quad (10)$$

where i_{avg} is a dimensionless factor given in Fig. 1. Equations 9 and 10 must be used with the following units: Q (m^3/s), a (m^2), h (m), and k (m/s). Fig. 1 shows that $i_{avg} = 1$ if $h < D$; therefore, increasing the soil component thickness beyond $D = h$ (hydraulic head) does not decrease the calculated rate of leakage through a composite liner.

Equations 9 and 10 are used in the case of liquid impoundments, canals, and dams, where the hydraulic head is large. In the case of landfills, Equation 9 is used if the low-permeability soil component of the composite liner is a geoclay, because the thickness of this material (typically 6 mm) is generally less than the hydraulic head on top of the liner. In this case, good contact conditions can be considered because: (i) geoclay panels have a smooth surface; and (ii) when bentonite hydrates, it swells which presses the geoclay against the geomembrane.

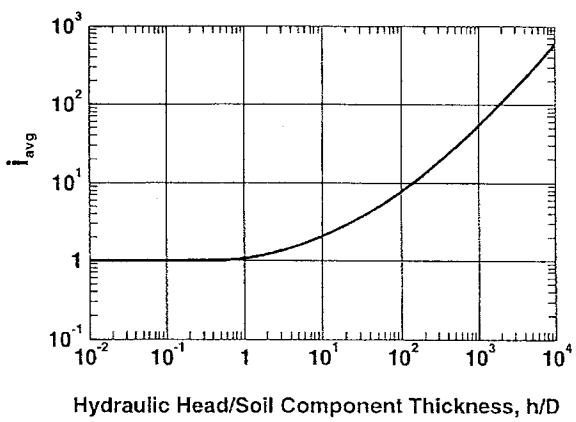

Fig. 1 Value of i_{avg}

In the case of Equations 7, 8, 9 and 10, the liquid first passes through the defect in the geomembrane, then flows laterally some distance between the geomembrane and the underlying low-permeability soil, and, finally, migrates into and eventually through the low-permeability soil. The quality of contact between the geomembrane and the soil governs the amount of lateral flow, hence the difference between Equations 7 and 8, and 9 and 10. Lateral flow would be eliminated in the case of perfect contact between the geomembrane and the soil. Perfect contact does not exist in the case of usual landfill composite liners, as indicated by Giroud and Bonaparte (1989b), but may exist if a low-permeability soil is deposited as a slurry on top of a geomembrane and consolidates with time under a large compressive stress. In this case, the rate of leakage through a geomembrane defect can be evaluated using an equation established by Forchheimer (1930):

$$Q = 4\ r\ h\ k = 4\ h\ k\ \sqrt{a/\pi} \quad (11)$$

where: r = radius of the geomembrane defect.

Equation 11 may be used as a basis to evaluate the typical composite liners used in landfills. Fig. 2 shows that the rate of leakage through a typical composite liner consisting of a geomembrane on a layer of compacted soil with a hydraulic conductivity of 10^{-9} m/s is approximately 1000 to 3000 times greater than the rate of leakage through the same geomembrane defect if the geomembrane were in perfect contact with the soil. It should not be concluded that composite liners are not effective. In fact, although they are not as effective as they could be if the geomembrane/soil contact were perfect, composite liners used in landfills are far more effective than other liners, as shown in Section 3.

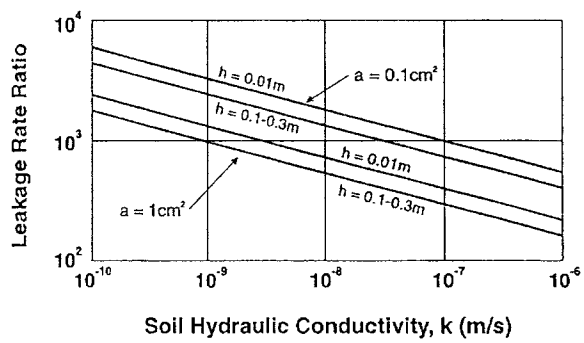

Fig. 2 Ratio between rates of leakage through a composite liner and a geomembrane in perfect contact with soil. (In both cases, the geomembrane has the same defect and is underlain by the same soil.)

2.2 Geomembrane defect size and frequency

Studies presented by Giroud and Bonaparte (1989a) have shown that geomembrane liners installed with strict construction quality assurance could be considered having a frequency of one to two defects per 4000 m^2

with a diameter of 2 mm (i.e., a defect area of 3.14 x 10^{-6} m^2). For the sake of simplicity, a frequency of one defect per 4000 m^2 is considered with a defect area of 0.1 cm^2 (10^{-5} m^2) for liner performance evaluation and a defect area of 1 cm^2 for conservative design.

Electric leak detection surveys (Laine, 1991) have shown that geomembrane liners installed with strict construction quality assurance have five or more defects per 4000 m^2 with a defect diameter less than 0.5 mm. For such defects where the diameter is less than the thickness of the geomembrane, Equations 1, 7, 8, 9 and 10 may not be valid. However, using Equations 1 and 7 for the sake of comparison shows that, in the case of 5 defects having a diameter of 0.5 mm, the rate of leakage is approximately 10 times less with a geomembrane alone and 3 times more with a composite liner than in the case of one defect having an area of 0.1 cm^2. These factors of 1/10 and 3 may be used to modify the rates of leakage presented in Section 3 which were established for one 0.1 cm^2 defect per 4000 m^2.

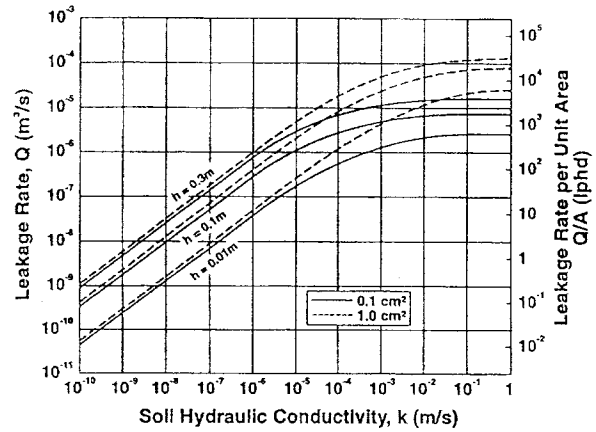

Fig. 3 Rate of leakage through a composite liner for good contact conditions. Values should be multiplied by 5.5 for poor contact conditions, as shown by dividing Equation 8 by Equation 7. If h > D, the above values should be multiplied by i_{avg} given in Fig. 1.

2.3 Graph for leakage rate evaluation

Equations 7, 8, 9 and 10 for composite liners are complex and a graph is useful for rapid leakage rate evaluation and to visualize the influence of parameters. Fig. 3 gives the leakage rate in m^3/s for one defect and the leakage rate per unit area in liters/hectare per day (lphd) assuming one defect per 4000 m^2. The linear portions of the curves were obtained using Equation 7, which is valid for $k < 10^{-6}$ m/s, according to Giroud et al. (1989). The non-linear portion of each curve was graphically interpolated between the end of the linear portion, which occurs for $k = 10^{-6}$ m/s, and the maximum value obtained using Equation 1, which is valid for large values of the hydraulic conductivity of the underlying medium. The non-linear portion of the curves reach the maximum value for $k = 10^{-1}$ m/s if $a = 0.1$ cm^2 and $k = 1$ m/s if $a = 1$ cm^2, as indicated in Section 2.1 after Equation 5.

Fig. 3 shows that when Equation 7 is valid (i.e., for $k < 10^{-6}$ m/s), the size of the geomembrane defect is not a significant parameter. The same would be true for Equations 8, 9 and 10.

3 COMPARISON OF LINERS

3.1 Leakage rate values

Leakage rates per unit area calculated using the equations presented in Section 2.1 are presented in Table 1, which shows that composite liners are significantly more effective than liners made with only one material.

Table 1. Leakage rate per unit area in liters per hectare per day (lphd)[a] through various types of liners.

Liner Type	Soil Hydraulic Conductivity k (m/s)	Hydraulic Head, h (m) 0.01	0.03	0.1	0.3
Soil (b)	10^{-7}	90000	90000	100000	150000
	10^{-8}	9000	9000	10000	15000
	10^{-9}	900	900	1000	1500
Geomembrane (c)	$>10^{-2}$	600	1000	2000	3000
Geomembrane on Semi-Permeable Medium (d)	10^{-3}	300	500	1100	2000
	10^{-4}	100	250	600	1400
	10^{-5}	40	100	200	600
	10^{-6}	10	20	60	150
Geoclay (e)	10^{-11}	25	50	150	450
Composite Liner with Compacted Soil Layer (f)	10^{-7}	1.5	4	12	30
	10^{-8}	0.3	0.7	2	6
	10^{-9}	0.05	0.15	0.4	1
Composite Liner with Geoclay (g)	10^{-11}	0.002	0.008	0.04	0.2

(a) 1 lphd ≈ 10^{-12} m/s ≈ 0.1 gpad (gallons/acre/day).
(b) Equation 6 with 0.3 < D < 0.9 m.
(c) Equation 1 with 1 defect/4000 m^2 having an area $a = 0.1$ cm^2.
(d) Interpolated between Equations 1 and 7 using Fig. 3 for $a = 0.1$ cm^2.
(e) Equation 6 with D = 6 mm.
(f) Equation 7 (for good contact) with 1 defect/4000 m^2 having an area $a = 0.1$ cm^2. In the case of poor contact conditions, leakage rates have to be multiplied by 5.5, as shown by dividing Equation 8 by Equation 7.
(g) Equation 9 with 1 defect/4000 m^2 with an area $a = 0.1$ cm^2.

3.2 Comparison between geomembrane and soil liners

Fig. 4, established using Equations 1 and 6, gives the ratio between the rates of leakage through a compacted soil liner (CSL) and a geomembrane (GM). The soil liner has a thickness, D, ranging from 0.3 to 0.9 m. The geomembrane has one defect per 4000 m^2. Two defect sizes are considered: a = 0.1 cm^2 and a = 1 cm^2. The geomembrane is assumed to be on a very permeable material; therefore, free flow conditions are ensured and Equation 1 is applicable (see Section 2.1 after Equation 5).

Fig. 4 shows that a geomembrane with one 0.1 cm^2 defect per 4000 m^2 is equivalent to a compacted soil liner with a hydraulic conductivity of 10^{-9} m/s. However, the comparison presented in Fig. 4 is only valid if the entire liner (whether it is geomembrane or compacted soil) is exposed to the liquid. This condition is approximately met by the primary liner in a landfill, but is not met by the secondary liner in a double-lined landfill. The secondary liner is exposed in only limited areas to the very small amount of liquid that leaks through the primary liner. All or most of this liquid would percolate into and through a secondary liner made of compacted soil, whereas most or all of the liquid would not encounter the defects of a geomembrane secondary liner and, therefore, would not leak through a geomembrane secondary liner. Clearly a geomembrane secondary liner is far superior to a compacted soil secondary liner.

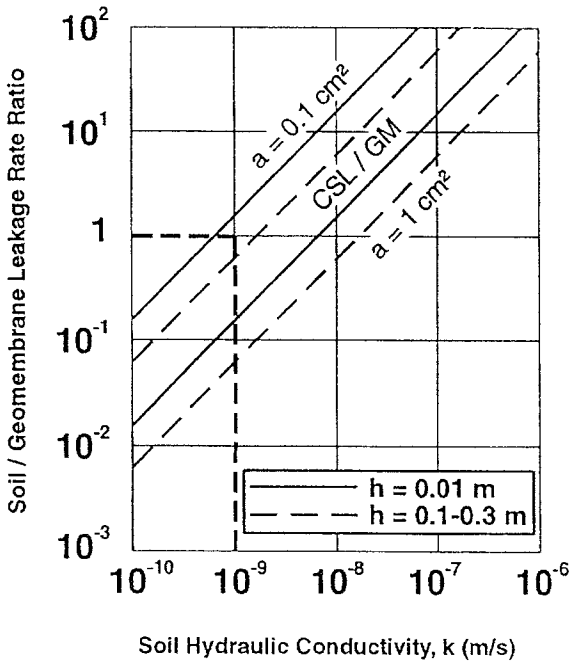

Fig. 4 Comparison between a geomembrane liner and compacted soil liners.

3.3 Evaluation of composite liners

In Fig. 5, established using Equations 1, 6 and 7, two types of curves provide comparisons involving composite liners:

- Curves (1) give the ratio between the rates of leakage through two liners: (i) the first liner is either a compacted clay liner (CCL) with k = 10^{-9} m/s or a geomembrane (GM) placed on a permeable soil (these two liners being equivalent as shown in Section 3.2); and (ii) the second liner is a composite liner with a soil component having a hydraulic conductivity k (GM + CSL). The geomembrane, whether it is used alone or as a component of a composite liner, has one 0.1 cm^2 defect per 4000 m^2.
- Curves (2) give the ratio between the rates of leakage through a compacted soil liner with a hydraulic conductivity k (CSL) and a composite liner made with a geomembrane placed on the same compacted soil liner (GM + CSL).

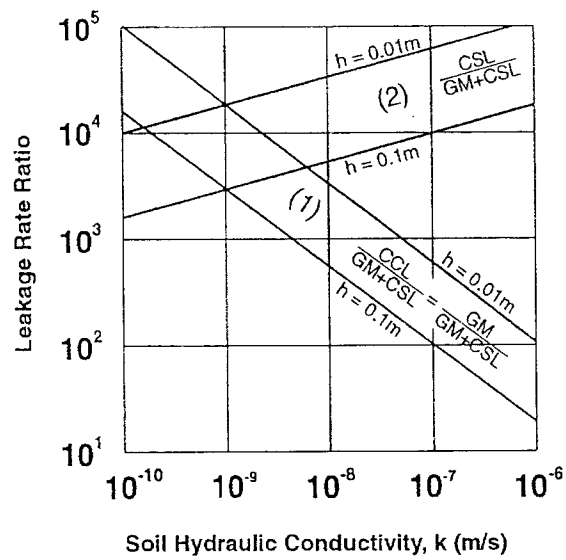

Fig. 5 Evaluation of composite liners.

The following conclusions, drawn from Fig. 5, are valid for k < 10^{-6} m/s, the limit of validity of Equation 7, and for small values of the hydraulic head, h, typical of landfill applications:

- The rate of leakage through a composite liner where the soil component has a hydraulic conductivity k = 10^{-9} m/s is 1000-10000 times less than the rate of leakage through a compacted soil liner with k = 10^{-9} m/s or a geomembrane placed on a permeable soil.
- A composite liner constructed with a soil component having k = 10^{-6} m/s allows 10-100 times less leakage than a compacted soil liner with k = 10^{-9} m/s or a geomembrane placed on a permeable soil.

- The rate of leakage through a composite liner made with a given soil ($k < 10^{-6}$ m/s) is at least 1000 times less than through the soil itself. In other words, placing a geomembrane on the soil decreases the leakage rate by a factor of 1000 or more.

3.4 Evaluation of geoclay liners

Geoclay (GCL) can be used alone as a geoclay liner. Fig. 6 compares such a liner to two other liners:

- The CCL/GCL curve gives the ratio between the rates of leakage through a compacted clay liner and a geoclay liner. This curve was obtained using Equation 6 with $k = 10^{-9}$ m/s and D = 0.3-0.9 m for the compacted clay liner, and $k = 10^{-11}$ m/s and D = 0.006 m (6 mm) for the geoclay liner.
- The GM/GCL curve gives the ratio between the rates of leakage through a geomembrane liner and a geoclay liner. This curve was obtained using: (i) Equation 1 for the geomembrane liner with one 0.1 cm^2 defect per 4000 m^2, assuming that the geomembrane rests on a very permeable soil ($k > 10^{-1}$ m/s); and (ii) Equation 6 for the geoclay liner with $k = 10^{-11}$ m/s and D = 0.006 m (6 mm).

Fig. 6 shows that, under hydraulic heads typical of landfills, the rate of leakage through a geoclay liner is 3 to 40 times less than through a compacted clay liner ($k = 10^{-9}$ m/s) and 7 to 25 times less than through a geomembrane liner located on a permeable soil. Again, the comparison between geomembrane and geoclay is not applicable to secondary liners in double-lined landfills, as discussed in Section 3.2.

Geoclay can also be used as the low-permeability soil component of a composite liner. In. Fig. 6, the GM + CCL/GM+GCL curve gives the ratio between the rates

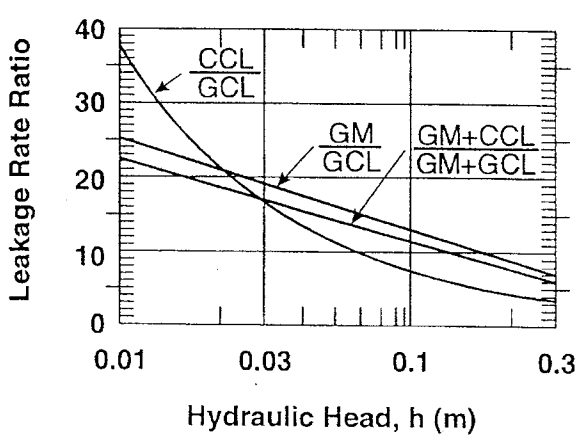

Fig. 6 Leakage rate ratios between liners involving geoclays, which are assumed free from defects.

of leakage through a conventional composite liner, i.e., geomembrane (GM) on a compacted clay layer (CCL) with $k = 10^{-9}$ m/s, and a composite liner consisting of a geomembrane on geoclay (GCL). This curve was established using Equation 7 for the conventional composite liner and Equation 9 for the composite liner with geoclay. It appears that the rate of leakage through a composite liner with geoclay is 6 to 23 times less than through a conventional composite liner.

4 CONCLUSIONS

The equations presented in this paper show that it is possible to evaluate the rate of leakage through all types of liners used in landfills. Comparisons based on these equations show that composite liners are significantly more effective than compacted soil liners or geomembranes used alone on permeable media, and that geoclay is a viable alternative to compacted soil in composite liners.

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An Introduction to Geotechnical Engineering

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TABLE 4-2 Typical Index Properties for Granular Soils*

	Particle Size and Gradation				Voids			
	Approx. Size Range (mm)		Approx. D_{10} (mm)	Approx. Range	Void Ratio		Porosity (%)	
	D_{max}	D_{min}		C_u	e_{max} (loose)	e_{min} (dense)	n_{max} (loose)	n_{min} (dense)
1. Uniform materials:								
(a) Equal spheres	—	—	—	1.0	0.92	0.35	48	26
(b) Standard Ottawa sand	0.84	0.59	0.67	1.1	0.80	0.50	44	33
(c) Clean, uniform sand (fine or medium)	—	—	—	1.2 to 2.0	1.0	0.40	50	29
(d) Uniform, inorganic silt	0.05	0.005	0.012	1.2 to 2.0	1.1	0.40	52	29
2. Well-graded materials:								
(a) Silty sand	2.0	0.005	0.02	5 to 10	0.90	0.30	47	23
(b) Clean, fine to coarse sand	2.0	0.05	0.09	4 to 6	0.95	0.20	49	17
(c) Micaceous sand	—	—	—	—	1.2	0.40	55	29
(d) Silty sand and gravel	100	0.005	0.02	15 to 300	0.85	0.14	46	12

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Designing with GRI Standard GC8

The GC8 standard offers surety—and a consensus approach to synthetic drainage product design.

The design, selection and specification of drainage geonets and geocomposites can be a very subjective affair. It is not uncommon to encounter projects where design engineers are not familiar with reduction factors used to calculate an allowable value of transmissivity or flow rate for a synthetic drainage product. To remedy this situation, the Geosynthetic Research Institute (GRI) published the GC8 standard in 2001. GC8's goal is to establish "uniform test methods and procedures in order for a design engineer to determine allowable flow rate (or transmissivity) of a candidate drainage geocomposite (or geonet) for site-specific conditions." The standard was developed in collaboration with manufacturers, laboratories and design engineers. Thus, it was—and continues to be—a consensus approach towards the design of synthetic drainage products. It is hoped that the information provided in this article will lead to a greater understanding of the standard and, hence, adoption of the procedure by those who are currently unaware of it.

The method

GC8 follows the same "design-by-function" methodology that is the cornerstone of the popular textbook *Designing with Geosynthetics* by Dr. Robert M. Koerner. The standard is focused on the determination of a "q_{allow}" value using the following formula:

Equation 1.

$$q_{allow} = q_{100} \left[\frac{1}{RF_{CR} x RF_{CC} x RF_{BC}} \right]$$

Where q_{allow} = allowable flow rate, q_{100} = flow rate of the geocomposite under laboratory-simulated site conditions for 100 hours, RF_{CR} = reduction factor for creep, RF_{CC} = reduction factor for chemical clogging, and RF_{BC} = reduction factor for biological clogging.

GC8 further recommends the use of a factor of safety, *FS*, for flow rate based on actual design flow requirements, q_{reqd}, as follows:

Equation 2.

$$FS = \frac{q_{allow}}{q_{reqd}}$$

GC8 does not provide design guidelines for calculating q_{reqd}. For landfill projects, useful references on this topic include Giroud et al. (2000 a), Richardson et al. (2000 a, b) and Richardson and Zhao (1998). GC8 provides base line values for chemical and biological clogging reduction factors, RF_{CC} and RF_{BC}, respectively, as shown in **Table 1**. Thus, to calculate the required flow, q_{reqd}, an engineer only needs laboratory data for q_{100} and RF_{CR}, and to assume a relevant factor of safety, *FS*.

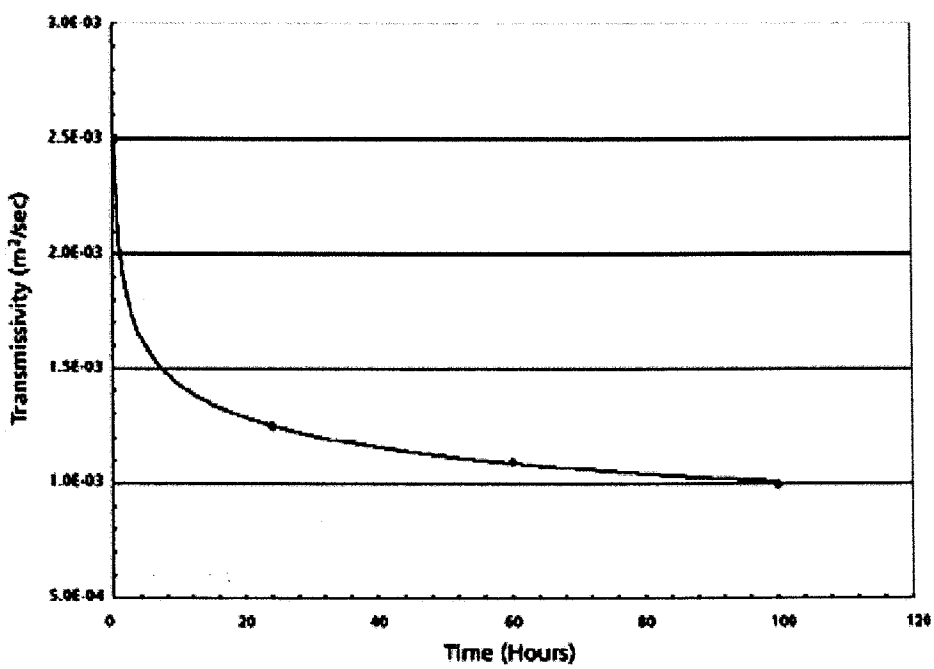
While GC8 speaks in terms of flow rates, project specifications more typically refer to the transmissivity of the drainage composite. The relationship between flow rate and transmissivity in the laboratory is given as

Equation 3.

$$q_{100} = \theta_{100}\, i$$

where θ_{100} is 100-hour transmissivity and i is the flow gradient used in the test.

Figure 1. Typical transmissivity behavior of synthetic drainage products with time.



Hundred-hour transmissivity, θ_{100}

The transmissivity value for any synthetic drainage product, measured according to ASTM procedure D 4716, depends on normal stress, seating time, gradient and boundary conditions. A manufacturer's laboratory, or a third party laboratory, must be provided

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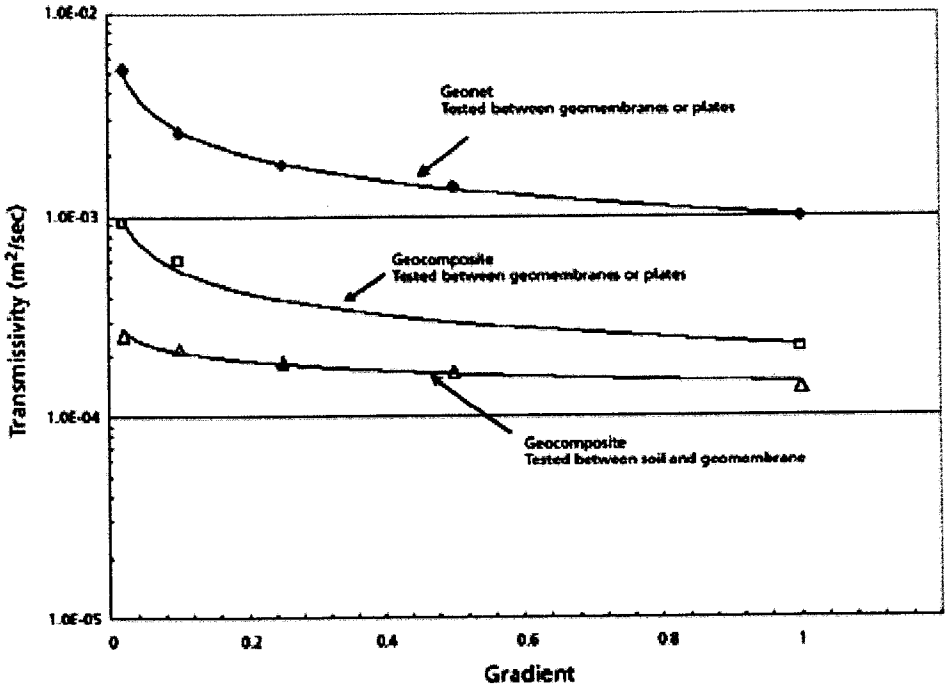
correct test parameters by the designer in order to obtain the value of θ_{100} for use in GC8. The tendency to use a safety factor on normal stress while performing the test is invalid. This violates the very purpose of the GC8 standard, which is to use a factor of safety according to **Equations** 1 and 2; not on individual aspects of the transmissivity test.

The influence of test duration on transmissivity of a synthetic drainage product is illustrated in **Figure** 1. Depending on the type of product and test boundary conditions, the initial compression and certain portion of creep occurs within the first few hours of the test. Within the initial 24 hours, and certainly within 100 hours, the decrease in transmissivity from initial compression of the product has already taken place as indicated by the curve becoming asymptotic to the x-axis. For the purpose of design, a transmissivity value at the end of a 100-hour test is required. The curve is provided here only for the purpose of explanation.

Figure 2. Effect of boundary conditions of geonet transmissivity.



For the same gradient and normal stress, the soft (or soil) boundary conditions always lead to a lower value of transmissivity than hard (plate or geomembrane) boundaries. The reason for this is the higher intrusion of geotextile into the geonet structure under soft boundary conditions. **Figure** 2 illustrates the effect of boundary conditions on transmissivity of a geonet. Notice that the geocomposite transmissivity under soil is less than that between geomembranes (or plates), which in turn is less than the transmissivity of the geonet itself between geomembranes (plates). **Equation** 1 does not include any explicit reduction factor for intrusion, as the 100-hour transmissivity test already includes this effect. Since intrusion of a geotextile into the geonet structure also depends on normal stress, it is very important for the test laboratory to be provided actual normal stress without any escalation or multiplication factor.

Reduction factor for creep, RF_{CR}

All polymeric materials creep, i.e., strain at constant stress. A 100-hour transmissivity test accounts only for initial compression and creep of a geonet within 100

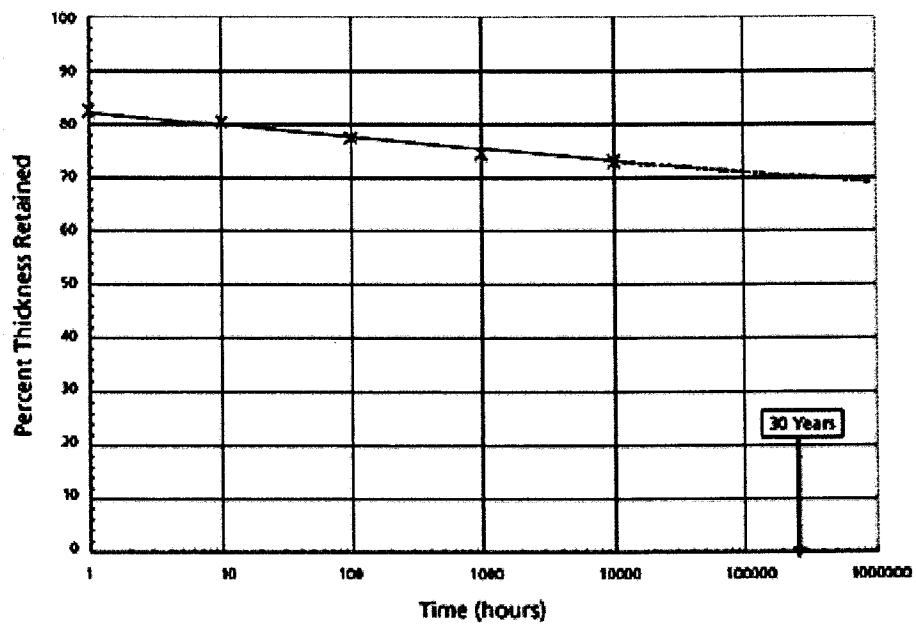
Table 1. Range of clogging reduction factors (from GRI Standard GC8).

Application	Chemical Clogging (RF_{CC})	Biological Clogging (RF_{BC})
Sports fields	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.0 to 1.2	1.1 to 1.3
Roof and plaza decks	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock and soil slopes	1.1 to 1.5	1.0 to 1.2
Drainage blankets	1.0 to 1.2	1.0 to 1.2
Landfill caps	1.0 to 1.2	1.2 to 3.5
Landfill leak detection	1.1 to 1.5	1.1 to 1.3
Landfill leachate collection	1.5 to 2.0	1.1 to 1.3

Table 2. Values of customary factors of safety (modified from Bowles, J.E., 1988).

Structure / Mode	Factor of Safety
Earthworks (Dams, fills, etc.)	1.2 to 1.6
Retaining Structures	1.5 to 2
Footings	2 to 3
Seepage	1.5 to 5

Figure 3. Typical geonet response in a creep test. (Note: The below curve is obtained from SIM test.)



hours. Thus, the transmissivity value from a 100-hour test must be modified to account for creep of the geonet beyond 100 hours and over the life of a project, say over 50 years. This is accomplished through a reduction factor for creep, RF_{CR}, in **Equation 1**. The creep reduction factor is obtained from a 10,000-hour conventional compression creep test or a SIM (Stepped Isothermal Method) creep test. Unlike transmissivity, creep of a geonet is independent of boundary conditions. Geonet manufacturers perform these tests internally, or through independent third party labs, and publish this data for their respective products. Since creep of any geonet depends primarily on stress, it is of paramount importance for a design engineer to provide correct normal stress to a manufacturer. At this stage, a factor of safety or escalation factor should not be used on normal stress, as this would invalidate calculations of an allowable value of transmissivity according to GC8 procedure.

The explanation of creep of geonets is provided in **Figure 3** where percent thickness retained is plotted against time for a biplanar geonet at 15,000 psf normal stress in a SIM test. It is seen that there is a linear relationship between percent thickness retained and logarithm of time. This linear relationship can be extrapolated to the design life of a project to obtain thickness at, say, 30 years. (Note: Such extrapolations are normally performed for no more than one log cycle.) The creep reduction factor was shown by Giroud et al. (2000 b) to equal the following:

Equation 4.

$$RF_{CR} = \left[\frac{t_{CO} - \frac{\mu}{\rho}}{t_{CR} - \frac{\mu}{\rho}} \right]^3$$

Where t_{CO} is the thickness of the geonet at 100 hours, t_{CR} is the thickness of the geonet predicted by the long term creep curve, μ is the mass per unit area, and ρ is the mass density of the geonet polymer.

For Example:

- Original thickness of geonet in **Figure 3** = t_O = 0.6604 cm
- From **Figure 3**, thickness at 100 hours = t_{CO} = (% thickness retained at 100 hours x t_O) / 100 = 0.78 x t_O = 0.515 cm
- From **Figure 3**, t_{CR} = (% thickness retained at, say, 30 years x t_O) / 100 = 0.70 x t_O = 0.462 cm
- Mass per unit area of the geonet in **Figure 3**, μ = 0.1098 g/cm^2
- Density of the geonet in **Figure 3**, ρ = 0.94 g/cm^3

Inputting the values in **Equation 4**, RF_{CR} = 1.5. The value of reduction factor for creep thus obtained can be used in **Equation 1** to obtain allowable flow rate, q_{allow}, for the product. The allowable transmissivity is simply equal to the allowable flow rate divided by the gradient, *i*, used in the 100-hour transmissivity test per **Equation 3**.

Factor of safety for transmissivity, FS

Once the allowable transmissivity, q_{allow}, is known, it can be compared to the required transmissivity, q_{reqd}, determined by design. Assuming that the laboratory flow gradient is the same as the design gradient, then the factor of safety is given by the ratio of q_{allow}/q_{reqd}. If the factor of safety so obtained is inadequate, then either the layout of the project must be changed to lower the required value of transmissivity, or a different product must be evaluated to obtain a higher

value of allowable transmissivity. If the flow gradient used in the laboratory is not the same as that used in design, then **Equation 3** should be used to compare flow under field flow gradient conditions. This later condition is common in the design of floor drainage systems in landfill since a flow gradient less than 0.10 is not commonly used in the transmissivity test.

What is a reasonable, or adequate, value of factor of safety for transmissivity? GC8 does not recommend a factor of safety for transmissivity. Thus, the engineer must use her judgement and experience in selecting a factor of safety. **Table 2** from the popular book *Foundation Analysis and Design* by J. E. Bowles provides factors of safety customarily used in geotechnical engineering. Additionally, a factor of safety of 1.5 to 3 is used against geomembrane puncture in landfills and 2 to 3 against rupture of geogrids in reinforced slopes and walls. The authors recommend using a safety factor, *FS*, of 2.0.

Conclusions

GRI GC8 provides a detailed and comprehensive method to calculate allowable transmissivity of geonets and geocomposites. The data required for such calculations is readily available, except for the 100-hour transmissivity test which must be performed for site-specific conditions. The 100-hour test should be performed with normal loads, gradients and boundary conditions that accurately model field conditions. The use of partial factors of safety on individual test parameters is discouraged and will lead to an overly conservative estimate of the required transmissivity. The cost impact of such partial factors of safety is considerably greater than the cost of a well run 100-hour transmissivity test.

GFR

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DESIGN OF LATERAL DRAINAGE SYSTEMS FOR LANDFILLS

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on the underlying liner system and therefore a reduced potential leakage. Thus equivalence here is based on equal flow properties and not equal leakage of the lateral drain and liner systems.

Table 3.2. Values of the equivalency factor, *E*, between a geosynthetic liquid collection layer and a granular liquid collection layer when the prescribed maximum liquid thickness is 0.3 m (1 ft), as a function of the slope, *β*, and the length, *L*, of the liquid collection layer.

	Slope of the liquid collection layer, tanβ								
m (ft)	0.02	0.03	0.04	0.05	0.1	1/4	1/3	1/2	1
15 (50)	2.43	2.00	1.78	1.65	1.39	1.24	1.21	1.18	1.15
30 (100)	1.78	1.57	1.46	1.39	1.26	1.19	1.17	1.16	1.15
45 (150)	1.57	1.42	1.35	1.31	1.22	1.17	1.16	1.15	1.14
60 (200)	1.46	1.35	1.30	1.27	1.20	1.16	1.15	1.15	1.14

3.1.4 Adequacy of HELP Model

Research by Thiel and Stewart (1993), Soong and Koerner (1997) indicate that the HELP model significantly underestimates percolation into a lateral drainage layer. Eight seepage-induced landfill slope failures have been recorded and analyzed to confirm this by Soong and Koerner (1997). The federal and state minimum permeability value of $1*10^{-2}$ cm/sec or $1*10^{-3}$ cm/sec for drainage soils was found too low by a factor of 10, and in some cases 100. Higher permeability drainage media or high performance drainage geocomposites are recommended. When the permeability of drainage media is increased to $1*10^{-1}$ cm/sec (increased by a factor of 10 over $1*10^{-2}$ cm/sec), the minimum 'prescriptive' transmissivity of a geosynthetic product is increased to $(2.4 - 6.0)*10^{-3}$ m^3/sec-m.

3.2 Long-Term-In-Soil Performance of Geocomposite Drains

Lateral drainage systems degrade with time due to the very liquids they carry and the normal loads they are subjected to. A geocomposite liquid collection layer must have sufficient flow capacity to ensure that there is no pressure buildup within the liquid collection layer. Therefore, to ensure long-term performance, the hydraulic design of a geocomposite liquid collection layer must ensure that the liquid collection layer has sufficient flow capacity under the conditions that exist in the field during the entire design life of the liquid collection layer. The flow capacity under those conditions is referred to as "long-term-in-soil flow capacity".

Thus, the designer must provide surplus initial hydraulic capacity in the lateral drain to ensure that flow within the lateral drain remains unconfined. The discussion in this section is intended to provide the reader with a comprehensive background in the various factors that can degrade the performance of a lateral drain. Many of these factors can be quantified during laboratory testing of the geocomposite. However, many factors are not readily quantified. This requires significant judgement on the part of the designer. Such judgement must be tempered by the criticality of the application and the impact of potential failure.

The long term performance of a lateral drain requires a larger initial transmissivity, θ_{LTIS}, than that obtained from the design equations, $\theta_{req'd}$. This process was initially quantified by Koerner (1998) as follows:

$$FS = \frac{\theta_{LTIS}}{\theta_{req'd}} \qquad \text{Eq. 3.2}$$

$$\theta_{LTIS} = \frac{\theta_{measured}}{RF_{in} \cdot RF_{cr} \cdot RF_{cc} \cdot RF_{bc}} \qquad \text{Eq. 3.3}$$

where FS is the overall safety factor for drainage, θ_{LTIS} is the long-term-in-soil hydraulic transmissivity of the drainage geocomposite, $\theta_{req'd}$ is the required transmissivity (e.g., for MTG= $3*10^{-5}$ m^3/sec-m), $\theta_{measured}$ is the transmissivity measured in accordance with ASTM D4716, and RF are service reduction factors described as follows:

RF_{in} = reduction factor for elastic deformation, or intrusion of the adjacent geotextiles into the drainage channel.
RF_{cr} = reduction factor for creep deformation of the drainage core and/or adjacent geotextile into the drainage channel.
RF_{cc} = reduction factor for chemical clogging and/or precipitation of chemicals in the drainage core space.
RF_{bc} = reduction factor for biological clogging in the drainage core space.

Suggested empirical default values of the reduction factors are listed in Table 3.1 (Koerner, 1998). Currently, laboratory testing can be performed to evaluate RF_{in} and RF_{cr} on a site and drainage composite specific basis. Such testing is discussed in this chapter.

Table 3.1 Recommended preliminary reduction factor values for determining allowable flow rate or transmissivity of geonets (Koerner, 1998)

Application area	RF_{in}	RF_{cr}	RF_{cc}	RF_{bc}
Surface water drains for covers	1.3 - 1.5	1.1 - 1.4	1.0 - 1.2	1.2 – 1.5
Leachate Collection and Removal Systems (LCRS)	1.5 - 2.0	1.4 - 2.0	1.5 - 2.0	1.5 - 2.0
Leachate Detection Systems (LDS)	1.5 - 2.0	1.4 - 2.0	1.5 - 2.0	1.5 - 2.0

While the above total safety factors may appear to be very conservative there may be long-term service reduction factors not accounted for. For instance, Figure 3.1 shows extensive root penetration into a geonet that was recovered from a failed landfill cover. The root penetration was so dense that the transmissivity of the geonet drainage core was essentially reduced to zero. The authors feel that root penetration in cover lateral drains can be minimized only by using high capacity drainage composites that quickly remove water from the drain so that roots are not attracted within the core.

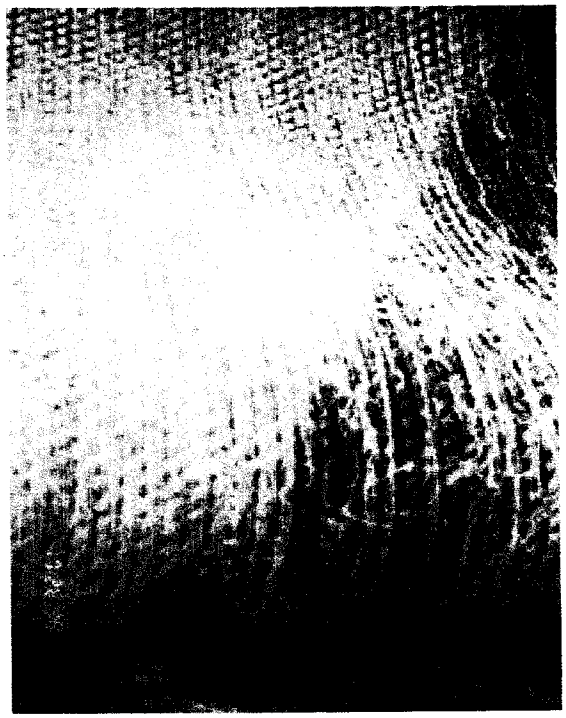

Figure 3.1 Root penetration into the geonet

More recent work by Giroud et al. (2000) has defined additional long-term service factors that influence the performance of a geocomposite lateral drainage system. This work builds on the previous work by Koerner (1998) and follows the basic equation form presented as Eq. 3.3. Giroud observed that the flow capacity of a geocomposite in the field can be reduced by a variety of mechanisms that depend on the following parameters: the applied stress, time, contact with the adjacent materials, and environmental conditions (e.g. presence of chemicals, biological activity, temperature).

More specifically, the thickness and/or the hydraulic conductivity of the transmissive core of a geocomposite may be reduced by instantaneous compression of the core, intrusion of the geotextile filter into the core, time-dependent compression (i.e. creep) of the core, and additional intrusion of the geotextile into the core due to time-dependent deformation of the geosynthetic. These four mechanisms are caused by the applied stresses.

In addition, chemical degradation of the polymeric compound(s) used to make the transmissive core may reduce its effective thickness and/or its hydraulic conductivity. Finally, clogging of the transmissive core may reduce its effective thickness and/or its hydraulic conductivity. Clogging results from physical, chemical and biological mechanisms; biological clogging is typically caused by the growth of micoorganisms (Giroud 1996), but an extreme case is that of clogging due to root penetration in the drainage medium, see Figure 3.1.

A given mechanism (e.g. compression or clogging) may result in (or may be interpreted as) a reduction in effective thickness and/or a reduction in hydraulic conductivity. Therefore, to evaluate the decrease in flow capacity of a geocomposite, it is simpler to use the hydraulic transmissivity, which is the product of thickness

and hydraulic conductivity (see Equation 3.1). Accordingly, from a practical standpoint, the decrease in flow capacity due to the mechanisms described above is expressed by using reduction factors on the hydraulic transmissivity as follows:

$$\theta_{LTIS} = \frac{\theta_{measured}}{\Pi(RF)} = \frac{\theta_{measured}}{RF_{IMCO} \times RF_{IMIN} \times RF_{CR} \times RF_{IN} \times RF_{CD} \times RF_{PC} \times RF_{CC} \times RF_{BC}} \qquad \text{Eq. 3.6}$$

where θ_{LTIS} = long-term-in-soil hydraulic transmissivity of the considered geosynthetic., $\theta_{measured}$ = value of hydraulic transmissivity measured in a laboratory test, and $\Pi(RF)$ = product of all reduction factors.

The mechanism-specific reduction factors include the following:

RF_{IMCO} = reduction factor for immediate compression, i.e. decrease of hydraulic transmissivity due to compression of the transmissive core following immediately the application of stress;
RF_{IMIN} = reduction factor for immediate intrusion, i.e. decrease of hydraulic transmissivity due to geotextile intrusion into the transmissive core following immediately the application of stress;
$\boldsymbol{RF_{CR}}$ = reduction factor for creep, i.e. time-dependent hydraulic transmissivity reduction due to creep of the transmissive core under the applied stress;
$\boldsymbol{RF_{IN}}$ = reduction factor for delayed intrusion, i.e. decrease of hydraulic transmissivity over time due to geotextile intrusion into the transmissive core resulting from time-dependent deformation of the geotextile;
RF_{CD} = reduction factor for chemical degradation, i.e. decrease of hydraulic transmissivity due to chemical degradation of the polymeric compound(s) used to make the geocomposite;
RF_{PC} = reduction factor for particulate clogging, i.e. decrease of hydraulic transmissivity due to clogging by particles migrating into the transmissive core;
$\boldsymbol{RF_{CC}}$ = reduction factor for chemical clogging, i.e. decrease of hydraulic transmissivity due to chemical clogging of the transmissive core; and
$\boldsymbol{RF_{BC}}$ = reduction factor for biological clogging, i.e. decrease of hydraulic transmissivity due to biological clogging of the transmissive core.

Note that $\boldsymbol{RF_{IN}}$, $\boldsymbol{RF_{CR}}$, $\boldsymbol{RF_{CC}}$, and $\boldsymbol{RF_{BC}}$ were previously discussed by Koerner (1998).

Each reduction factor corresponds to a mechanism that reduces the hydraulic transmissivity of the geocomposite in the field. If one of these mechanisms occurs during the hydraulic transmissivity test in the laboratory to the same extent as in the field, then the corresponding reduction factor is equal to 1.0. It is important to understand that a reduction factor equal to one does not necessarily mean that the related mechanism affecting the hydraulic transmissivity of a virgin material does not exist. It simply means that the effect of this mechanism is already included into $\theta_{measured}$. An ideal hydraulic transmissivity test would perfectly simulate in the laboratory all the mechanisms that reduce the hydraulic transmissivity in the field such that all reduction factors would be equal to 1.0. However, such a test is not achievable from a practical standpoint, because it would be extremely complex and would require a very long time.

Examination of Eq. 3.8 indicates the following:

- RF_{IMCO} and RF_{IMIN} correspond to instantaneous mechanisms (i.e. mechanisms that take place as soon as the stress is applied), whereas the other reduction factors correspond to time-dependent mechanisms.

- RF_{IMCO}, RF_{IMIN}, RF_{CR}, and RF_{IN}, result from mechanical mechanisms, i.e. they are directly related to the applied stress. In contrast, RF_{CD}, RF_{PC}, RF_{CC}, and RF_{BC} result from physico-chemical mechanisms and, as such, they are not directly related to the applied stress.
- The physico-chemical mechanisms do not occur during typical hydraulic transmissivity tests that are performed with pure water. In contrast, the mechanical mechanisms may occur during the hydraulic transmissivity test, which affects the magnitude of RF_{IMCO}, RF_{IMIN}, RF_{CR}, and RF_{IN}, as discussed below.

It is important to note that the four reduction factors that result from mechanical mechanisms depend on the conditions under which the hydraulic transmissivity is measured. These conditions include: the stress applied during the hydraulic transmissivity test, the time during which the stress is applied before the flow rate (from which the hydraulic transmissivity is derived) is measured (the "seating time"), and the nature and behavior of the boundary materials in contact with the drainage composite during the transmissivity test. From this viewpoint, the following comments can be made:

- RF_{IMCO} can be eliminated (i.e. $RF_{IMCO} = 1.0$) if the hydraulic transmissivity is measured after a stress equal to, or greater than, the stress in the soil is applied to the specimen of transmissive material subjected to the hydraulic transmissivity test.
- RF_{IMIN} can be eliminated (i.e. $RF_{IMIN} = 1.0$) if the hydraulic transmissivity test simulates the boundary conditions created by the presence of materials adjacent to the transmissive material.
- RF_{CR} and RF_{IN} can be decreased if the hydraulic transmissivity is measured after the stress has been applied for a certain period of time (seating time), because part of the creep of the transmissive core and part of the delayed intrusion would have occurred before the hydraulic transmissivity is measured.

An extreme case would occur if the hydraulic transmissivity is measured on the transmissive core placed between two smooth plates, under no load, with pure water (so none of the physico-bio-chemical mechanisms can take place), and during a period of time that is so short that none of the time–dependent mechanisms can develop. In this extreme case, all of the eight reduction factors defined above would have their maximum value. A typical hydraulic transmissivity test is between: (i) the ideal case where all mechanisms are perfectly simulated and, consequently, all reduction factors would be equal to 1.0; and (ii) the extreme case where all of the eight reduction factors would have their maximum value. Two more typical laboratory test conditions are described below.

In the first typical case of test conditions, the transmissive core is placed between two rigid plates and a load is equal to or greater than the design load is sustained for a certain period of time (the seating time). In this case, the instantaneous compression takes place before the hydraulic transmissivity is measured. Therefore, $RF_{IMCO} = 1$. Also, some creep occurs during the seating time. As a result, the value of RF_{CR} is less than in the theoretical case where the hydraulic transmissivity would be measured at time zero. Equation 3.6 then becomes:

$$\theta_{allow} = \frac{\theta_{measured}}{\prod(RF)} = \frac{\theta_{measured}}{RF_{IMIN} \times RF_{CR} \times RF_{IN} \times RF_{CD} \times RF_{PC} \times RF_{CC} \times RF_{BC}} \qquad \text{Eq. 3.7}$$

[change allow to LTIS in equation]

Seating times of 100 or 300 hours are often recommended in the United States (Holtz et al. 1997). During such seating times, a significant amount of creep takes place. As a result, RF_{CR} is significantly less than it would be if the seating time were short. Also, it is likely that RF_{IN} is significantly less than it would be if the seating time were short.

In the second typical case of test conditions, the boundary conditions created by the presence of adjacent materials are simulated. To that end, the geocomposite is placed between two materials (soil or geosynthetic) that are identical to, or that simulate, the materials that are adjacent to the considered geocomposite in the field, and the sustained load is equal to or greater than the design load. Therefore, $RF_{IMCO} = 1$ and $RF_{IMIN} = 1$. Also, some creep and some time-dependent intrusion of geotextile into the transmissive core channels occur during the seating time. As a result, the values of RF_{CR} and RF_{IN} are less than in the theoretical case where the hydraulic transmissivity would be measured at time zero. Equation 3.6 then becomes:

$$\theta_{LTIS} = \frac{\theta_{measured}}{\prod(RF)} = \frac{\theta_{measured}}{RF_{CR} \times RF_{IN} \times RF_{CD} \times RF_{PC} \times RF_{CC} \times RF_{BC}} \qquad \text{Eq. 3.8}$$

The determination of RF_{CR}, RF_{IN}, RF_{CD}, RF_{PC}, RF_{CC} and RF_{BC} requires long-duration tests. Due to lack of time, such tests cannot be performed for the design of a specific project. Therefore, values obtained from **Tables** 3.1 or 3.3, from the literature, or from the geosynthetic manufacturer should be used. **Table** 3.3 provides guidance regarding the values of the reduction factors for geonets and geocomposites having geonets as the transmissive core (which are the most frequently used geosynthetic liquid collection layers). However, the design engineer is cautioned that the values of the reduction factors may significantly vary depending on the type of geocomposite and the exposure conditions (stress, chemical composition of the soil and liquid). Also, as pointed out above, the values of some of the time-dependent reduction factors (e.g. RF_{CR} and RF_{IN}) may significantly vary depending on the conditions under which the hydraulic transmissivity is measured. The values given in **Table** 3.3 correspond to the case where the seating time is of the order of 100 hours or more and the boundary conditions due to adjacent materials are simulated in the hydraulic transmissivity test.

Table 3.3. Guidance for the selection of some of the reduction factors on the flow capacity of geonets and geocomposites having a geonet transmissive core.

Examples of application	Normal stress	Liquid	RF_{IN}	RF_{CR}	RF_{CC}	RF_{BC}
Landfill cover drainage layer, Low retaining wall drainage	Low	Water	1.0 – 1.2	1.1 – 1.4	1.0 – 1.2	1.2 – 1.5
Embankment, Dams, Landslide repair, High retaining wall drainage	High	Water	1.0 – 1.2	1.4 – 2.0	1.0 – 1.2	1.2 – 1.5
Landfill leachate collection layer, Landfill leakage collection and detection layer, Leachate pond leakage collection and detection layer	High	Leachate	1.0 – 1.2	1.4 – 2.0	1.5 – 2.0	1.5 – 2.0

Notes: Table 3.3 was developed using reduction factor values from Koerner (1998). Design engineers are cautioned that the values of the reduction factors may significantly vary depending on the type of geocomposite and the exposure conditions (stress, chemical composition of the soil and liquid). Also, RF_{IN} and RF_{CR} depend on the testing conditions under which the hydraulic transmissivity is measured. The reduction factor values given in **Table** 3.3 correspond to the case where the seating time is of the order of 100 hours or more and the boundary conditions due to adjacent materials are simulated in the hydraulic transmissivity test. Finally, due to lack of relevant data, no guidance is provided for RF_{CD} and RF_{PC}.

Also, it should be noted that RF_{CR}, RF_{CD}, RF_{CC} and RF_{BC} (and, to a lesser degree, RF_{IN} and RF_{PC}) correspond to time-dependent mechanisms. Therefore, the values of RF_{CR}, RF_{CD}, RF_{CC} and RF_{BC} (and, to a lesser degree, RF_{IN} and RF_{PC}) selected by the design engineer depend on the design life of the liquid collection layer. In cases where the liquid supply rate varies with time, the design engineer may consider several time periods. For example, in the case of landfills with no leachate recirculation, three phases may be considered: (i) construction and pre-operational phase; (ii) operational phase; and (iii) post-closure phase. As time elapses, the

leachate collection system will typically experience a reduction in the rate of leachate that needs to be collected, but may concurrently experience a reduction of its flow capacity due to time-dependent mechanism such as creep and clogging.

The above discussion is for geocomposites, in particular, for geocomposites whose transmissive core is a geonet (which are the most frequently used geosynthetic liquid collection layers). In the case where the geosynthetic liquid collection layer is a thick needle-punched nonwoven geotextile, the mechanisms described above exist with the exception of geotextile intrusion into the transmissive core since, in this case, the geotextile itself is the transmissive medium. In this case, the reduction factors presented above exist, but no guidance is proposed herein regarding their values.

Finally, it should be noted that the various reduction factors may not be completely independent. For example, chemical degradation may affect creep resistance (i.e. may increase RF_{CR}), and, as shown by Palmeira and Gardoni (2000), the presence of soil particles in a needle-punched nonwoven geotextile (i.e. particulate clogging) may reduce the geotextile's compressibility (i.e. it may reduce RF_{IMCO} and RF_{CR} while increasing RF_{PC}).

In the absence of site specific testing data, the authors recommend the upper limits of the above default values for landfill covers, average default values for leachate collection systems, and lower limits for leakage detection systems. This reflects the service life of the final cover, the potential for significant compressive creep or intrusion of the leachate collection systems, the large quantity of leachate to be handled by the leachate collection system, and the expected lower level of intrusion and leachate volume to be conveyed by the leakage detection system. When a design drainage safety factor of 2 is used, the total default long-term service reduction factor (including the reduction factors) suggested (Richardson and Zhao, 1998) is as follows:

- **6 for landfill closures** (design drainage safety factor (2), intrusion (1.2), creep (1.4), biological clogging (1.2), chemical clogging (1.5), i.e., 2*1.2*1.4*1.2*1.5 = 6.0);
- **20 for leachate collection systems** (2*1.2*2.0*2.0*2.0 = 19.6);
- **20 for leakage detection systems** (2*1.2*2.0*2.0*2.0 = 19.6).

Thus

$\theta_{ultimate}$ =6*3*10^{-5} m^3/sec-m =1.8*10^{-4} m^3/sec-m for cover drains
$\theta_{ultimate}$ =20*3*10^{-5} m^3/sec-m = 6 *10^{-4} m^3/sec-m for leachate collection drains.
$\theta_{ultimate}$ =10*3*10^{-5} m^3/sec-m = 6 *10^{-4} m^3/sec-m for leakage collection drains.

3.3 Laboratory Long-Term Transmissivity Evaluation Base On Site Conditions

As discussed above, an ideal transmissivity test would perfectly simulate all the field mechanisms that reduce the transmissivity during the service life such that all of the reduction factors would be equal to 1.0. Unfortunately, this is not possible. This means that the designer must clearly understand to what degree each of the reduction factors has been simulated in the laboratory transmissivity test. Information of such considerations is currently not incorporated in ASTM D-4716. This section discusses what information is currently known regarding such considerations.

3.3.1 Geotextile Intrusion, RF_{IN}

The transmissivity value must be obtained under normal loads exceeding the field-anticipated long-term

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Rates of Leakage Through Landfill Liners

ABSTRACT

This paper describes methods for evaluating rates of leakage through landfill liners constructed with geomembranes. The paper addresses both geomembranes alone and geomembranes used in composite liners. Leakage through liners constructed with geomembranes can occur by fluid permeation through intact geomembranes and flow through geomembrane holes. Only leakage through geomembrane holes is considered in the paper. Leakage through a geomembrane hole is dependent on the hydraulic conductivities of the materials overlying and underlying the geomembrane. Three cases of leakage are considered: (i) leakage through geomembranes alone; (ii) leakage through composite liners; and (iii) leakage through geomembranes overlain by a drainage layer that impedes flow toward the geomembrane hole. A comparison of the leakage rates for these three cases shows that leakage through a hole in the geomembrane component of composite liner can be up to 100,000 times smaller than leakage through a hole in a geomembrane alone. It is also shown that the presence of sand overlying a geomembrane hole can reduce the rate of leakage through the hole by a factor of up to 50 compared to the case of a geomembrane alone.

INTRODUCTION

All hazardous waste landfills in the United States and an increasing number of municipal solid waste landfills are constructed with double liner systems. The lining systems of these landfills include the following four elements, from top to bottom: a leachate collection layer; a primary liner; a leakage detection and collection layer; and a secondary liner. In this paper, the leachate collection layer and the leakage detection and collection layer are generically called drainage layers.

This paper discusses the evaluation of the rate of leakage through the primary and secondary liners.

The primary or secondary liner can be a geomembrane or a composite liner, i.e., a liner composed of a geomembrane placed on a layer of low-permeability soil (i.e., a soil with a hydraulic conductivity less than 10^{-6} m/s (10^{-4} cm/s) and usually in the range of 10^{-8} to 10^{-10} m/s (10^{-6} to 10^{-8} cm/s)). Soil liners alone are not considered.

The leachate collection layer and the leakage detection and collection layer can be constructed with a variety of drainage materials. Some have high permeabilities, such as geonets and coarse gravels; some have medium permeabilities, such as sands and fine gravels. Typical hydraulic conductivities of drainage materials are: 10^{-1} to 1 m/s (10 to 100 cm/s) for coarse gravel; 10^{-1} m/s (10 cm/s) for geonets; 10^{-2} m/s (1 cm/s) for fine gravel; and 10^{-5} to 10^{-3} m/s (10^{-3} to 10^{-1} cm/s) for sand. The influence of the hydraulic conductivity of the drainage material on the leakage rate will be evaluated.

LEAKAGE MECHANISMS

There are essentially two mechanisms of leakage through geomembranes: fluid permeation through an intact geomembrane and flow through geomembrane holes.

Leakage due to permeation is not considered in this paper because, for landfills, leakage rates due to fluid permeation are usually much smaller than leakage rates due to flow through geomembrane holes. A review of this subject is presented in Giroud and Bonaparte (7).

Regarding leakage through geomembrane holes, several cases can be considered:

- If a geomembrane with a hole is overlain and underlain by high-permeability materials (such as geonet or coarse gravel), flow through the hole is not significantly impeded. Therefore, for this case, the flow of liquid can be considered as free flow through an orifice and the leakage rate is essentially governed by the size of the hole.

- If a geomembrane with a hole is placed on a layer of low-permeability soil (such as clay, silt, clayey soil, etc.) to form a composite liner, the low-permeability soil significantly impedes the flow of liquid through the hole, provided that the geomembrane is in close contact with the low-permeability soil.

- If a geomembrane with a hole, and placed on a high-permeability material, is overlain by a sand or a fine gravel, flow through the hole may be somewhat impeded, so that the rate of leakage through the hole is lower than in the case where the geomembrane is overlain or underlain by a high-permeability material, but higher than in the case where the geomembrane is placed on a low-permeability soil to form a composite liner.

These three cases will be discussed below, and equations to evaluate leakage rates will be presented.

RATE OF LEAKAGE DUE TO DEFECTS IN GEOMEMBRANES ALONE

In the context of this paper, a geomembrane alone is a geomembrane overlain and underlain by high-permeability materials (such as geonets or coarse gravels). In this case, unless the hole is a slit with a width less than the thickness of the geomembrane or a pinhole with a diameter less than the thickness of the geomembrane, Bernoulli's equation for free flow through an orifice can be used to evaluate the leakage rate (Giroud, 6):

$$Q = C_B \, a \sqrt{2gh} \qquad (1)$$

where: Q = steady-state rate of leakage through one geomembrane hole; a = area of the hole in the geomembrane; g = acceleration of gravity; and h = head of liquid on top of the geomembrane. C_B is a dimensionless coefficient, valid for any Newtonian fluid, related to the shape of the edges of the aperture; for sharp edges, which is assumed to be the case for geomembrane holes, C_B = 0.6. Basic SI units are: Q (m^3/s), a (m^2), g (m/s^2), and h (m). As discussed subsequently, and shown in Figure 1, Equation 1 can be used if the soil underlying the geomembrane has a hydraulic conductivity greater than 10^{-3} m/s (10^{-1} cm/s) when the geomembrane hole area is 0.1 cm^2 (0.016 in^2) and greater than 10^{-2} m/s (1 cm/s) when the geomembrane hole area is 1 cm^2 (0.16 in^2).

RATE OF LEAKAGE THROUGH A COMPOSITE LINER

The mechanism of leakage through a composite liner with a hole in the geomembrane is as follows: the liquid first migrates through the hole in the geomembrane; the liquid may then travel laterally some distance in the space, if any, between the geomembrane and the low-permeability soil; finally, the liquid migrates into and eventually through the low-permeability soil.

There may be no space between the geomembrane and soil components of a composite liner if the geomembrane is sprayed directly onto the low-permeability soil layer. This technique is not very often used and, in the more usual case of a geomembrane manufactured in a plant, there will be some space between the geomembrane and soil components of a composite liner in almost all applications because:

- the geomembrane has wrinkles (note that geomembrane wrinkles may exist even under very high pressures as shown by Stone (10));
- there are clods or irregularities in the underlying soil surface; and/or
- even when the underlying soil surface is smooth, the geomembrane bridges small spaces between soil particles.

Laboratory test results discussed by Giroud and Bonaparte (7) seem to indicate that some lateral flow almost always occurs between the geomembrane and the underlying soil, even under laboratory test conditions where the geomembrane is placed as flat as possible on a soil layer that has a smooth surface.

In order to establish a method for evaluating the rate of leakage through composite liners with a hole in the geomembrane, Giroud and Bonaparte (7) have made a thorough review of the results of composite liner model tests conducted by Fukuoka (4, 5) and Brown et al. (1), and theoretical analyses carried out by Faure (2, 3), Sherard (9), Fukuoka (5), and Brown et al. (1). Giroud and Bonaparte (7) indicate that a key factor influencing the rate of leakage through a composite liner is the quality of contact between the geomembrane and low-permeability soil components of the composite liner. They ranked the experimental and theoretical results they reviewed as a function of the contact quality from a lower bound, corresponding to the theoretical case of perfect contact, to an upper bound, corresponding to no contact at all. They also proposed a method of interpolation between the various experimental and theoretical results. This method is described in detail in two publications [USEPA (11); Giroud and Bonaparte (7)]. Subsequently, Giroud et al. (8), using the proposed interpolation method, established the following empirical equations:

$$Q = 0.21\ a^{0.1}\ h^{0.9}\ k_s^{0.74} \quad \text{for good contact} \tag{2}$$

$$Q = 1.15\ a^{0.1}\ h^{0.9}\ k_s^{0.74} \quad \text{for poor contact} \tag{3}$$

where: Q = steady-state rate of leakage through one hole in the geomembrane component of a composite liner; a = area of the hole in the geomembrane; h = head of liquid on top of the geomembrane; and k_s = hydraulic conductivity of the low-permeability soil underlying the geomembrane. Equations 2 and 3 are not dimensionally homogeneous; they can only be used with the following units: $Q(m^3/s)$, $a(m^2)$, $h(m)$, and $k_s(m/s)$.

The experimental data used to empirically establish Equations 2 and 3 suggest that the use of these equations should be restricted to cases where the hydraulic conductivity of the low-permeability soil is less than 10^{-6} m/s (10^{-4} cm/s). The theoretical analyses used to empirically establish Equations 2 and 3 also suggest that the use of these equations should be restricted to cases where the head of liquid on top of the geomembrane is less than the thickness of the low-permeability soil layer underlying the geomembrane. If this condition is fulfilled, the leakage rate does not significantly depend on the thickness of the low-permeability soil layer. This is why Equations 2 and 3 do not show a functional dependence of leakage rate on the thickness of the low-permeability soil layer.

The good and poor contact conditions are defined as follows:

- The good contact condition corresponds to a geomembrane installed with as few wrinkles as possible, on top of a low-permeability soil layer that has been adequately compacted and has a smooth surface.

- The poor contact condition corresponds to a geomembrane that has been installed with a certain number of wrinkles, and/or placed on a low-permeability soil that has not been well compacted and does not appear smooth.

These two contact conditions can be considered as typical field conditions. They are bounded by two extreme field conditions, the best case and the worst case, which can be defined as follows:

- In the best case: (i) the soil is well compacted, flat and smooth, has not been deformed by rutting due to construction equipment, and has no clods nor cracks; (ii) the geomembrane is flexible and has no wrinkles; and (iii) the geomembrane and soil are in close contact.

- In the worst case: (i) the soil is poorly compacted, has an irregular surface, and is cracked; and (ii) the geomembrane is stiff and exhibits a pattern of large, connected wrinkles.

RATE OF LEAKAGE THROUGH A GEOMEMBRANE OVERLAIN BY A DRAINAGE MATERIAL

As indicated above, high-permeability materials (such as geonets and coarse gravels) located above or below a geomembrane are not expected to significantly affect the flow of liquid through a hole in the geomembrane, and the flow rate is approximately the same as in the case of free flow through the hole. On the other hand, if a geomembrane resting on a high-permeability material (such as geonet or

coarse gravel) is overlain by a medium-permeability drainage material (such as sand or fine gravel), the flow toward the geomembrane hole is impeded by the drainage material, and the flow rate is less than in the case of free flow. A typical field situation is a geomembrane primary liner overlain by a sand leachate collection layer and underlain by a geonet leakage detection and collection layer.

In order to evaluate the leakage rate reduction due to the presence of the drainage material above the geomembrane, the following rationale has been used:

- When sand or fine gravel is placed above a geomembrane, excellent contact is expected between the sand or gravel and the geomembrane, because sand and gravel are cohesionless, and they follow the shape of the geomembrane, even if it exhibits wrinkles.

- However, even if the contact between a granular material and a flat boundary such as a geomembrane seems perfect, there is usually a preferential flow path in the granular material next to the boundary, because the porosity of a granular material in the vicinity of a flat boundary is usually greater than the porosity within the material.

- From the above, it appears that a lower bound solution for the leakage rate is provided by the equation for radial flow towards the geomembrane hole, since this equation corresponds approximately to the case of perfect contact without preferential flow, according to Giroud and Bonaparte (7).

- An obvious upper bound solution for the leakage rate is provided by Bernoulli's equation for free flow (Equation 1).

- Approximate theoretical evaluations of the rate of flow in the zone of greater porosity of the granular drainage material in the vicinity of the geomembrane were made by the authors. Using these approximate evaluations as a guide, several empirical approaches were attempted. It was found that a satisfactory approximate value for the leakage rate could be obtained by averaging the logarithms of the leakage rates obtained with the lower bound and upper bound solutions mentioned above.

The empirical equation thus obtained is:

$$Q = 3\ a^{0.75}\ h^{0.75}\ k_d^{0.5} \qquad (4)$$

where: Q = steady-state rate of leakage through one geomembrane hole; a = area of the hole in the geomembrane; h = head of liquid on top of the geomembrane; k_d = hydraulic conductivity of the drainage material overlying the geomembrane. Equation 4 is not dimensionally homogeneous; it can only be used with the following units: $Q(m^3/s)$, $a(m^2)$, $h(m)$, and $k_d(m/s)$.

Equation 4 is intended only for the case of granular drainage materials and, therefore, should only be used when the hydraulic conductivity of the drainage layer material is greater than 10^{-6} m/s (10^{-4} cm/s). Also, some of the analyses used to establish Equation 4 suggest that use of the equation should be limited to cases where the head of liquid on top of the geomembrane, h, is less than the thickness of the drainage layer; this condition is usually fulfilled in the case of landfills.

COMPARISON OF LEAKAGE RATES

Two tables were established to compare leakage rates through a geomembrane alone, a composite liner, and a geomembrane overlain by a drainage layer.

Equations 1 and 2 were used to establish Table 1, which compares the rates of leakage through a hole in the geomembrane component of a composite liner with those through a hole in a geomembrane alone, i.e., a geomembrane overlain and underlain by high-permeability materials (such as geonets or coarse gravels). This table was established assuming that the head of liquid above the geomembrane hole is constant (i.e., that there is no drawdown of liquid over the hole). This table shows that there is great benefit in using composite liners. For example, in the case of a small hole (i.e., 0.1 cm^2 = 0.016 in^2), it appears that the ratio between the rates of leakage through a hole in geomembrane alone and a composite liner are, for the case of "good contact", in the following ranges:

- 25,000 to 60,000 if $k_s = 10^{-10}$ m/s (10^{-8} cm/s)
- 5,000 to 10,000 if $k_s = 10^{-9}$ m/s (10^{-7} cm/s)
- 800 to 2,000 if $k_s = 10^{-8}$ m/s (10^{-6} cm/s)
- 150 to 400 if $k_s = 10^{-7}$ m/s (10^{-5} cm/s)
- 30 to 70 if $k_s = 10^{-6}$ m/s (10^{-4} cm/s),

where k_s is the hydraulic conductivity of the low-permeability soil component of the composite liner. In each range, the lower value is for a head of liquid on top of the geomembrane of 0.1 m (4 in.) and the higher value is for 0.01 m (0.4 in.). The beneficial effect of the composite liner is slightly greater if the hole size is greater than the considered 0.1 cm^2 (0.016 in^2).

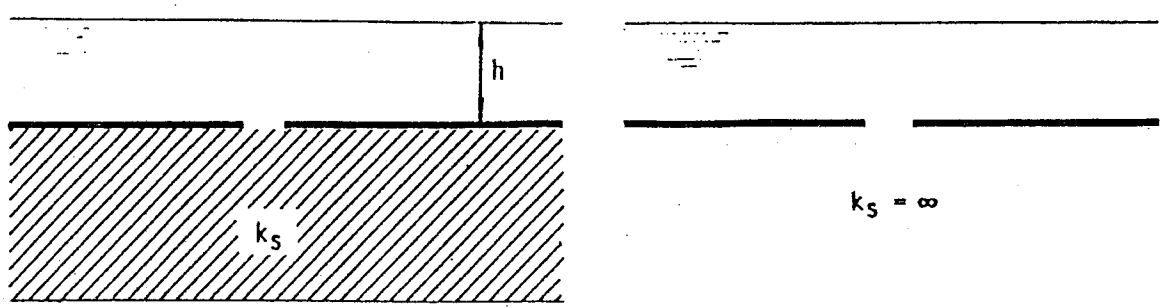
Equations 1 and 4 were used to establish Table 2, which compares the rates of leakage through a hole in: (i) a geomembrane overlain by a sand or fine gravel and underlain by a high-permeability material such as a geonet or coarse gravel; and (ii) a geomembrane alone, i.e., a geomembrane overlain and underlain by high-permeability materials. This table shows that the drainage material overlying the geomembrane can have a significant influence on the rate of leakage through a hole in the geomembrane. For example, in the case of a small hole (e.g., 0.1 cm^2 = 0.016 in^2), it appears that the ratios between the rates of leakage through a hole in a geomembrane alone and a hole in a geomembrane overlain by a sand or fine gravel are in the following ranges:

- 30 to 50 if $k_d = 10^{-5}$ m/s (10^{-3} cm/s)
- 10 to 15 if $k_d = 10^{-4}$ m/s (10^{-2} cm/s)
- 3 to 5 if $k_d = 10^{-3}$ m/s (10^{-1} cm/s),

where k_d is the hydraulic conductivity of the drainage layer material overlying the geomembrane. In each range, the lower value is for a head of liquid on top of the geomembrane of 0.1 m (4 in.) and the higher value is for 0.01 m (0.4 in.). The effect of drainage materials having a hydraulic conductivity equal to or greater than 10^{-2} m/s (1 cm/s) is negligible.

Table 1. *Ratio of Leakage Rates Between Composite Liner and Geomembrane Alone.* This table was obtained by dividing Equation 2 (composite liner with good contact) by Equation 1 (geomembrane alone, i.e., geomembrane overlain and underlain by a high-permeability material).

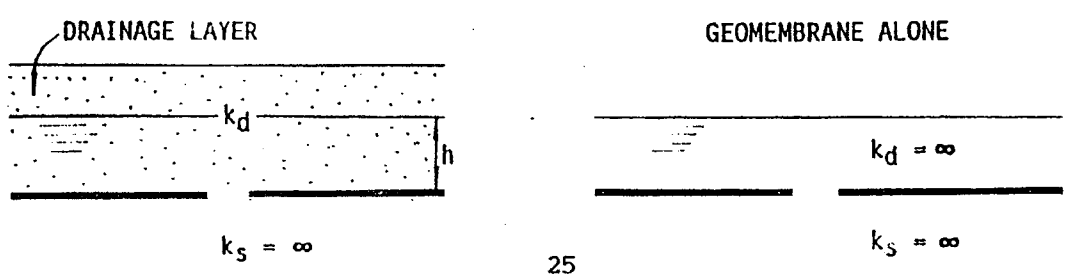
Hole Size	Hydraulic Conductivity of Low-Permeability Soil, k_s m/s (cm/s)	Depth of Liquid, h (m (in.))			
		0.001 (0.04)	0.01 (0.4)	0.1 (4)	0.3 (12)
1 cm^2 (0.16 in^2)	10^{-10} (10^{-8})	7.9×10^{-7}	2.0×10^{-6}	5.0×10^{-6}	7.7×10^{-6}
	10^{-9} (10^{-7})	4.3×10^{-6}	1.1×10^{-5}	2.7×10^{-5}	4.3×10^{-5}
	10^{-8} (10^{-6})	2.4×10^{-5}	6.0×10^{-5}	1.5×10^{-4}	2.3×10^{-4}
	10^{-7} (10^{-5})	1.3×10^{-4}	3.3×10^{-4}	8.3×10^{-4}	1.3×10^{-3}
	10^{-6} (10^{-4})	7.2×10^{-4}	1.8×10^{-3}	4.6×10^{-3}	7.1×10^{-3}
0.1 cm^2 (0.016 in^2)	10^{-10} (10^{-8})	6.3×10^{-6}	1.6×10^{-5}	4.0×10^{-5}	6.1×10^{-5}
	10^{-9} (10^{-7})	3.4×10^{-5}	8.7×10^{-5}	2.2×10^{-4}	3.4×10^{-4}
	10^{-8} (10^{-6})	1.9×10^{-5}	4.8×10^{-4}	1.2×10^{-3}	1.9×10^{-3}
	10^{-7} (10^{-5})	1.0×10^{-3}	2.6×10^{-3}	6.6×10^{-3}	1.0×10^{-2}
	10^{-6} (10^{-4})	5.7×10^{-3}	1.4×10^{-2}	3.6×10^{-2}	5.6×10^{-2}
		Ratio of Leakage Rates Between Composite Liner and Geomembrane Alone			



COMPOSITE LINER GEOMEMBRANE ALONE

Table 2. *Effect on Leakage Rate of the Drainage Layer Overlying the Geomembrane.* This table was obtained by dividing Equation 4 by Equation 1.

Hole Size	Hydraulic Conductivity of Drainage Material, k_d m/s (cm/s)	Depth of Leachate, h (m (in.)) 0.001 (0.04)	0.01 (0.4)	0.1 (4)	0.3 (12)
1 cm^2 (0.16 in^2)	10^{-5} (10^{-3})	0.006	0.012	0.021	0.027
	10^{-4} (10^{-2})	0.021	0.037	0.065	0.085
	10^{-3} (10^{-1})	0.065	0.115	0.205	0.270
	10^{-2} (1)	0.205	0.365	0.649	0.855
	10^{-1} (10)	0.649	>1	>1	>1
	1 (100)	>1	>1	>1	>1
6.1 cm^2 ~~1~~ cm^2 (0.016 in^2)	10^{-5} (10^{-3})	0.012	0.021	0.037	0.048
	10^{-4} (10^{-2})	0.037	0.065	0.115	0.152
	10^{-3} (10^{-1})	0.115	0.205	0.365	0.481
	10^{-2} (1)	0.365	0.649	>1	>1
	10^{-1} (10)	>1	>1	>1	>1
	1 (100)	>1	>1	>1	>1
		Ratio of Leakage Rates Between Geomembrane Overlain by a Drainage Material and Geomembrane Alone			



It therefore appears that placing soil below or above a geomembrane significantly decreases the leakage rate through a hole in the geomembrane. However, the two beneficial effects should not be added to each other. For example, if a geomembrane is underlain by clay and overlain by sand, the beneficial effect of the sand cannot be added to the beneficial effect of the clay. The rationale is as follows: the hydraulic conductivity of clay is much lower than that of sand and, therefore, it controls the velocity of liquid flow through the geomembrane hole; the presence of sand does not have any noticeable influence on flow velocity and, consequently, on leakage rate.

RATE OF LEAKAGE THROUGH A QUASI-COMPOSITE LINER

There are many practical situations where clay or clayey soils are not available to construct a composite liner and where a geomembrane is placed on a layer of sandy or silty soil (either the natural subgrade or a compacted layer of bedding soil) with a hydraulic conductivity in the range of 10^{-6} - 10^{-4} m/s (10^{-4} - 10^{-2} cm/s). Although this is not as good as a composite liner where the low-permeability soil component has a hydraulic conductivity less than 10^{-6} m/s (10^{-4} cm/s), the presence of the sandy or silty soil under the geomembrane is likely to decrease the leakage rate through a geomembrane hole compared to the case of a geomembrane placed on a highly pervious soil. The association of a geomembrane and a medium-permeability soil can be called a quasi-composite liner.

At the present time, to the best of our knowledge, there is no method to evaluate the rate of leakage through a quasi-composite liner due to a hole in the geomembrane. Equations 2 and 3 are valid only if the soil component of the composite liner has a hydraulic conductivity less than 10^{-6} m/s (10^{-4} cm/s). If Equation 2 or 3 are used with a value of the hydraulic conductivity greater than 10^{-6} m/s (10^{-4} cm/s), the equations overestimate the leakage rate because they exaggerate the influence of lateral flow between the geomembrane and soil. (Lateral flow is expected to be very small when the soil underlying the geomembrane does not have a low hydraulic conductivity.)

Another approach to calculating the leakage rate through a hole in the geomembrane component of a quasi-composite liner would be to use Equation 4, which was developed to evaluate the effect of the overlying material, with k_s substituted for k_d. This would tend to underestimate the leakage rate because Equation 4 was established assuming excellent contact between the overlying granular material and the geomembrane, while the contact quality may not be as good when the granular soil is below the geomembrane.

It may therefore be concluded that Equation 2 provides an upper bound solution and Equation 4 (with k_s instead of k_d) a lower bound solution for the case of a quasi-composite liner.

It is interesting to use both equations to determine the hydraulic conductivity of the underlying soil for which the leakage rate is the same as in the case of free flow (Equation 1). For a 0.1 cm^2 (0.016 in^2) hole, this occurs at k_s approximately equal to 10^{-4} m/s (10^{-2} cm/s) with Equation 2, and k_s approximately equal to 10^{-2} m/s (1 cm/s) with Equation 4. By interpolating between these two values, it can arbitrarily be concluded that free flow occurs when the hydraulic conductivity of the material underlying the geomembrane is on the order of 10^{-3} m/s (10^{-1} cm/s) or greater. This value is consistent with the results of some tests by Brown et al. (1).

To evaluate the beneficial effect of a quasi-composite liner between $k_s = 10^{-6}$ m/s (10^{-4} cm/s) and $k_s = 10^{-3}$ m/s (10^{-1} cm/s), interpolation on a logarithmic scale is suggested, as shown in Figure 1. This figure also illustrates the beneficial effect of composite and quasi-composite liners in the case of a geomembrane hole with an area of 1 cm^2 (0.16 in^2).

For practical applications, Figure 1 can be used for a rapid evaluation of the beneficial effect of a composite liner, whether it is a true composite liner ($k_s < 10^{-6}$ m/s (10^{-4} cm/s)) or a quasi-composite liner ($k_s \geq 10^{-6}$ m/s (10^{-4} m/s)). Figure 1 is based on good contact between the geomembrane and soil components of the composite liner.

RELATIONSHIP BETWEEN LEAKAGE AND LEACHATE GENERATION

The above discussion on the effect of the drainage material overlying the geomembrane on the rate of leakage through a hole in a geomembrane liner could lead to the belief that it is preferable to use sand rather than a more permeable material such as geonet or gravel to construct leachate collection layers. For equal heads of leachate on the geomembrane, the rate of leakage through the primary liner is indeed smaller if the leachate collection layer material is sand rather than gravel, because sand will impede the flow of leachate towards the geomembrane hole, thereby reducing the leakage rate as compared to the case where the leachate collection layer material has a higher permeability than sand. However, such a comparison is not correct because, for a constant rate of leachate generation, the larger the hydraulic conductivity of the leachate collection layer material, the smaller the leachate head on the geomembrane. Therefore, to make a fair comparison between leachate collection layer materials regarding their influence on leakage rate, it is necessary to consider a given rate of leachate generation instead of a given head of leachate on the geomembrane. The authors are currently investigating the combined influence of drainage layer permeability and leachate generation rate on leakage rates through geomembrane holes.

LIMITATIONS

The methods of evaluating rates of leakage through geomembrane holes presented above are based on theoretical analyses and a limited number of laboratory tests. These methods still need to be compared to leakage rates measured in additional laboratory tests and in actual landfills that have reliable leakage detection and collection systems. To date, only very limited data are available and interpretation is always difficult because: (i) the sizes of geomembrane defects are not known; (ii) liquid heads acting on the primary liners are not known; and (iii) many landfills have liquids in their leakage detection and collection layers from sources other than leakage through the primary liner. Therefore, in the present state of knowledge, the above methods should be used with caution and only by experienced engineers.

CONCLUSIONS

This paper has described methods for evaluating rates of leakage through landfill liners constructed with geomembranes. The methods were applied to both geomembranes alone and to composite liners comprised of a geomembrane upper component and a soil lower component. Comparisons of the results of the evaluations demonstrates the effectiveness of composite liners. Table 1 shows that leakage rates through a geomembrane hole are significantly reduced by placing

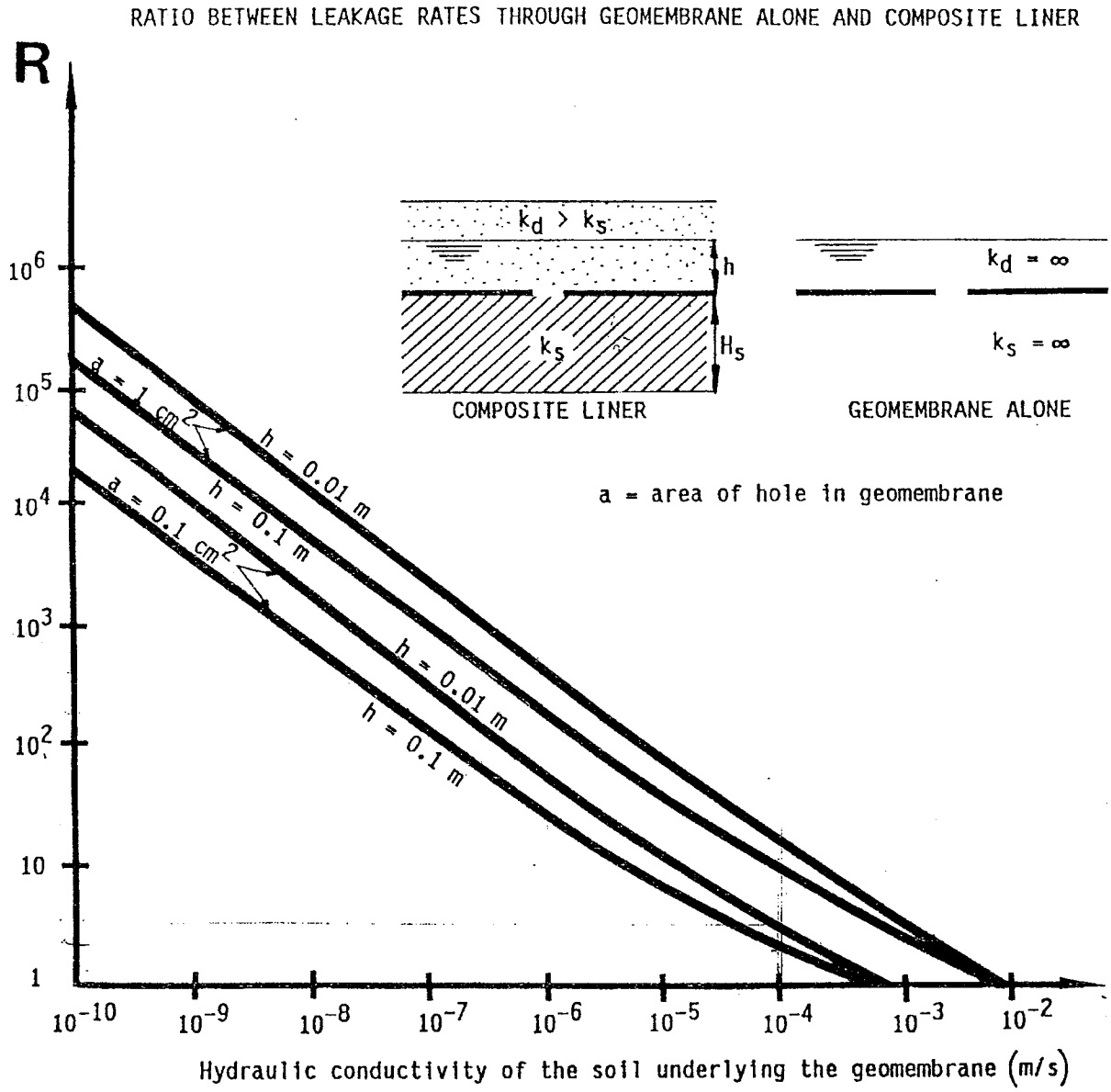


Figure 1. *Effectiveness of a Composite Liner.* This figure gives the ratio, R, between the leakage rates (due to a geomembrane hole) through a geomembrane alone and through a composite liner, as a function of the hydraulic conductivity, k_s, of the soil component of the composite liner. The portions of the curves for $k_s < 10^{-6}$ m/s (10^{-4} cm/s) were established by dividing Equation 1 by Equation 2. The portions of the curves for $k_s > 10^{-6}$ m/s were obtained by interpolation between the portions of the curves for $k_s < 10^{-6}$ m/s and the value of k_s for which free flow is expected. The curves are independent of the thickness, H_s, of the soil layer if $h < H_s$.

a layer of low-permeability soil under the geomembrane. The paper also demonstrates (Table 2) that, for a given head on top of a geomembrane, the presence of a layer of sandy or silty soil on top of or beneath the geomembrane significantly impedes the flow rate through a hole in the geomembrane.

The leakage rate evaluations have been combined into a chart (Figure 1) that can be used with Equation 1 to estimate leakage rates through geomembrane holes when the liquid head acting on top of the geomembrane is known or can be estimated. This chart can be used for practical applications in conjunction with guidelines provided by Giroud and Bonaparte (7) on the sizes and frequencies of geomembrane holes that could be encountered in the field.

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Technical Paper by J.P. Giroud, K. Badu-Tweneboah and K.L. Soderman

Comparison of Leachate Flow Through Compacted Clay Liners and Geosynthetic Clay Liners in Landfill Liner Systems

ABSTRACT: The purpose of this paper is to provide an approach for comparing the effectiveness of geosynthetic clay liners (GCLs) and compacted clay liners (CCLs) used in association with geomembranes to form composite liners. Comparing the effectiveness of these two types of composite liners is required in "equivalency demonstrations" intended to demonstrate that a geomembrane-GCL composite liner is equivalent to a conventional geomembrane-CCL liner prescribed by a regulation. In the first part of the paper, the contribution of the geomembrane is ignored and the paper presents analytical evaluations of advective leachate flow through GCLs, low-permeability soil layers (such as CCLs), and two-layer systems including a GCL and a low-permeability soil layer. The analyses presented explain why some calculations typically performed for landfill liner system design or for equivalency demonstrations lead to the paradoxical result that the advective flow of leachate is greater when a GCL is placed on a layer of low-permeability soil than when the GCL is placed on a layer of high-permeability soil. The second part of the paper presents an analytical method for comparing the effectiveness of a composite liner including a GCL and a composite liner including a CCL. This analytical method enables design engineers to compare the effectiveness of various composite liners without neglecting the beneficial effect of the geomembrane. A parametric study presented in the paper shows that neglecting the geomembrane liner in equivalency demonstrations (which is frequently done in the current state of practice) penalizes composite liners that incorporate a GCL.

KEYWORDS: Liner system, Flow rate, Compacted clay liner, Geosynthetic clay liner, CCL-GCL comparison, Composite liner.

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1 INTRODUCTION

1.1 Purpose of this Paper

Equations are available to calculate the rate of liquid migration through a composite liner, due to geomembrane defects, when the liquid head on top of the liner is small compared to the thickness of the low-permeability soil component of the composite liner, whether the defect is small (Giroud et al. 1989) or large (Giroud et al. 1992). Equations are also available for the case where the head of liquid on top of the liner is large compared to the thickness of the low-permeability soil component of the composite liner (Giroud et al. 1992,1994); however, in such a case, graphs are necessary to obtain the value of one of the terms of the equations, which is cumbersome.

In this paper, it is shown that the graphs can be replaced by equations, which leads to an entirely analytical method for the evaluation of the rate of leachate migration through a composite liner, regardless of the head of liquid on top of the liner.

1.2 Composite Liner

A composite liner is a liner that consists of two or more components. In the context of this paper, the term composite liner will be used for liners that consist of two components, a geomembrane and a low-permeability soil, the geomembrane being on top of the low-permeability soil.

The low-permeability soil component of a composite liner is generally either a compacted clay liner (CCL) or a geosynthetic clay liner (GCL). The thickness of a CCL is typically between 0.3 and 1.5 m whereas the thickness of a hydrated GCL depends on the compressive stress applied during hydration and is typically between 5 and 10 mm, i.e. on the order of 100 times less than the thickness of a CCL. The hydraulic conductivity of both CCLs and GCLs depends on the nature of the material, the nature of the liquid, and the applied compressive stress; when the liquid is water or a leachate that does not affect the hydraulic conductivity of clay, including bentonite, the hydraulic conductivity of a CCL is typically between 1×10^{-10} and 1×10^{-9} m/s whereas the hydraulic conductivity of a GCL is typically between 5×10^{-12} and 5×10^{-11} m/s, i.e. on the order of 10 to 100 times less than the hydraulic conductivity of a CCL.

1.3 Liquid Migration Through a Composite Liner

Since an intact geomembrane has an extremely low permeability, most of the liquid migration through a composite liner occurs through geomembrane defects. In this paper, the only mechanism of liquid migration that is considered is flow through geomembrane defects. The liquid considered herein is water or any aqueous solution such as leachate from municipal or hazardous solid waste landfills.

If there is a defect in the geomembrane component of a composite liner, the liquid passes first through the geomembrane defect, then it flows laterally some distance between the geomembrane and the low-permeability soil, and, finally it infiltrates into and through the low-permeability soil layer which is the second component of the composite liner (Figure 1). Flow in the space between the geomembrane and the low-perme-

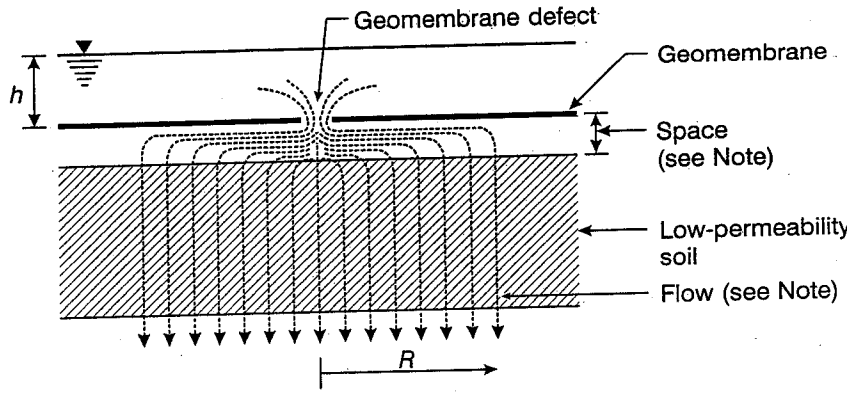

Figure 1. Liquid migration through a composite liner.

Note: The space between the geomembrane and the low-permeability soil is exaggerated to show interfac flow. The flow in the soil is assumed to be vertical and R is the radius of the wetted area.

ability soil is called interface flow, and the area covered by the interface flow is call the wetted area.

The quality of the contact between the two components of a composite liner (i.e. t geomembrane and the low-permeability soil) is one of the key factors governing the ra of flow through the composite liner, because it governs the radius of the wetted ar (Figure 1). *Good* and *poor* contact conditions have been defined by Bonaparte et (1989) as follows:

- *Good* contact conditions correspond to a geomembrane installed, with as f wrinkles as possible, on top of a low-permeability soil layer that has been adequate compacted and has a smooth surface.
- *Poor* contact conditions correspond to a geomembrane that has been installed w a certain number of wrinkles, and/or placed on a low-permeability soil that has been well compacted and does not appear smooth.

For good contact conditions, it is assumed that there is sufficient compressive str to maintain the geomembrane in contact with the low-permeability soil layer. In case of a GCL, good contact conditions may be assumed because GCLs are usua installed flat, and because the bentonite slurry that may exude from a hydrated G contributes to establishing a close contact between the geomembrane and the GCL, p vided sufficient compressive stress is applied.

Other factors affecting the rate of flow through a composite liner are the size of defect, the hydraulic conductivity of the low-permeability soil underlying the geome brane, and the head of liquid on top of the geomembrane. If hydrostatic conditions p vail, the head of liquid on top of the geomembrane is equal to the depth of liquid (Fig 2a) and, if the liquid is unconfined and flowing along a slope (Figure 2b), the head liquid on top of the geomembrane, h, is given by the following equation:

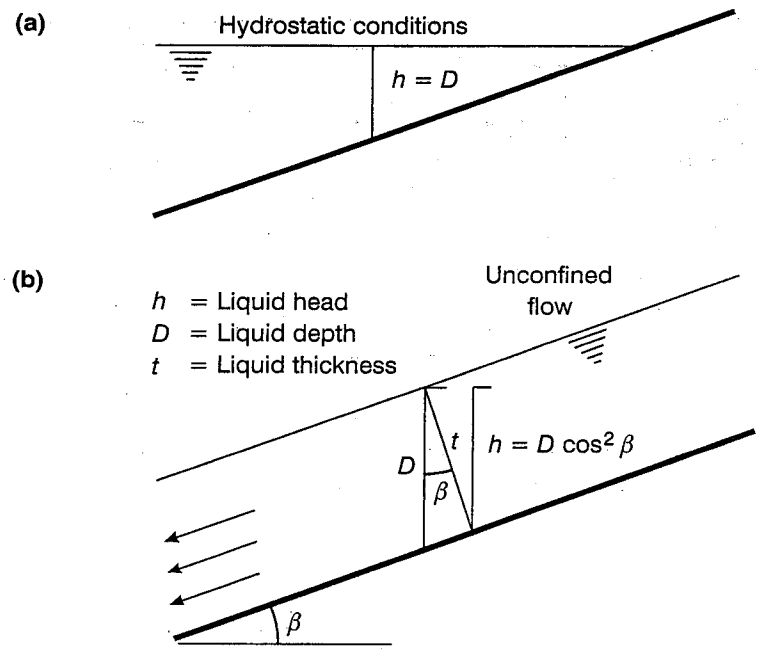

Figure 2. Head of liquid on top of the liner in the case of a liner on a slope: (a) hydrostatic conditions; (b) unconfined flow along the slope.

$$h = t \cos \beta = D \cos^2 \beta \tag{1}$$

where: t = thickness of liquid; D = depth of liquid; and β = slope angle.

1.4 Geomembrane Defects

The following defect shapes are considered in this paper (Figure 3a):

- circular, with a surface area, a, and a diameter, d;
- square, with a side length, b;
- rectangular, with a length, B, and a width, b; and
- infinitely long ($B = \infty$) with a width, b.

1.5 Parameters and Units

The parameters that appear in the liquid migration rate equations are defined in Figure 3b. In the case of a liner on a slope, the liquid head, h, is defined in Figure 2 and by Equation 1.

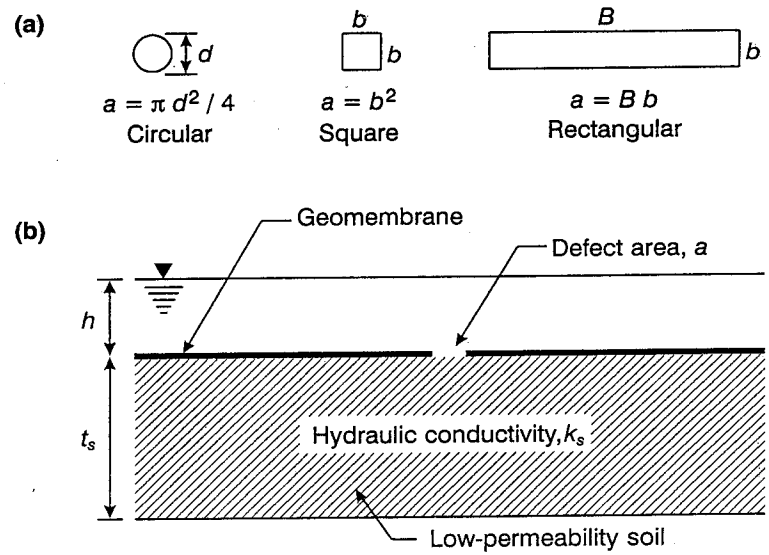

Figure 3. Definition of parameters used in the equations: (a) plan view showing shapes geomembrane defects; (b) cross section.

Note: If the composite liner is on a slope, the liquid head on top of the liner is defined in Figure 2.

In the equations that follow, Q is the rate of liquid migration through the geomer brane defect. When the defect has an infinite length, the equation gives Q^*, which the liquid migration rate per unit length of geomembrane defect.

It is important to note that the equations for liquid migration rate that follow are sem empirical and can only be used with the following basic SI units: h (m), t_s (m), B (m b (m), a (m^2), k_s (m/s), Q (m^3/s), and Q^* (m^2/s). (Note: t_s = thickness of the low-perm ability soil component of the composite liner; k_s = hydraulic conductivity of the lo permeability soil component of the composite liner; and all other symbols were defin above.)

2 EXISTING EQUATIONS TO CALCULATE THE RATE OF LIQUID MIGRATION THROUGH COMPOSITE LINERS

2.1 Equations for Small Head

The following equations have been proposed for the case where the head of liquid top of the liner is less than the thickness of the low-permeability soil component of t composite liner.

Case of a Circular Defect. In the case of a circular defect, the following equation has been established by Giroud et al. (1989):

$$Q = C_{qo}\ a^{0.1}\ h^{0.9}\ k_s^{0.74} \tag{2}$$

hence:

$$Q = 0.976\, C_{qo}\ d^{0.2}\, h^{0.9}\, k_s^{0.74} \tag{3}$$

where C_{qo} is the contact quality factor (dimensionless) for a circular or square hole, with:

$$C_{qogood} \le C_{qo} \le C_{qopoor} \tag{4}$$

where: C_{qogood} = value of C_{qo} in the case of good contact conditions; and C_{qopoor} = value of C_{qo} in the case of poor contact conditions. (The good and poor contact conditions were defined in Section 1.3.) The following values were established by Giroud et al. (1989):

$$C_{qogood} = 0.21 \tag{5}$$

$$C_{qopoor} = 1.15 \tag{6}$$

Case of a Square Defect. In the case of a square defect, the following equation has been established by Giroud et al. (1992):

$$Q = C_{qo}\ b^{0.2}\ h^{0.9}\ k_s^{0.74} \tag{7}$$

In this case, the value of C_{qo} is the same as in the case of a circular defect discussed above.

Case of a Defect of Infinite Length. In the case of a defect of infinite length ($B = \infty$ in Figure 3a), the following equation has been established by Giroud et al. (1992):

$$Q^* = C_{q\infty}\ b^{0.1}\ h^{0.45}\ k_s^{0.87} \tag{8}$$

where $C_{q\infty}$ is the contact quality factor (dimensionless) for a defect of infinite length, with:

$$C_{q\infty\, good} \le C_{q\infty} \le C_{q\infty\, poor} \tag{9}$$

where: $C_{q\infty good}$ = value of $C_{q\infty}$ in the case of good contact conditions; and $C_{q\infty poor}$ = valu of $C_{q\infty}$ in the case of poor contact conditions. The following values were establishe by Giroud et al. (1992):

$$C_{q\infty\, good} = 0.52 \tag{10}$$

$$C_{q\infty\, poor} = 1.22 \tag{11}$$

Case of a Rectangular Defect. In the case of a rectangular defect, the following equa tion has been established by Giroud et al. (1992):

$$Q = C_{qo}\ b^{0.2}\ h^{0.9}\ k_s^{0.74} + C_{q\infty}(B-b)\ b^{0.1}\ h^{0.45}\ k_s^{0.87} \tag{12}$$

where C_{qo} and $C_{q\infty}$ have the values defined above.

2.2 Equations for Large Head

When the head of liquid on top of the liner is greater than the thickness of the low permeability soil component of the composite liner, Equations 2, 3, 7, 8 and 12 are no valid. Instead, the following equations should be used, as shown by Giroud et al. (1992)

- Circular defect:

$$Q = C_{qo}\ i_{avgo}\ a^{0.1}\ h^{0.9}\ k_s^{0.74} \tag{13}$$

- Square defect:

$$Q = C_{qo}\ i_{avgo}\ a^{0.2}\ h^{0.9}\ k_s^{0.74} \tag{14}$$

- Infinitely long defect:

$$Q^* = C_{q\infty}\ i_{avg\infty}\ b^{0.1}\ h^{0.45}\ k_s^{0.87} \tag{15}$$

- Rectangular defect:

$$Q = C_{qo}\ i_{avgo}\ b^{0.2}\ h^{0.9}\ k_s^{0.74} + C_{q\infty}\ i_{avg\infty}\ (B-b)\ b^{0.1}\ h^{0.45}\ k_s^{0.87} \tag{16}$$

where: i_{avgo} = average hydraulic gradient in the low-permeability soil in the case of circular or square defect; and $i_{avg\infty}$ = average hydraulic gradient in the low-permeabilit soil in the case of a defect of infinite length. The values of i_{avgo} and $i_{avg\infty}$ are given in th graphs presented in Figure 4.

It appears that, when the head of liquid is greater than the thickness of the low-perme ability soil component of the composite liner, the calculation of the rate of liquid migra

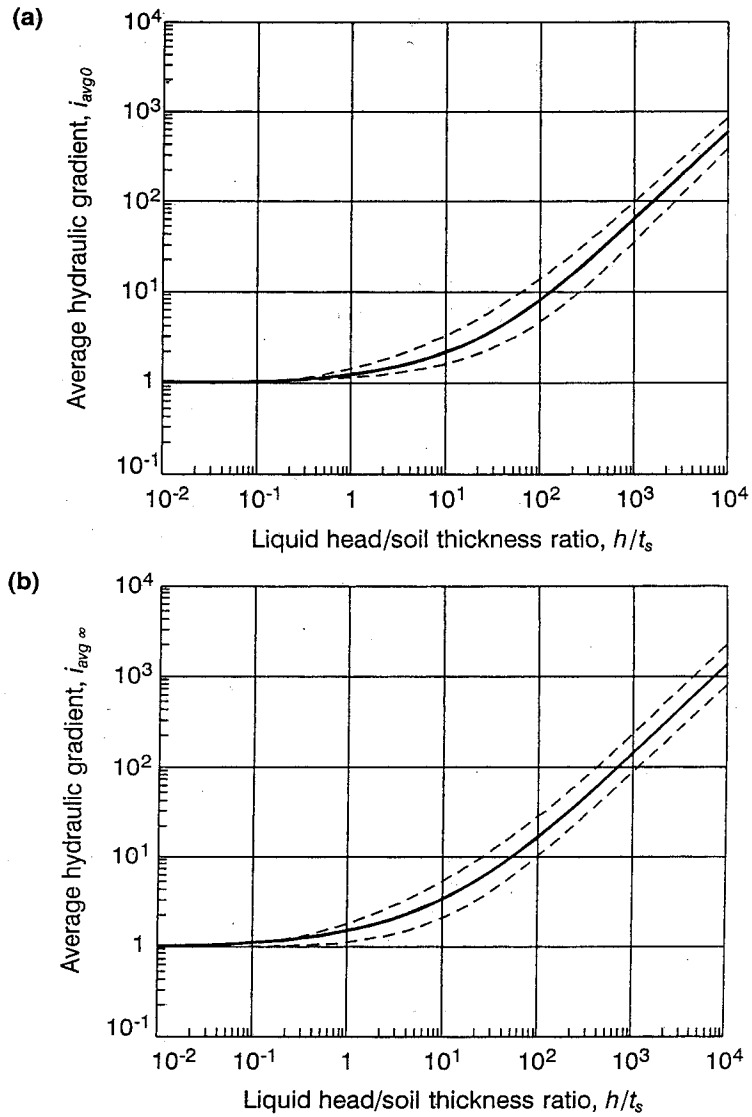

Figure 4. Values of the average hydraulic gradient to be used in equations for liquid migration rate calculations: (a) case of a circular or square defect; (b) case of an infinitely long defect (from Giroud et al. 1992).

Note: The dashed lines represent the upper and lower limit of the range of values for i_{avgo} and $i_{avg\infty}$, since for a given value of h/t_s there is not a unique value of i_{avgo} and $i_{avg\infty}$ (Giroud et al. 1992). The solid lines represent the curves given by Equation 17 (Figure 4a) and Equation 18 (Figure 4b).

tion is not entirely analytical since graphs must be used. This is not convenient, esp cially when calculations for a large number of cases have to be performed.

3 NEW EQUATIONS TO CALCULATE THE RATE OF LIQUID MIGRATION THROUGH COMPOSITE LINERS

3.1 Analytical Expression of the Average Hydraulic Gradient

After numerous attempts, it was found that a good approximation of the values of i_a and $i_{avg\infty}$ presented in Figure 4 is given by the following equations:

$$i_{avgo} = 1 + 0.1\,(h/t_s)^{0.95} \quad (1$$

$$i_{avg\infty} = 1 + 0.2\,(h/t_s)^{0.95} \quad (1$$

3.2 New Equations for Liquid Migration Rate

Combining Equations 13 to 16 with Equations 17 and 18 gives the following equ tions that can be used to calculate the rate of liquid migration through composite line

- Circular defect:

$$Q = C_{qo}\ [1 + 0.1\,(h/t_s)^{0.95}]\ a^{0.1}\ h^{0.9}\ k_s^{0.74} \quad (1$$

hence:

$$Q = 0.976\ C_{qo}\left[1 + 0.1\left(h/t_s\right)^{0.95}\right] d^{0.2}\ h^{0.9}\ k_s^{0.74} \quad (2$$

- Square defect:

$$Q = C_{qo}\ [1 + 0.1\,(h/t_s)^{0.95}]\ b^{0.2}\ h^{0.9}\ k_s^{0.74} \quad (2$$

- Infinitely long defect:

$$Q^* = C_{q\infty}\ [1 + 0.2\,(h/t_s)^{0.95}]\ b^{0.1}\ h^{0.45}\ k_s^{0.87} \quad (2$$

- Rectangular defect:

$$Q = C_{qo}\ [1 + 0.1\,(h/t_s)^{0.95}]\ b^{0.2}\ h^{0.9}\ k_s^{0.74} + C_{q\infty}\,[1 + 0.2\,(h/t_s)^{0.95}]\,(B - b)\ b^{0.1}\ h^{0.45}\ k_s^{0.87} \quad (2$$

Values of C_{qo} are given by Equations 4, 5 and 6. Values of $C_{q\infty}$ are given by Equations 9, 10 and 11. The other parameters are defined in Section 1.5. Equations 19 to 23 are semi-empirical and they must be used with the units defined in Section 1.5.

It should be noted that, when the head of liquid on top of the liner is smaller than the thickness of the low-permeability soil component of the composite liner, the value of i_{avgo} and $i_{avg\infty}$ given by Equations 17 and 18, respectively, is approximately equal to 1, and Equations 19, 20, 21, 22 and 23 become identical to Equations 2, 3, 7, 8 and 12, respectively.

3.3 Limitations

The limits of validity of the above equations are discussed in detail by Giroud et al. (1997). These limits can be summarized as follows:

- If the defect is circular, the defect diameter should be no less than 0.5 mm and not greater than 25 mm. In the case of defects that are not circular, it is proposed to use these limitations for the defect width.
- The liquid head on top of the geomembrane should be equal to or less than 3 m.
- The hydraulic conductivity of the low-permeability soil underlying the geomembrane should be equal to or less than a certain value k_G, which is less than the value k_{GB} for which the relevant equation for the considered defect type (i.e. an equation selected from Equations 19 to 23) and Bernoulli's equation for free flow through an orifice give the same value of the rate of liquid migration through the geomembrane defect.

To ensure a smooth transition between liquid migration rates calculated using Equations 19 to 23 and those calculated using Bernoulli's equation, Giroud et al. (1997) propose the following value for k_G:

$$k_G = k_{GB}/10 \tag{24}$$

In the case where the geomembrane defect is circular, k_G is given by the following equation (Giroud et al. 1997):

$$k_G = \left\{ \frac{0.3891\, d^{1.8}}{C_{qo}\left[1 + 0.1\left(\frac{h}{t_s}\right)^{0.95}\right] h^{0.4}} \right\}^{1/0.74} \tag{25}$$

Equation 25 must be used with the units defined in Section 1.5. Values of k_G calculated using Equation 25 with $C_{qo} = 0.21$ (i.e. good contact conditions, as indicated by Equation 5) and $t_s = 0.6$ m are given in Table 1.

Table 1. Hydraulic conductivity, k_G, of the low-permeability soil underlying t geomembrane that gives the upper limit of validity of the equation for liquid migratio through a circular defect in a geomembrane underlain by a low-permeability so (Equation 20).

Head of liquid on top of the geomembrane, h (m)	Geomembrane defect diameter, d (mm)						
	0.5	1	2	3	5	10	11.284
0.01	2.6×10^{-7}	1.4×10^{-6}	7.5×10^{-6}	2.0×10^{-5}	7.0×10^{-5}	3.8×10^{-4}	5.1×10^{-4}
0.03	1.4×10^{-7}	7.7×10^{-7}	4.1×10^{-6}	1.1×10^{-5}	3.8×10^{-5}	2.1×10^{-4}	2.8×10^{-4}
0.1	7.3×10^{-8}	3.9×10^{-7}	2.1×10^{-6}	5.7×10^{-6}	2.0×10^{-5}	1.1×10^{-4}	1.4×10^{-4}
0.3	3.8×10^{-8}	2.1×10^{-7}	1.1×10^{-6}	3.0×10^{-6}	1.0×10^{-5}	5.6×10^{-5}	7.5×10^{-5}
1	1.8×10^{-8}	9.5×10^{-8}	5.1×10^{-7}	1.4×10^{-6}	4.7×10^{-6}	2.6×10^{-5}	3.4×10^{-5}
3	7.1×10^{-9}	3.8×10^{-8}	2.1×10^{-7}	5.6×10^{-7}	1.9×10^{-6}	1.0×10^{-5}	1.4×10^{-5}

Notes: The tabulated values of k_G were calculated using Equation 25 with $C_{qo} = 0.21$ (good contact) and = 0.6 m. The defect diameter of 11.284 mm corresponds to a defect surface area of 1 cm^2.

3.4 Example

A composite liner consists of a geomembrane placed on a GCL having a thicknes of 6 mm and a hydraulic conductivity of 2×10^{-11} m/s. The geomembrane has a rectar gular defect with a width of 1 mm and a length of 15 mm. The head of liquid on top c the composite liner is 25 mm. Calculate the rate of liquid migration through this defec

The rate of liquid migration through the composite liner is calculated as follows usin Equation 23:

$$Q = C_{qo}[1 + 0.1(25/6)^{0.95}](1\times10^{-3})^{0.2}(25\times10^{-3})^{0.9}(2\times10^{-11})^{0.74} +$$
$$C_{q\infty}[1 + 0.2(25/6)^{0.95}](15-1)\times10^{-3}(1\times10^{-3})^{0.1}(25\times10^{-3})^{0.45}(2\times10^{-11})^{0.8}$$

hence:

$$Q(\mathrm{m}^3/\mathrm{s}) = C_{qo}(1.53\times10^{-10}) + C_{q\infty}(1.17\times10^{-12})$$

Assuming good contact between the geomembrane and the GCL, Equations 5 and 1 give:

$$C_{qo} = 0.21 \qquad C_{q\infty} = 0.52$$

hence:

$$Q(\mathrm{m/s^3}) = 0.21 \times 1.53 \times 10^{-10} + 0.52 \times 1.17 \times 10^{-12} = 3.21 \times 10^{-11} + 6.08 \times 10^{-13}$$

hence:

$$Q = 3.27 \times 10^{-11}\ \mathrm{m/s^3} = 2.8 \times 10^{-3}\ \text{liters/day} = 1.0\ \text{liter/year}$$

It is interesting to note that, if the defect had been square with a side length of 1 mm, the rate of liquid migration through the defect would have been expressed by the first term of the above equation (3.21×10^{-11} m^3/s), which is much greater than the second term. In other words, the calculated rate of liquid migration is only slightly greater through the 15 mm × 1 mm defect than through the 1 mm × 1 mm defect. This is because, in this particular example, the radius of the wetted area, calculated as indicated by Giroud et al. (1992), is very large compared to the defect size and is far more dependent on defect width than on defect length. (The calculation gives a wetted area radius of approximately 0.6 m.)

4 CONCLUSIONS

The equations presented in this paper provide design engineers with an entirely analytical method to calculate the rate of liquid migration through a composite liner, due to geomembrane defects, for liquid heads on top of the liner up to 3 m. The new equations are equivalent to the existing method (Giroud et al. 1992, 1994) which requires both equations and graphs when the head of liquid on top of the liner is greater than the thickness of the low-permeability soil component of the composite liner. However, the new equations are more convenient because the values that had to be obtained from graphs are now incorporated into the equations (Equations 19 to 23).

The new equations are particularly useful in cases where the low-permeability soil component of the composite liner is a GCL because the head of liquid on top of the liner is often greater than the thickness of the GCL.

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NOTATIONS

Basic SI units are given in parentheses.

a	=	defect area (m^2)
B	=	length of rectangular defect (m)
b	=	width of rectangular defect (m)
b	=	side length of square defect (m)
C_q	=	contact quality factor (dimensionless)
C_{qo}	=	contact quality factor for a circular or square defect (dimensionless)
C_{qogood}	=	value of C_{qo} in the case of good contact conditions (dimensionless)
C_{qopoor}	=	value of C_{qo} in the case of poor contact conditions (dimensionless)
$C_{q\infty}$	=	contact quality factor for a defect of infinite length (dimensionless)
$C_{q\infty\ good}$	=	value of $C_{q\infty}$ in the case of good contact conditions (dimensionless)
$C_{q\infty\ poor}$	=	value of $C_{q\infty}$ in the case of poor contact conditions (dimensionless)
D		depth of liquid on top of the geomembrane (m)
d	=	diameter of circular defect (m)
h	=	head of liquid on top of the geomembrane (m)
i_{avgo}	=	average hydraulic gradient in the low-permeability soil in the case of circular or square defect (dimensionless)
$i_{avg\infty}$	=	average hydraulic gradient in the low-permeability soil in the case of a infinitely long defect (dimensionless)
k_G	=	value of k_s above which Equations 19 to 23 are not valid (m/s)
k_{GB}	=	value of k_s for which Equation 19 to 23 and Bernoulli's equation for fre flow through an orifice give the same value of the rate of liquid migratio through a geomembrane defect (m/s)

k_s	=	hydraulic conductivity of the low-permeability soil component of the composite liner (m/s)
Q	=	liquid migration rate through the considered geomembrane defect (m^3/s)
Q^*	=	liquid migration rate per unit length of geomembrane defect in the case of an infinitely long defect (m^2/s)
R	=	radius of wetted area (m)
t	=	thickness of liquid on top of the geomembrane (m)
t_s	=	thickness of the low-permeability soil component of the composite liner (m)
β	=	slope angle (°)

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Technical Paper by J.P. Giroud

EQUATIONS FOR CALCULATING THE RATE OF LIQUID MIGRATION THROUGH COMPOSITE LINERS DUE TO GEOMEMBRANE DEFECTS

ABSTRACT: Equations available to date for calculating the rate of liquid migration through a composite liner due to geomembrane defects require the use of graphs to obtain the value of one of the terms of the equations for the case where the liquid head is larger than the thickness of the low-permeability soil component of the composite liner. In this paper, it is shown that the terms that require graphs can be expressed analytically, which leads to a new set of equations that provides an entirely analytical means of calculating the rate of liquid migration through composite liners. This new set of equations is particularly useful when the liquid head is large compared to the thickness of the low-permeability soil component of the composite liner, which is often the case when the low-permeability soil associated with the geomembrane to form a composite liner is a geosynthetic clay liner. A numerical example is given.

KEYWORDS: Liquid migration, Leachate migration, Leakage, Composite liner, Equations.

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1 INTRODUCTION

Geosynthetic clay liners (GCLs) are increasingly accepted as a replacement for compacted clay liners (CCLs) in landfill liner systems. In most instances where a GCL is used to replace a CCL in a landfill liner system, an equivalency demonstration is required by regulation. This demonstration typically consists of establishing that the expected performance of the liner system with a GCL is equivalent or superior to the expected performance of the same liner system with a CCL. The equivalency demonstration may include analyses of advective flow of leachate through the liners, diffusion of leachate constituents through the liners, slope stability, settlement, chemical compatibility with leachate, etc.

Even when an equivalency demonstration is not required, the design engineer needs to evaluate and compare the expected performance of the various liner system alternatives considered in design, especially when one of the alternatives consists of replacing a CCL with a GCL.

This paper presents guidance for one aspect of the comparison between liner systems that incorporate a GCL or a CCL: the rate of advective flow of leachate through the liners. Leachate migration mechanisms other than advective flow are not discussed in this paper; this does not mean that mechanisms other than advective flow are not important.

2 BACKGROUND INFORMATION

2.1 Composite Liners

A composite liner is a liner that consists of two or more components. In virtually all cases where a composite liner is used in a landfill, the composite liner consists of a geomembrane and a low-permeability soil layer. Typically, the geomembrane component of the composite liner is placed on top of the low-permeability soil layer, which decreases percolation of leachate into the liner and promotes lateral flow of leachate in the leachate collection layer overlying the composite liner since the geomembrane is less permeable than the low-permeability soil (Figure 1). In other words, leachate collection and removal is maximized and percolation of leachate into the liner is minimized.

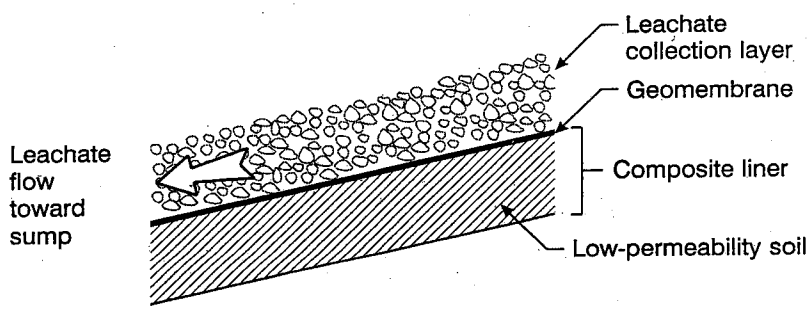

Figure 1. Leachate flow in the leachate collection layer overlying a composite liner.

2.2 The Low-Permeability Soil Component in a Composite Liner

In the current state of practice for landfill design, the low-permeability soil component of composite liners is either a CCL or a GCL. CCLs are typically 0.3 to 1.5 m thick and are constructed in lifts approximately 0.15 m thick after compaction. GCLs are on the order of 5 to 10 mm thick and consist of a thin layer of clay soil associated with one or two geosynthetic layers. GCLs currently available in North America have three configurations: (i) a clay layer with a geotextile glued to its top and bottom surfaces; (ii) a clay layer located between two geotextiles which are stitched together or needle-punched through the clay; or (iii) a clay layer glued to a geomembrane.

The type of clay used in GCLs currently available in North America is bentonite. Bentonite is the name given (from Fort Benton, Montana, USA) to the highly plastic clay which consists of the montmorillonite mineral. Bentonites used to fabricate GCLs are processed in an unhydrated state and appear to have a granular or powdery consistency. Upon hydration (i.e. absorption of water), the bentonite swells to form a continuous clay layer of low permeability. It is this low permeability that has resulted in the use of GCLs as barrier layers in liquid containment applications. The hydraulic conductivities of GCLs and CCLs vary depending on the compressive stress. The measured hydraulic conductivities of hydrated GCLs in laboratory tests designed to simulate the compressive stress conditions in a landfill are typically in the range of 5×10^{-12} to 5×10^{-11} m/s. These hydraulic conductivities are roughly 10 to 100 times lower than the hydraulic conductivities of CCLs which are typically between 1×10^{-10} and 1×10^{-9} m/s. The physical characteristics and engineering properties of commercially-available GCL products are described by Daniel and Boardman (1993).

The hydraulic conductivity values given above are for the case when the liquid permeating the CCL or the GCL is water or a low-concentration leachate that does not affect the hydraulic conductivity of the CCL and the GCL. Examples of such leachates are given by Ruhl and Daniel (1997). In this paper, it is assumed that the liquid permeating the CCL or the GCL does not affect the hydraulic conductivity of the CCL or the GCL.

2.3 Roles of the Low-Permeability Soil Component in a Composite Liner

The low-permeability soil component of a composite liner has several roles, as discussed below:

- The low-permeability soil *decreases the leachate flow through defects in the overlying geomembrane* compared to the case where the geomembrane is placed on a high-permeability material. In other words, the leachate flow through a geomembrane defect is smaller in the case of a composite liner than in the case of a geomembrane liner. This is a very important consideration since leachate migration through geomembranes occurs primarily through geomembrane defects. Both CCLs and GCLs are effective in significantly reducing the advective flow of leachate through defects in the overlying geomembrane (Giroud and Bonaparte 1989; Giroud et al. 1994; Koerner and Daniel 1993).
- The low-permeability soil *acts as a diffusion barrier and delays the diffusion into the ground of chemical compounds* that diffuse through the geomembrane or migrate

through geomembrane defects. CCLs are more effective diffusion barriers than GCLs because diffusion through a CCL is only slightly faster than through a hydrated GCL and the thickness of a CCL is typically on the order of 100 times the thickness of a GCL.

- The low-permeability soil *may absorb some of the chemical compounds* that diffuse through the geomembrane or migrate through geomembrane defects, thereby decreasing the concentrations of chemical compounds in the leachate that migrates into the ground. This mechanism is often referred to as attenuation. The attenuation in CCLs is more effective than in GCLs because the larger thickness of CCLs more than compensates for the greater absorption capability of bentonite in GCLs (Daniel 1996).
- The low-permeability soil *contributes to the durability of the composite liner* because clay minerals are expected to remain virtually unchanged for a very long time since they are the end products of the degradation of other minerals and do not have significant potential for further degradation (Mitchell and Jaber 1990). Geomembranes also are expected to last for a long time: in the present state of knowledge, it is possible to predict that geomembranes typically used in landfills will last more than 100 years (Haxo and Haxo 1988). However, the low-permeability soil component of a composite liner is generally believed to be more durable than the geomembrane component. Furthermore, there is more design experience with clay mineral particles (including bentonite) than with the polymeric compounds used to manufacture geomembranes. For all of these reasons, some regulations and design engineers consider an extreme design case where the geomembrane has deteriorated while the low-permeability soil is still essentially intact. This design case will be extensively discussed in this paper. It should however be noted that a GCL may not contribute to the durability of a composite liner as well as a CCL since the geosynthetic components of the GCL may deteriorate in the long term, leaving a layer of bentonite that may not be thick enough to remain continuous in the absence of the support provided by the associated geosynthetics, although the bentonite particles are expected to last for an extremely long period of time.

2.4 Uses of Composite Liners

Composite liners are used in most modern landfills because they can be far more effective than either a geomembrane alone or a layer of low-permeability soil alone (Giroud and Bonaparte 1989; Giroud et al. 1994). Typical examples of liner systems incorporating composite liners are shown in Figure 2. The following points should be noted in Figure 2:

- A liner in a modern landfill is always overlain by a leachate collection layer. In the case of a double liner, there are two leachate collection layers: the primary leachate collection layer which overlies the primary liner and the secondary leachate collection layer (also called leakage detection and collection layer) which overlies the secondary liner.

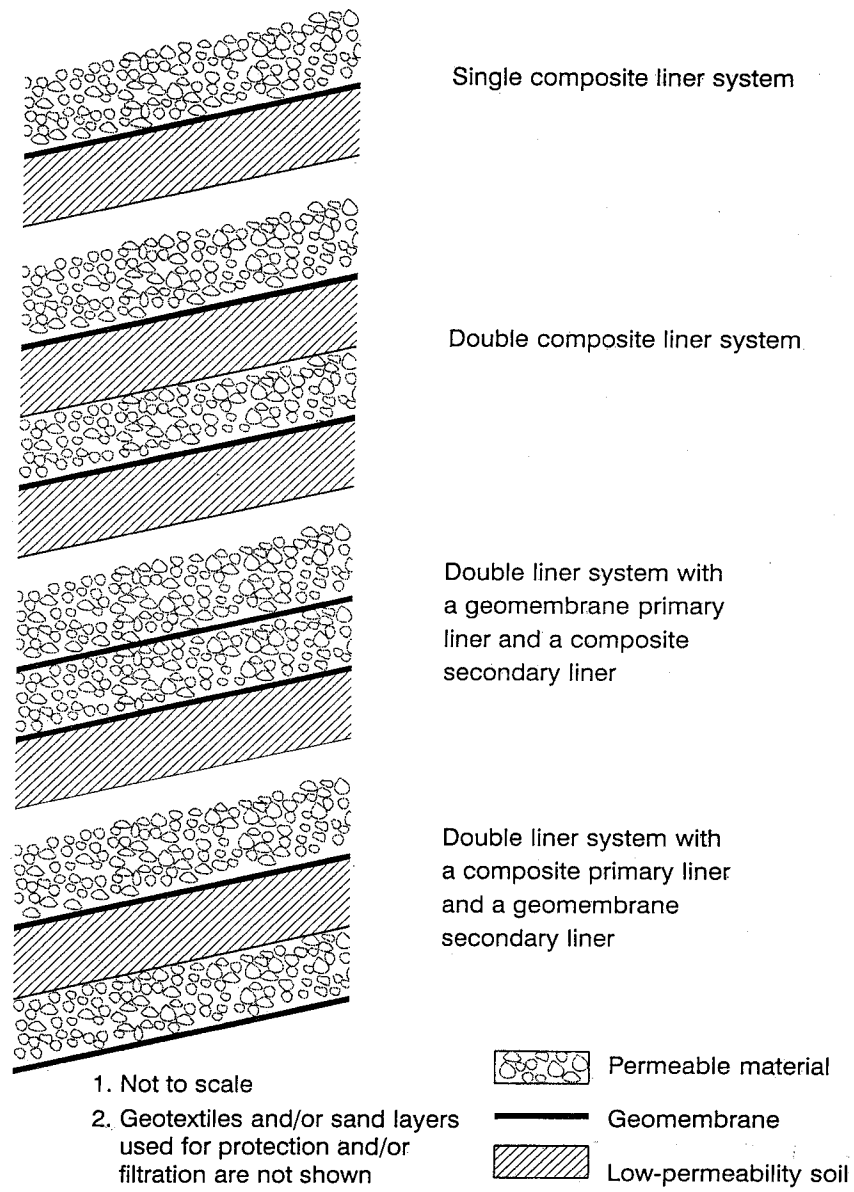

Figure 2. Typical uses of composite liners in landfill liner systems.
Note: The terminology "liner system" refers to a combination of liners and leachate collection layers.

- A composite liner is not a double liner. A composite liner is a single liner that consists of two components acting together. A double liner consists of two liners acting independently and separated by the secondary leachate collection layer.

2.5 Regulatory Requirements Regarding Composite Liners

Most regulations in North America prescribe, for the low-permeability soil component of composite liners, a CCL with a maximum hydraulic conductivity of 1×10^{-9} m/s. The thickness prescribed for the CCL component of a composite liner depends on the regulation. In the United States, federal regulations require, for hazardous waste landfills, a double liner with a secondary composite liner where the CCL thickness is 0.9 m, and, for municipal solid waste landfills, a single composite liner where the CCL thickness is 0.6 m. A CCL with a thickness, t_{CCL}, of 0.6 m and a hydraulic conductivity, k_{CCL}, of 1×10^{-9} m/s will be used in this paper as a basis for comparison with GCLs. This CCL will be referred to as "the standard CCL". Also, a composite liner that consists of a geomembrane and a standard CCL will be referred to as "the standard composite liner".

In the United States, whereas federal regulations for municipal solid waste landfills require a single composite liner, some states go beyond the minimum federal requirement, and instead prescribe double liners that include one or two composite liners. In the case of primary composite liners where the soil component is a CCL, it is recognized that compaction of the primary liner CCL may be detrimental to the geosynthetics used in the secondary liner and the secondary leachate collection system, due to the stresses caused by heavy compaction equipment. Therefore, some regulations indicate that the soil component of the primary composite liner should consist of two layers: (i) an upper low-permeability soil layer that consists of a CCL; and (ii) a lower layer, sometimes referred to as "structural fill", placed with lightweight equipment and whose purpose is to protect the underlying geosynthetics from stresses induced by the equipment compacting the CCL (Figure 3). Specifications for these two layers are, for example:

- The upper layer (i.e. the CCL) should have a maximum hydraulic conductivity of 1×10^{-9} m/s and a minimum thickness of 0.15 m after compaction.
- The lower layer (i.e. the structural fill) should have a minimum thickness of 0.30 m after compaction.

Some specifications also require that the structural fill have a maximum hydraulic conductivity, such as 1×10^{-7} m/s. Calculations presented in Section 3.5 will show that a layer with a hydraulic conductivity of 1×10^{-7} m/s placed under a layer with a hydraulic conductivity of 1×10^{-9} m/s does not provide any significant contribution to leachate migration control; therefore, the maximum hydraulic conductivity requirement for the structural fill is pointless from the viewpoint of advective flow.

2.6 Use of Structural Fill With GCLs

In the present state of practice, regulations usually do not mention GCLs directly, but make their use possible through equivalency demonstrations. However, in one state in the United States, the regulation specifies that, if a GCL is used in a composite primary

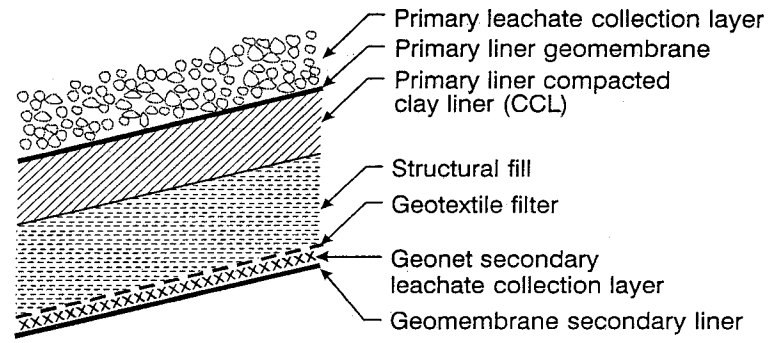

Figure 3. Structural fill located beneath the composite primary liner to protect the underlying geosynthetics during compaction of the CCL component of the primary liner.
Note: The terminology "Primary liner geomembrane" means that the geomembrane is only one of the components of the primary liner, whereas the terminology "Geomembrane secondary liner" means that, in the above example, the secondary liner consists of a geomembrane alone.

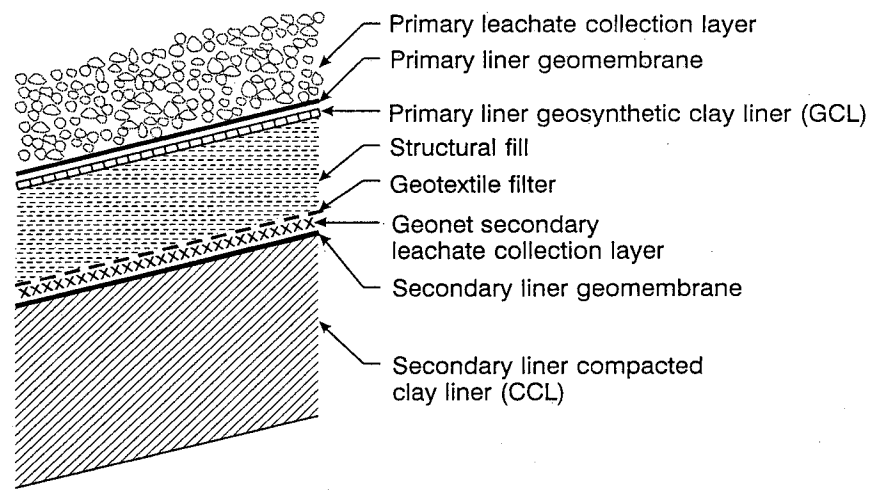

Figure 4. Structural fill located beneath the composite primary liner.
Note: This figure is identical to Figure 3, except that: in the composite primary liner, the CCL has been replaced by a GCL; and, in the secondary liner, the geomembrane liner has been replaced by a composite liner.

liner of a double liner system, it should be placed on a structural fill as shown in Figure 4. The cross section shown in Figure 4 is the same as the cross section shown in Figure 3, except that the CCL component of the composite primary liner has been replaced by

a GCL. (Also, to be consistent with the regulation of the considered state, the secondary liner in Figure 4 is a composite liner.)

The structural fill in the case where the composite primary liner includes a CCL (Figure 3) is justified because it protects the underlying geosynthetics from potential damage by the heavy equipment used to compact the clay of the CCL. In the case where the soil component of the composite primary liner is a GCL, no heavy equipment is needed and the structural fill is not needed for protecting the underlying geosynthetics. Furthermore, the structural fill is potentially detrimental for two reasons:

- There is a risk (however small) that the lightweight equipment used to place the structural fill can cause some damage to the underlying geosynthetics. (The same risk exists when the composite primary liner incorporates a CCL, but this risk is then outweighed by the benefit due to the fact that the structural fill protects the underlying materials during compaction of the clay.)
- The thickness of the structural fill increases the thickness of the liner system, thereby decreasing the storage capacity of the landfill.

It has also been argued by others that a low-permeability structural fill placed under the GCL would contribute to a reduction of the rate of flow through the geomembrane-GCL composite liner. Calculations presented in Section 3.5 will show that the contribution to leachate flow control of any material (even with a low permeability) placed under the GCL is either zero or negligible.

From the foregoing discussion and calculations presented in Section 3.5, it is clear that the use of a structural fill (with a low-permeability requirement) beneath a GCL, in the case of a composite primary liner of a double liner system, has several disadvantages and does not contribute to reducing the advective flow through a geomembrane-GCL composite liner. The only justification of a low-permeability structural fill beneath a GCL might be a decrease in the migration of contaminants by diffusion, and attenuation of contaminant concentrations. As mentioned in Sections 1 and 2.9, the evaluation of these two effects is beyond the scope of this paper, which does not mean that these effects are not important.

2.7 Use of a GCL in Addition to a CCL

In some conservative designs, a GCL is used in addition to a geomembrane and a CCL (or other low-permeability soil) to form a three-layer composite liner. Such designs are often proposed by some landfill owners/operators to facilitate the permitting process. In such cases, it may not be required to submit an equivalency demonstration (since a geomembrane-CCL composite liner would meet the regulatory requirements), but it is useful to evaluate the expected performance of the three-layer composite liner system.

2.8 Uses of GCLs in Landfill Liner Systems

Regulations often indicate that a liner system other than the minimum required system can be approved if it can be demonstrated that the expected performance of the proposed system is equivalent or superior to the expected performance of the minimum required system. As mentioned in Section 1, such demonstrations are generally called "equivalency demonstrations".

Since approximately 1990, equivalency demonstrations have been used to justify the use of a GCL in place of the CCL component of composite liners prescribed in regulations. Examples of landfill liner systems where a GCL was used to replace the CCL prescribed in the regulations are given in Table 1. The examples presented in Table 1 show that GCLs are increasingly accepted in North America as a replacement for CCLs in landfill liner systems. (It should be noted that Table 1 does not include the numerous cases where GCLs have been used to replace CCLs in landfill final cover systems.)

In the current state of landfill design in the United States, GCLs are always used in conjunction with geomembranes to form composite liners. However, there may be instances where GCLs are used without geomembranes (i.e. either alone or with CCLs) in containment structures other than landfills, or even in landfills in countries where regulations are different from the regulations in force in the United States.

2.9 Guidance for Equivalency Demonstrations

Regulations generally do not provide guidance regarding the methods to be used to perform equivalency demonstrations. Therefore, it is the design engineer's responsibility to determine the approach to be used in the equivalency demonstration. Since the purpose of a liner system is to prevent, or at least minimize, the migration of leachate into the ground, every equivalency demonstration should include a calculation to show that the rate of leachate migration is less through the proposed liner system with a GCL than through the prescribed liner system with a CCL. Usually, only advective flow is considered. The design engineer may also address the migration of chemicals through the liner system as a result of diffusion; this aspect is not addressed in this paper which is devoted to advective flow, as indicated in Section 1.

3 FLOW THROUGH SOIL LAYERS

3.1 Introduction

As indicated in Section 2.3, there are situations in landfill design where the geomembrane component of a composite liner is ignored and, as a result, only the soil layers are considered. Section 3 of this paper is devoted to flow through soil layers (including the bentonite layer in a GCL).

The results of the calculations presented in Section 3 can be used for the design situations where the geomembrane component of a composite liner is ignored, and for the containment structures, other than landfills, where soil liners are used without a geomembrane. The results of the calculations presented in Section 3 also provide useful information for the case where the geomembrane is not ignored (i.e. the composite liner case discussed in Section 4).

Table 1. Examples of landfills in North America where a GCL was used in the liner system and was permitted based on an equivalency demonstration.[a]

Name of landfill/owner	State/Province	Type of liner	First date	Material directly beneath the GCL	Designer
City of Midland Landfill/ City of Midland	Texas	Single composite	1994	Prepared subbase	HDR Engineering
City of Odessa Landfill/ City of Odessa	Texas	Single composite	1994	Compacted subbase	Freese-Nichols, Inc.
City of Loredo Landfill/City of Loredo	Texas	Single composite	1994	Compacted subbase	Huntingdon
City of Lubbock Landfill/ City of Lubbock	Texas	Single composite	1995	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Park, Hill, Smith & Cooper
City of Snyder Landfill/ City of Snyder	Texas	Single composite	1994	Compacted subbase	Freese-Nichols, Inc.
La Paz County Landfill/La Paz	Arizona	Single composite	1995	Compacted subbase (sand)	Vector Engineering
Mahoning County Landfill/ Mahoning Landfill, Inc.	Ohio	Single composite	1995	0.9 m of compacted soil $k \leq 1 \times 10^{-9}$ m/s	Earth Sciences Consultants
Carbon Limestone Landfill/BFI	Ohio	Single composite	1994	0.9 m of compacted soil $k \leq 1 \times 10^{-9}$ m/s	Paul C. Rizzo Associates
Vasco Road Sanitary Landfill/BFI	California	Single composite	1994	Cushion geotextile or geocomposite on natural subgrade (rock)	GeoSyntec
B&J Sanitary Landfill/ NorCal Waste Systems	California	Single composite	1995	0.3 m of compacted clay $k \leq 1 \times 10^{-7}$ m/s	GeoSyntec
Cummings Road Sanitary/ NorCal Waste Systems	California	Single composite	1994	Cushion geotextile or geocomposite on natural subgrade (rock)	GeoSyntec
Puente Hills/Los Angeles County	California	Single composite	1994	Shotcrete	GeoSyntec
Lopez Canyon/City of Los Angeles	California	Single composite[b]	1994	Shotcrete	GeoSyntec
Olinda Alpha/Orange County	California	Single composite	1995	Shotcrete	GeoSyntec
Newland Park Landfill/ Wicomico County	Maryland	Single composite	1995	Structural fill (silty sand)	GeoSyntec
City of Dothan Landfill/City of Dothan	Alabama	Single composite	1995	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Poly Engineering/ GeoSyntec
Jefferson County Landfill/ Jefferson County	Alabama	Single composite	1996	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	ESA Engineering
Enoree Landfill/Greenville County	South Carolina	Single composite	1992	0.6 m of compacted soil	Tribble & Richardson
			1995	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Tribble & Richardson
			1996	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	HDR Engineering
York County Landfill/ York County	South Carolina	Single composite	1996	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	B.P. Barber & Associates
Screaming Eagle Road Landfill/ USA Waste	South Carolina	Single composite	1990	Natural soil subgrade	Golder Associates
			1994	0.45 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	
Oakridge Landfill/USA Waste	South Carolina	Single composite	1990	Natural soil subgrade	Golder Associates
			1994	0.45 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	
Greenwood County Landfill/ Greenwood County	South Carolina	Single composite	1993	0.6 m of compacted soil $k \leq 1 \times 10^{-8}$ m/s	S&ME
Bradley County Landfill/ Bradley County	Tennessee	Single composite	1994	0.6 m of compacted soil $k \leq 1 \times 10^{-8}$ m/s	Santek
Summitt Landfill/City of Chattanooga	Tennessee	Single composite	1995	0.6 m of compacted soil $k \leq 1 \times 10^{-8}$ m/s	Consolidated Tech, Inc.
NW Tennessee Disposal Landfill/ Continental Waste	Tennessee	Single composite	1995	0.6 m of compacted soil $k \leq 1 \times 10^{-8}$ m/s	Vaughn & Melton
Decatur Landfill/Decatur County	Tennessee	Single composite	1996	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	GeoSyntec
Addington Landfill/ Addington Environmental	Kentucky	Single composite	1995	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Addington Environmental
Accomack County Landfill/ Accomack County	Virginia	Single composite	1996	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	SCS Engineers
Spottsylvania County Landfill/ Spottsylvania County	Virginia	Single composite	1996	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Draper Aden Associates
Amelia Landfill/Chambers	Virginia	Single composite	1994	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	GZA
Charles City Landfill/Chambers	Virginia	Single composite	1994	0.3 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	GeoSyntec
Southern States Landfill/Allied Waste	Georgia	Single composite	1996	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	HHNT
Monroe County Landfill/ Monroe County	Georgia	Single composite	1996	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Moreland Altobelli
Atkinson County Landfill/ Atkinson County	Georgia	Single composite	1996	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	E&C Consultants

Table 1. Continued.

Hall County Landfill/Hall County	Georgia	Single composite	1996	0.6 m of compacted soil $k \leq 1 \times 10^{-7}$ m/s	Emcon
Pine Bluff Landfill/Sanifill	Georgia	Single composite	1994	0.6 m of compacted soil $k \leq 1 \times 10^{-6}$ m/s	Jordan, Jones & Goulding
Trail Ridge Landfill/ Waste Management	Florida	Double liner (Composite secondary)	1992	0.15 m of compacted subbase	England-Thims & Miller
CSL Ash Monofill/Waste Management	Florida	Double liner (Composite secondary)	1990	0.15 m of compacted subbase	GeoSyntec
Sand Springs Landfill/ARCO	Oklahoma	Double liner (Composite secondary)	1992	Natural soil subgrade beneath secondary liner GCL	Morrison Knudsen/GeoSyntec
New Hanover Landfill/ New Hanover County	North Carolina	Double liner (Composite secondary)	1993	Structural fill (sand)	PBS&J
Saint Etienne/Waste Management	Quebec	Double liner (Composite primary)	1993	Secondary leachate collection layer composed of a geotextile filter placed on top of a geonet drainage layer	ADS
Berman Road Landfill/USA Waste	Florida	Double composite	1992	Geotextile beneath primary liner GCL Natural subgrade (sand) or structural fill (sand) beneath secondary liner GCL	GeoSyntec
Alcoa Massena Secure Landfill/Alcoa	New York	Double composite	1992	Geonet beneath primary liner GCL Natural soil subgrade beneath secondary liner GCL	GeoSyntec
Grand Central Landfill/USA Waste	Pennsylvania	Double composite	1993	1.0 mm thick high density polyethylene (HDPE) geomembrane	Earth Resources Group
Allis Park Landfill/BFI	Michigan	Double composite	1994	0.6 m of compacted soil $\leq 1 \times 10^{-9}$ m/s beneath primary liner GCL, natural soil subgrade beneath secondary liner GCL	Midwestern Consulting, Inc.
Coffin Butte Landfill/ Valley Landfills, Inc.	Oregon	Double composite	1995	0.3 m thick sand secondary leachate collection and detection layer	Emcon

Notes: (a) This table does not include the numerous cases where GCLs were used in landfill covers; (b) GCL used in side slopes only.

3.2 Flow Through a Single Soil Layer

Darcy's Equation. The rate of advective flow of leachate through a single soil layer (Figure 5a) is given by Darcy's equation as follows:

$$v = q = Q/A = k\ i = k\,\Delta h / t \tag{1}$$

where: v = apparent flow velocity; q = unit flow rate (i.e. flow rate per unit area); Q = flow rate; A = surface area of the soil layer; k = hydraulic conductivity of the soil; i = hydraulic gradient; Δh = head loss through the soil layer; and t = thickness of the soil layer.

Equation 1 can be written as follows:

$$v = q = Q/A = \psi\,\Delta h \tag{2}$$

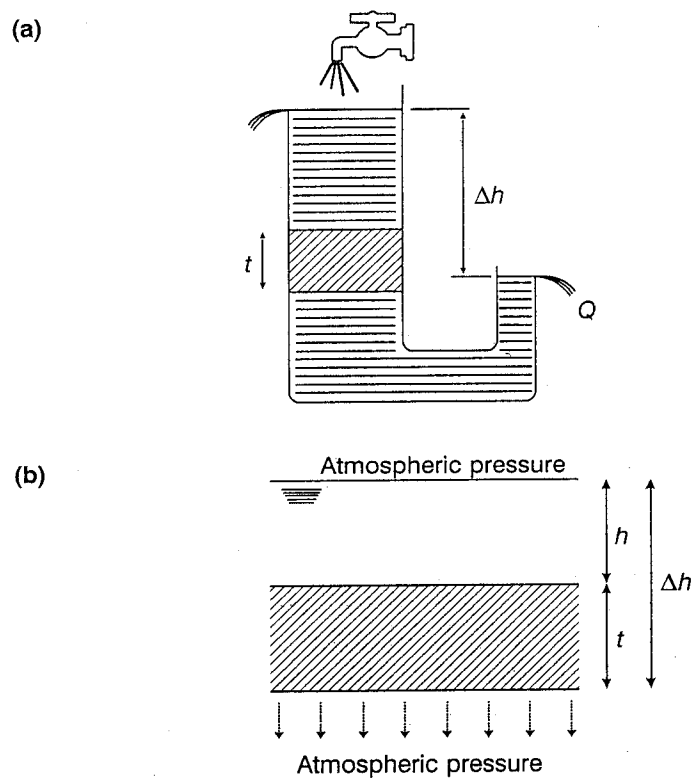

Figure 5. Flow through a single layer of soil: (a) case of a given head loss; (b) case of gravity flow.

where ψ is the permittivity of the soil layer defined as follows:

$$\psi = k / t \tag{3}$$

The permittivity is a characteristic of the *soil layer* (since it includes the thickness) whereas the hydraulic conductivity is a characteristic of the *soil*. Typical values of thickness, hydraulic conductivity and permittivity of GCLs used in this paper for numerical examples are given in Table 2.

As shown by Equation 2, the permittivity ψ is directly obtained from a flow test where Q is measured, Δh is imposed, and A is known. The thickness of the sample is not involved in the measurement. Therefore, in the case of a GCL where the thickness (and, in particular, the thickness of bentonite) is not easy to measure, it is tempting to use the permittivity to characterize the hydraulic behavior. However, as seen in subsequent sections, it is often necessary to know both the hydraulic conductivity, k, and the thickness, t; therefore, knowing only the permittivity, $\psi = k/t$, is not sufficient.

Gravity Flow. In the case of gravity flow (Figure 5b), assuming the soil layer is saturated, the hydraulic head loss is:

$$\Delta h = h + t \tag{4}$$

where: h = head of leachate on top of the liner.

Combining Equations 1 and 4 shows that, in the case of gravity flow, Darcy's equation becomes:

$$q = k \frac{h + t}{t} = k(1 + h / t) \tag{5}$$

Table 2. Typical GCL characteristics used in this paper for numerical comparisons.

Characteristics of the hydrated bentonite layer in a GCL	Applied compressive stress		
	High (e.g. > 200 kPa)	Medium (e.g. 50-100 kPa)	Low (e.g. < 20 kPa)
Thickness, t_{GCL} (mm)	5	7	9
Porosity, n_{GCL} (%)	68	77	82
Hydraulic conductivity, k_{GCL} (m/s)	5×10^{-12}	1×10^{-11}	5×10^{-11}
Permittivity, ψ_{GCL} (s^{-1})	1×10^{-9}	1.4×10^{-9}	5.6×10^{-9}

Notes: Each tabulated porosity is an average value calculated for the thickness tabulated above in the same column, considering an initial (i.e. before hydration) bentonite mass per unit area of approximately 5 kg/m^2 (Giroud et al. 1997). The tabulated hydraulic conductivities are typical values based on numerous test data for hydrated GCLs having an initial bentonite mass per unit area of approximately 5 kg/m^2. These hydraulic conductivities depend on the magnitude of the applied compressive stress; the compressive stress to be considered is a representative compressive stress during landfill operations (i.e. when leachate migration control is most needed). In each column, the permittivity was derived from the thickness and hydraulic conductivity using Equation 3. The values presented in the above table should not be regarded as general guidance. They are only intended to put in perspective the numerical values used in this paper for illustration purposes.

Flow Rate Comparison. The following equation can be derived from Equation 5:

$$\frac{q_{CCL}}{q_{GCL}} = \frac{k_{CCL}(1 + h / t_{CCL})}{k_{GCL}(1 + h / t_{GCL})} \tag{6}$$

where: q_{CCL} = unit rate of flow through a CCL; q_{GCL} = unit rate of flow through a GCL; k_{CCL} = hydraulic conductivity of the CCL; k_{GCL} = hydraulic conductivity of the GCL; t_{CCL} = thickness of the CCL; t_{GCL} = thickness of the GCL; and h = head of leachate on top of the liner.

A comparison of calculated advective flow rate ratios (i.e. q_{CCL}/q_{GCL}) between CCLs and GCLs is presented in Table 3. The CCL considered in Table 3 is the standard CCL defined in Section 2.5, whose characteristics are:

- thickness, t_{CCL} = 0.6 m; and
- hydraulic conductivity, $k_{CCL} = 1 \times 10^{-9}$ m/s.

Three cases are considered in Table 3 for the characteristics of the GCL. These three cases are summarized in Table 2.

It appears in Table 3 that the comparison between flow rates through CCLs and GCLs depends on the hydraulic conductivity of the GCL (which depends in great part on the applied compressive stress, as shown in Table 2) and on the leachate head. However, Table 3 shows that, in most cases, the flow rate is smaller through a GCL than through a CCL.

Table 3. Ratio between rates of advective flow through a CCL and a GCL, q_{CCL}/q_{GCL}.

GCL characteristics: Thickness, t_{GCL} (mm) Hydraulic conductivity, k_{GCL} (m/s)			5 5×10^{-12}	7 1×10^{-11}	9 5×10^{-11}
	(m)	(mm)			
Leachate head on top of the liner, h	0	0	200.00	100.00	20.00
	0.01	10	67.78	41.86	9.63
	0.05	50	19.70	13.30	3.31
	0.1	100	11.11	7.63	1.93
	0.3	300	4.92	3.42	0.87
	0.6	600	3.31	2.31	0.59
	1.0		2.65	1.85	0.48
	10		1.77	1.24	0.32
	∞		1.67	1.17	0.30

Notes: The tabulated values of the advective flow rate ratio were calculated using Equation 6 with the following characteristics of the CCL: thickness, t_{CCL} = 0.6 m; and hydraulic conductivity, $k_{CCL} = 1 \times 10^{-9}$ m/s. (This is the "standard CCL" defined in Section 2.5.) The characteristics of the GCL are from Table 2. It is important to note that the values tabulated above are related to a CCL and a GCL used alone (i.e. without a geomembrane).

It is important to note that the above conclusions are valid only if there is no geomembrane on top of the CCL or GCL, or if the geomembrane is ignored for design purposes. The case where the geomembrane is taken into account will be discussed in Section 4 and conclusions somewhat different will be drawn.

Equivalency Condition. From Equation 6, with $q_{GCL}/q_{CCL} \leq 1$, it is possible to derive the value of the maximum hydraulic conductivity for a GCL to be superior, or equivalent, to a CCL regarding advective flow control:

$$k_{GCL} \leq \text{Max}\left(k_{GCL}\right) = \frac{k_{CCL}\left(1 + h / t_{CCL}\right)}{1 + h / t_{GCL}} \tag{7}$$

Derivations (not reproduced here) show that the derivative of Max(k_{GCL}) with respect to the leachate head, h, is proportional to ($t_{GCL} - t_{CCL}$). Since this term is always negative, Max(k_{GCL}) decreases as h increases. Therefore, if the condition expressed by Equation 7 is verified for a given value of h, it is verified for all values of h that are smaller than the given value. This leads to the following condition for a GCL to be equivalent, or superior, to the "standard CCL" defined in Section 2.5 (thickness 0.6 m and hydraulic conductivity 1×10^{-9} m/s) *for all leachate heads smaller than 0.3 m* (a typical maximum head specified in regulations):

$$k_{GCL} \leq \text{Max}\left(k_{GCL}\right) = \frac{\left(1 \times 10^{-9}\right)\left(1 + 0.3 / 0.6\right)}{1 + 0.3 / t_{GCL}} \tag{8}$$

with k_{GCL} in m/s and t_{GCL} in m.

The condition expressed by Equation 8 can be written as follows:

$$k_{GCL} \leq \text{Max}\left(k_{GCL}\right) = \frac{1.5 \times 10^{-9}}{1 + 300 / t_{GCL}} \tag{9}$$

with k_{GCL} in m/s and t_{GCL} in mm.

More conservatively, one may prefer to specify that the GCL must be equivalent, or superior, to a given CCL for all heads. Based on the above discussion, one must then use the limit value of Equation 7 when h tends toward infinity, hence:

$$k_{GCL} \leq \text{Min}\left[\text{Max}\left(k_{GCL}\right)\right] = k_{CCL}\ t_{GCL} / t_{CCL} \tag{10}$$

hence the condition to be met by a GCL to be equivalent, or superior, to a given CCL *for all leachate heads*:

$$\frac{k_{GCL}}{t_{GCL}} \leq \frac{k_{CCL}}{t_{CCL}} \tag{11}$$

Combining Equations 3 and 11 gives the following expression for the above condition:

$$\psi_{GCL} \leq \psi_{CCL} \tag{12}$$

where: ψ_{GCL} = permittivity of the GCL; and ψ_{CCL} = permittivity of the CCL.

In other words, if the permittivity of the GCL is less than, or equal to, the permittivity of the CCL, the GCL is superior, or equivalent, to the CCL regarding advective flow control, regardless of the leachate head. According to Equation 3, the permittivity of the standard CCL defined in Section 2.5 is:

$$\psi_{CCL} = \frac{1 \times 10^{-9}}{0.6} = 1.7 \times 10^{-9}\ \text{s}^{-1} \tag{13}$$

Combining Equations 12 and 13 gives the following conditions for a GCL to be superior, or equivalent, to a standard CCL (t_{CCL} = 0.6 m and $k_{CCL} = 1 \times 10^{-9}$ m/s) *for all leachate heads*:

$$\psi_{GCL} \leq 1.7 \times 10^{-9}\ \text{s}^{-1} \tag{14}$$

The equivalency conditions expressed analytically by Equation 9 (*for leachate heads smaller than 0.3 m*) and by Equation 10 (*for all leachate heads*) have been expressed numerically for a range of GCL thickness values (Table 4). Comparing the requirements presented in Table 4 and the typical GCL hydraulic conductivity values presented in Table 2, it appears that, in most cases, the currently available GCLs control leachate migration better than CCLs. This conclusion is, of course, consistent with the conclusion drawn from Table 3, since Equations 9 and 10 used to establish Table 4 were derived from Equation 6 which was used to develop Table 3.

Table 4. Maximum hydraulic conductivity and permittivity required for a GCL to be equivalent, or superior, to a standard CCL.

Thickness of hydrated bentonite layer in GCL, t_{GCL}		Maximum hydraulic conductivity required for a GCL to be equivalent to the standard CCL (t_{CCL} = 0.6 m, $k_{CCL} = 1 \times 10^{-9}$ m/s)	
(mm)	(m)	For all leachate heads ($0 < h < \infty$), Min [Max (k_{GCL})] (m/s)	For leachate heads less than 0.3 m ($0 < h < 0.3$ m), Max (k_{GCL}) (m/s)
5	0.005	8.3×10^{-12}	2.5×10^{-11}
6	0.006	1.0×10^{-11}	2.9×10^{-11}
7	0.007	1.2×10^{-11}	3.4×10^{-11}
8	0.008	1.3×10^{-11}	3.9×10^{-11}
9	0.009	1.5×10^{-11}	4.4×10^{-11}
10	0.010	1.7×10^{-11}	4.8×10^{-11}
Regardless of the thickness of the bentonite layer in the GCL		Maximum permittivity required for a GCL to be equivalent to the standard CCL, Max (ψ_{GCL}) = 1.7×10^{-9} s^{-1}	Not applicable

Notes: The values of Min [Max (k_{GCL})] were calculated using Equation 10 and the value of Max (k_{GCL}) using Equation 9. The value of Max (ψ_{GCL}) is from Equation 14. Note that $\psi = k/t$ (Equation 3).

Travel Time. In addition to the advective flow rate, it may be useful to determine the time necessary for leachate to flow through a soil layer. The true velocity of flow through soil, between two points located on the same flow line, is given by the following equation:

$$v' = v / n \tag{15}$$

where: v' = true velocity of flow; and n = soil porosity. Ideally the porosity used in Equation 15 should be the effective porosity, n_e, which is defined as the ratio of the portion of the pore volume where advective flow takes place, and the total volume of the considered sample. (The pore volume that corresponds to $n - n_e$ is filled with liquid that does not move because it is trapped in the tortuous geometry of the pore space, or because it is adsorbed on clay particles.) It is difficult to quantify the effective porosity because it depends on a number of parameters such as the composition and structure of the clay, the composition of the liquid that permeates the clay and the hydraulic gradient. For the sake of simplicity, the porosity, n, will be used in Equation 15. As a result, the values of the steady-state travel time calculated using Equation 18 are greater than the values that would have been calculated with an effective porosity, n_e, less than the porosity, n. From the foregoing discussion, the steady-state travel time is, at best, a simplistic calculation. Furthermore, it should be remembered that the approach used herein consists of considering only advective flow. Therefore, the steady-state travel time should mostly be viewed as a conventional way to evaluate liners.

The steady-state travel time, $\bar{t}_{sst}$, or time required for leachate to advectively flow through a liner, such as a soil layer, under steady-state flow conditions is derived as follows from Equation 15:

$$\bar{t}_{sst} = \frac{t}{v'} = \frac{n\,t}{v} \tag{16}$$

Combining Equations 1 and 16 gives:

$$\bar{t}_{sst} = \frac{n\,t^2}{k\,\Delta h} \tag{17}$$

Combining Equations 4 and 17 gives the steady-state travel time, $\bar{t}_{sst}$, in the case of gravity flow:

$$\bar{t}_{sst} = \frac{n\,t}{k(1 + h / t)} \tag{18}$$

Table 5 presents values of the steady-state travel time calculated using Equation 18 for the standard CCL and the GCLs defined in Table 2. For the standard CCL, a porosity of 30 to 40% was adopted after a review of field data. It appears in Table 5 that, for the leachate heads that typically exist in landfills (0.01 to 0.1 m), the steady-state travel time is less in the case of GCLs than in the case of CCLs.

Table 5. Steady-state travel time (years), i.e. time required for leachate to pass through a CCL or a GCL used alone (i.e. without a geomembrane), under steady-state flow conditions.

Physical characteristics:			CCL	GCL		
Thickness, t (mm)			600	5	7	9
Porosity, n (%)			30-40	68	77	82
Hydraulic conductivity, k (m/s)			1×10^{-9}	5×10^{-12}	1×10^{-11}	5×10^{-11}
	(m)	(mm)	$\bar{t}_{sst\,CCL}$	$\bar{t}_{sst\,GCL}$	$\bar{t}_{sst\,GCL}$	$\bar{t}_{sst\,GCL}$
Leachate head on top of the liner, h	0	0	5.71-7.61	21.56	17.09	4.68
	0.01	10	5.61-7.49	7.19	7.04	2.22
	0.05	50	5.27-7.02	1.96	2.10	0.71
	0.1	100	4.89-6.52	1.03	1.12	0.39
	0.3	300	3.81-5.07	0.35	0.39	0.14
	0.6	600	2.85-3.81	0.18	0.20	0.07
	1.0		2.14-2.85	0.11	0.12	0.04
	10		0.32-0.43	0.01	0.01	0.00
	∞		0	0	0	0

Notes: The tabulated steady-state travel times were calculated using Equation 18, and then converted from seconds into years. The considered CCL is the "standard CCL" defined in Section 2.5. The 30 to 40% porosity range for the CCL is typical, based on a review of field data. The fact that a CCL with a higher porosity (40%) gives a greater steady-state travel time than a CCL with a smaller porosity (30%) may seem paradoxical; this results from the fact that the same "standard" hydraulic conductivity (1×10^{-9} m/s) is used with the two porosities. The characteristics of the GCL are from Table 2.

Again, it is important to remember that the above conclusions are valid only if there is no geomembrane on top of the CCL or GCL, or if the geomembrane is ignored for design purposes. The case where the geomembrane is taken into account will be discussed in Section 4 and conclusions somewhat different will be drawn.

3.3 Equivalent Hydraulic Conductivity of Several Layers of Soil

Development of Equations. As indicated in Sections 2.5 and 2.6, the CCL or the GCL is sometimes underlain by a layer of structural fill; therefore, it is useful to study the hydraulic behavior of two-layer soil systems to evaluate the contribution of the structural fill to leachate control. Also, as indicated in Section 2.7, in some conservative designs, a GCL is used in addition to a CCL; in this case, results from the study of two-layer systems will make it possible to evaluate the relative contribution of the CCL and the GCL.

A system of two soil layers subjected to a head loss, Δh (Figure 6) is considered. The hydraulic conductivity, k_{syst}, of the system of two soil layers is as follows, according Equation 1:

$$k_{syst} = \frac{q_{syst}}{\Delta h / t_{syst}} \tag{19}$$

with:

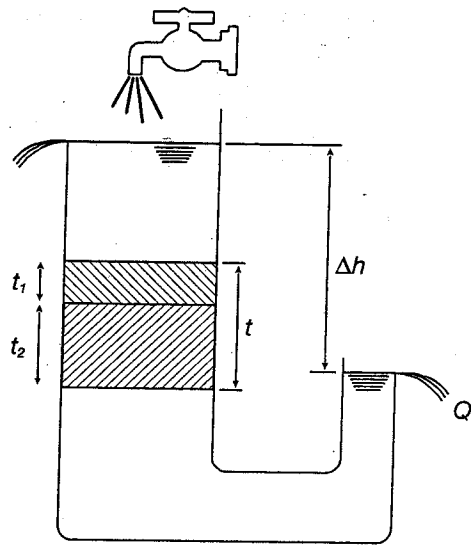

Figure 6. Flow through a two-layer soil system subjected to a given head loss Δh.

$$t_{syst} = t_1 + t_2 \tag{20}$$

where: q_{syst} = unit rate of flow through the two-layer system; t_{syst} = thickness of the system of two soil layers; t_1 = thickness of Layer 1; and t_2 = thickness of Layer 2.

The hydraulic conductivity of the system of two soil layers can be derived as follows from the hydraulic conductivities of the two soil layers. Due to flow volume conservation, the unit rate of flow is the same through each of the two layers, hence from Equation 1:

$$q_{syst} = \frac{k_{syst}\,\Delta h}{t_{syst}} = \frac{k_1\,\Delta h_1}{t_1} = \frac{k_2\,\Delta h_2}{t_2} \tag{21}$$

where: Δh_1 = head loss through Layer 1; Δh_2 = head loss through Layer 2; k_1 = hydraulic conductivity of Layer 1; and k_2 = hydraulic conductivity of Layer 2.

The head loss through the two-layer system is the sum of the head losses through each layer, hence:

$$\Delta h = \Delta h_1 + \Delta h_2 \tag{22}$$

Combining Equations 19 to 22 gives:

$$\frac{t_{syst}}{k_{syst}} = \frac{t_1 + t_2}{k_{syst}} = \frac{t_1}{k_1} + \frac{t_2}{k_2} \tag{23}$$

According to Equation 3, which defines permittivity, Equation 23 can be written as follows:

$$\frac{1}{\psi_{syst}} = \frac{1}{\psi_1} + \frac{1}{\psi_2} \tag{24}$$

where: ψ_{syst} = permittivity of the system of two soil layers; ψ_1 = permittivity of Layer 1; and ψ_2 = permittivity of Layer 2.

In the case of a system of n layers, Equations 23 and 24 can be written as follows respectively:

$$\frac{t_{syst}}{k_{syst}} = \frac{t_1 + t_2 + \cdots + t_n}{k_{syst}} = \frac{t_1}{k_1} + \frac{t_2}{k_2} + \cdots + \frac{t_n}{k_n} \tag{25}$$

$$\frac{1}{\psi_{syst}} = \frac{1}{\psi_1} + \frac{1}{\psi_2} + \cdots + \frac{1}{\psi_n} \tag{26}$$

Geotechnical engineers are familiar with Equations 23 and 25, which are classical in soil mechanics (Terzaghi 1943). However, these equations are often misunderstood and misused, as shown in the discussion below and in Sections 3.4 and 3.5.

Discussion. The flow of liquid is more impeded by two layers of soil than by one. Therefore, when there are two soil layers, the flow rate is less than in the case where there is only one of the two soil layers (which will be confirmed analytically in Section 3.4). Therefore, it may be concluded that the hydraulic conductivity of a two-layer system is less than the hydraulic conductivity of either of the two materials. However, this conclusion is incorrect as shown below.

Equation 23 can be written as follows:

$$\frac{k_{syst}}{k_1} = \frac{1 + t_2 / t_1}{1 + (t_2 / t_1) / (k_2 / k_1)} \tag{27}$$

and:

$$\frac{k_{syst}}{k_2} = \frac{1 + t_2 / t_1}{k_2 / k_1 + t_2 / t_1} \tag{28}$$

If, for example $k_1 < k_2$, the above equations show that:

$$k_1 < k_{syst} < k_2 \tag{29}$$

In other words, the hydraulic conductivity of a system of two soil layers appears to be an average (defined by Equation 23) of the hydraulic conductivities of the two soil

layers. However, the flow rate through the two-layer system (for a given head loss) is less than the flow rate through any of the two layers used alone, as shown in Section 3.4.

Example 1. A two-layer system consists of a 5 mm thick GCL with a hydraulic conductivity of 1×10^{-11} m/s and a 600 mm thick CCL with a hydraulic conductivity of 1×10^{-9} m/s. Calculate the hydraulic conductivity of the two-layer system.

According to Equation 20, the thickness of the two-layer system is:

$$t_{syst} = 5 + 600 = 605 \text{ mm} = 0.605 \text{ m}$$

Then, Equation 23 gives:

$$\frac{0.605}{k_{syst}} = \frac{0.005}{1\times10^{-11}} + \frac{0.6}{1\times10^{-9}}$$

hence:

$$k_{syst} = 5.5 \times 10^{-10} \text{ m/s}$$

It appears that $k_1 < k_{syst} < k_2$, which is in accordance with Equation 29. However, it will be seen in Example 2 that the flow rate through the two-layer system (for a given head loss) is less than the flow rate through either the GCL alone or the CCL alone.

END OF EXAMPLE 1

3.4 Flow Rate Through a Saturated Two-Layer Soil System

It is useful to evaluate the rate of flow through a two-layer soil system and to compare it to the flow through a single layer of soil. This makes it possible to determine whether it is effective to place a GCL on top of a low-permeability soil to enhance the impermeability of a liner, or to determine if a structural fill or another soil layer under a GCL can contribute to the flow barrier. In Section 3.4, it is assumed that the soil is completely saturated as shown in Figure 6.

Combining Equations 19 and 23 gives the unit flow rate through the two-layer system subjected to a head loss Δh:

$$q_{syst} = \frac{\Delta h}{\frac{t_1}{k_1} + \frac{t_2}{k_2}} \tag{30}$$

The unit rate of flow through Layer 1, q_1, when Layer 1 is subjected to the same head loss Δh as the two-layer system, is expressed as follows based on Equation 1:

$$q_1 = \frac{k_1 \, \Delta h}{t_1} \tag{31}$$

The unit rate of flow through Layer 2, q_2, when Layer 2 is subjected to the same head loss Δh as the two-layer system, is expressed as follows based on Equation 1:

$$q_2 = \frac{k_2 \, \Delta h}{t_2} \tag{32}$$

Combining Equations 3, 30, and 31 gives the ratio between the flow rates through the two-layer system and Layer 1 when both are subjected to the same head loss, Δh:

$$\frac{q_{syst}}{q_1} = \frac{t_1 / k_1}{t_1 / k_1 + t_2 / k_2} = \frac{1/\psi_1}{1/\psi_1 + 1/\psi_2} \tag{33}$$

Combining Equations 3, 30, and 32 gives the ratio between the flow rates through the two-layer system and Layer 2 when both are subjected to the same head loss, Δh:

$$\frac{q_{syst}}{q_2} = \frac{t_2 / k_2}{t_1 / k_1 + t_2 / k_2} = \frac{1/\psi_2}{1/\psi_1 + 1/\psi_2} \tag{34}$$

From Equations 33 and 34:

$$\frac{q_{syst}}{q_1} + \frac{q_{syst}}{q_2} = 1 \tag{35}$$

It is therefore clear that $q_{syst} < q_1$ and $q_{syst} < q_2$, although k_{syst} is between k_1 and k_2 (Equation 29).

Example 2. The same two-layer system as in Example 1 is considered. Calculate the ratio between the rate of flow through the two-layer system and each of its two components acting alone under the same hydraulic head loss.

Equation 33 gives:

$$\frac{q_{syst}}{q_1} = \frac{0.005 / 1 \times 10^{-11}}{0.005 / 1 \times 10^{-11} + 0.6 / 1 \times 10^{-9}} = 0.455$$

Equation 34 gives:

$$\frac{q_{syst}}{q_2} = \frac{0.6 / 1 \times 10^{-9}}{0.005 / 1 \times 10^{-11} + 0.6 / 1 \times 10^{-9}} = 0.545$$

It appears that $q_{syst}/q_1 + q_{syst}/q_2 = 1$, which is consistent with Equation 35.

END OF EXAMPLE 2

In Example 2, the rates of flow through each of the two components used alone are of the same order and are approximately twice the rate of flow through the two-layer system, even though Example 1 had shown that the hydraulic conductivity of the two-layer system is greater than that of the GCL. From Examples 1 and 2, the rankings of hydraulic conductivities and flow rates can be summarized as follows:

$$k_1 < k_{syst} < k_2$$

$$q_{syst} < q_2 < q_1$$

Clearly, the ranking of flow rates is not the same as the ranking of permeabilities because the thicknesses of the three considered soil liners (t_1, t_2 and t_{syst}) are different.

3.5 Gravity Flow Through a Two-Layer Soil System

Development of Equations. The conditions considered in Section 3.4 are of academic interest only, since they are not the conditions that typically exist in a landfill. When candidate liners are compared in landfill design, they are not subjected to the same hydraulic head loss, Δh, but they are subjected to the same head on top of the uppermost liner, h (Figure 7). In other words, the conditions are those of gravity flow described in Section 3.2. In this case, the head loss is given by Equation 4 (assuming the two layers are saturated), and the unit flow rate is given by Equation 5 as follows:

- unit flow rate through the two-layer system:

$$q_{syst} = k_{syst}\left(\frac{h+t_1+t_2}{t_1+t_2}\right) \tag{36}$$

- unit flow rate through Layer 1 used alone:

$$q_1 = k_1\left(\frac{h+t_1}{t_1}\right) \tag{37}$$

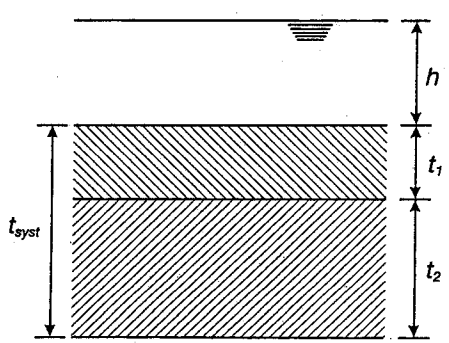

Figure 7. Gravity flow through a two-layer system.

- unit flow rate through Layer 2 used alone:

$$q_2 = k_2\left(\frac{h+t_2}{t_2}\right) \tag{38}$$

Combining Equations 23 and 36 gives the unit flow rate through the two-layer system as follows:

$$q_{syst} = \frac{h+t_1+t_2}{\dfrac{t_1}{k_1}+\dfrac{t_2}{k_2}} \tag{39}$$

Equation 39 can be used to calculate the flow rate through a two-layer system which consists of a GCL placed on a CCL or other soil layer.

Example 3. The same two-layer system as in Examples 1 and 2 is considered. Calculate the ratio between the flow rate through the two-layer system and the flow rate through the GCL alone if the head of leachate above the GCL is 100 mm.

Equation 37 gives the flow rate through the GCL used alone as follows:

$$q_{GCL} = 1 \times 10^{-11}\left(\frac{100+5}{5}\right) = 2.1 \times 10^{-10}\ \text{m/s}$$

Equation 39 gives the flow rate through the two-layer system as follows:

$$q_{syst} = \frac{0.1+0.005+0.6}{\dfrac{0.005}{1\times10^{-11}}+\dfrac{0.6}{1\times10^{-9}}} = 6.4 \times 10^{-10}\ \text{m/s}$$

The ratio between the two flow rates is:

$$\frac{q_{syst}}{q_{GCL}} = \frac{6.4\times10^{-10}}{2.1\times10^{-10}} = 3.05$$

The fact that the calculated value of the flow rate through the two-layer system is greater than the calculated value of the flow rate through one of the layers is discussed below.

END OF EXAMPLE 3

Discussion. In Example 3, two liners are compared: (i) a two-layer system which consists of a GCL underlain by a CCL; and (ii) the same GCL used alone. In both cases, the head of leachate above the GCL is the same. It appears in Example 3 that the calculated rate of flow through the two-layer system is three times greater than the calculated rate of flow through the GCL used alone (i.e. without the CCL). This result is paradoxi-

cal. To understand why this result was obtained, it is useful to consider the pressure and head diagram (Figure 8) which was established assuming that both layers of soil are saturated.

It appears in Figure 8 that, if Layer 1 is less permeable than Layer 2 (which is the case if Layer 1 is a GCL), Point A may be on the left of Point B because the head loss, Δh_1, through Layer 1 is greater than the head loss, Δh_2, through Layer 2. (However, depending on thicknesses and hydraulic conductivities, Point A may be on the right of Point B, even if Layer 1 is less permeable than Layer 2.) If indeed Point A is on the left of Point B, the minimum liquid pressure, p_{min}, in Layer 1 is negative (Figure 8) and the liquid pressure is negative in the entire Layer 2. This negative pressure is not compatible with the assumption of complete saturation of the two layers of soil. The condition for the pressure in the soil to be positive is derived below.

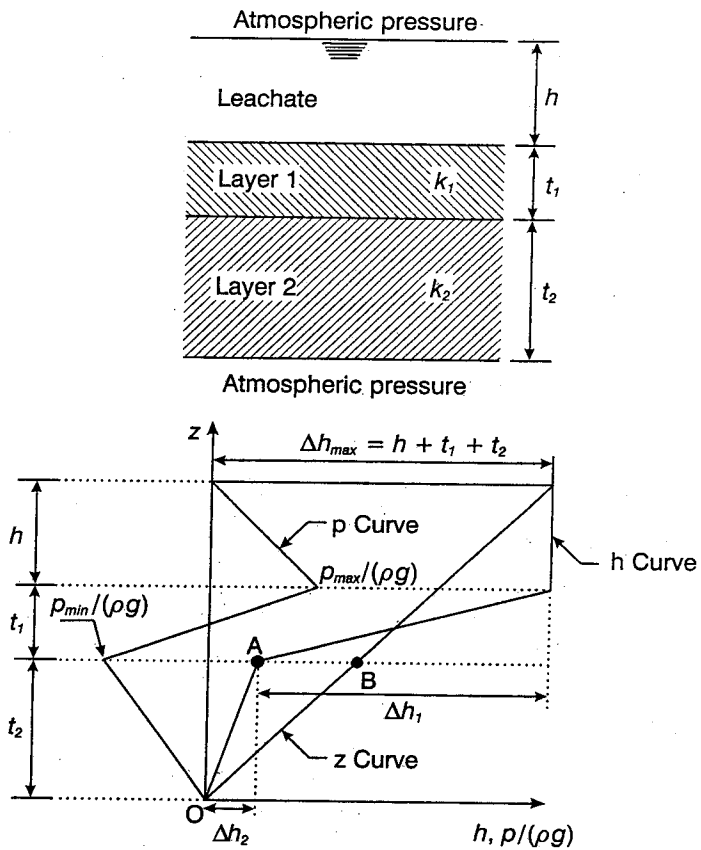

Figure 8. Pressure and head diagram for gravity flow through a two layer system assuming that both soil layers are saturated.

Figure 8 is based on the classical equation that defines the head loss:

$$\Delta h = p / (\rho g) + z \tag{40}$$

where: p = liquid pressure; ρ = liquid density; g = acceleration due to gravity; and z = altitude above the reference level used to define the head loss. In Figure 8, for the sake of simplicity, the reference level is the bottom of Layer 2, and the atmospheric pressure is used as the reference value for the pressure.

According to Equation 40:

$$p_{min} / (\rho g) = \Delta h - \Delta h_1 - t_2 \tag{41}$$

According to Figure 8 or Equation 22:

$$\Delta h - \Delta h_1 = \Delta h_2 \tag{42}$$

Combining Equations 41 and 42 gives:

$$p_{min} / (\rho g) = \Delta h_2 - t_2 \tag{43}$$

Combining Equations 21, 39 and 43, and reducing give:

$$\frac{p_{min}}{\rho g} = t_2 \left[\frac{1 + h / t_1 - k_2 / k_1}{k_2 / k_1 + t_2 / t_1} \right] \tag{44}$$

The pressure in the soil layer is positive if $p_{min} > 0$, hence:

$$k_2 / k_1 < 1 + h / t_1 \tag{45}$$

It is interesting to note that the thickness of the lower layer, t_2, is not a factor of the condition expressed by Equation 45.

Equation 45 can be written as follows:

$$\mathrm{Max}\left(k_2 / k_1\right) = 1 + h / t_1 \tag{46}$$

If the upper layer of the two-layer system is a GCL and the lower layer is a soil with a hydraulic conductivity k_s, Equation 46 becomes:

$$\mathrm{Max}\left(k_s / k_{GCL}\right) = 1 + h / t_{GCL} \tag{47}$$

hence:

$$\mathrm{Max}\left(k_s\right) = k_{GCL}\left(1 + h / t_{GCL}\right) \tag{48}$$

where Max (k_s) is the maximum value of the hydraulic conductivity of a soil underlying a GCL to ensure that the soil is saturated and contributes to the effectiveness of the hydraulic barrier.

If the soil underlying the GCL has a hydraulic conductivity, k_s, greater than Max (k_s), it cannot be saturated by advective flow passing through the GCL under the considered head, h, on top of the GCL. Values of Max (k_s/k_{GCL}) and Max (k_s) are given in Table 6. It appears that, for the leachate heads typically considered (and even for heads substantially larger), the soil underlying the GCL must have a very low permeability to be saturated. For example, under a GCL with a thickness of 7 mm and a hydraulic conductivity of 1×10^{-11} m/s, to be saturated under a leachate head on top of the GCL of 100 mm, the CCL must have a hydraulic conductivity less than 1.5×10^{-10} m/s. This requirement becomes 2.4×10^{-11} m/s if a leachate head of 10 mm is considered. These required hydraulic conductivities are too low to be met by a CCL. Therefore, it is clear that placing a low-permeability soil (even a good CCL) under a GCL provides no benefit regarding advective flow control. In other words, the hydraulic conductivity of GCLs is so low that virtually any soil layer (even most CCLs) underlying a GCL *acts as a drainage layer*; it does not become saturated and it does not contribute to the hydraulic barrier.

The fact that adding a low-permeability soil layer beneath a GCL provides no benefit regarding advective flow control (compared to the case of the same GCL used alone) should not lead the reader to conclude that placing a GCL on top of a low-permeability soil provides no benefit. Since the hydraulic conductivity of a GCL is much less than that of a low-permeability soil, placing a GCL on top of a low-permeability soil provides a significant benefit regarding advective flow control. As a result, placing a GCL on top of a CCL is an effective method for improving liner performance, compared to the case of a CCL used alone. This method is used mostly in the case of composite liners where a GCL is placed between the geomembrane and the CCL to improve the composite liner performance in critical areas such as sumps.

Table 6. Condition for the soil underlying a GCL to be saturated.

GCL characteristics: Thickness, t_{GCL} (mm)			5	7	9
Hydraulic conductivity, k_{GCL} (m/s)			5×10^{-12}	1×10^{-11}	5×10^{-11}
	(m)	(mm)	Max (k_s) (m/s)	Max (k_s) (m/s)	Max (k_s) (m/s)
Leachate head on top of the liner, h	0	0	5.0×10^{-12}	1.0×10^{-11}	5.0×10^{-11}
	0.01	10	1.5×10^{-11}	2.4×10^{-11}	1.0×10^{-10}
	0.05	50	5.5×10^{-11}	8.1×10^{-11}	3.3×10^{-10}
	0.1	100	1.1×10^{-10}	1.5×10^{-10}	6.1×10^{-10}
	0.3	300	3.1×10^{-10}	4.4×10^{-10}	1.7×10^{-9}
	0.6	600	6.1×10^{-10}	8.7×10^{-10}	3.4×10^{-9}
	1.0		1.0×10^{-9}	1.4×10^{-9}	5.6×10^{-9}
	10		1.0×10^{-8}	1.4×10^{-8}	5.6×10^{-8}
	∞		∞	∞	∞

Notes: The tabulated values of Max (k_s) were calculated using Equation 48. The characteristics of the GCL are from Table 2.

To further understand why a paradoxical result was obtained in Example 3, it is appropriate to calculate the ratio between the rate of flow through the two-layer system and the rate of flow through the upper layer of the system if it were used alone. Combining Equations 36 and 39 gives:

$$\frac{q_{syst}}{q_1} = \frac{h + t_1 + t_2}{h + t_1} \frac{t_1 / k_1}{t_1 / k_1 + t_2 / k_2} \tag{49}$$

After some mathematical manipulation, this ratio becomes:

$$\frac{q_{syst}}{q_1} = 1 - \frac{(t_2 / t_1)(1 + h / t_1 - k_2 / k_1)}{(1 + h / t_1)(k_2 / k_1 + t_2 / t_1)} \tag{50}$$

It appears that q_{syst} is smaller than q_1 only if the condition expressed by Equation 45 is satisfied. Therefore, the paradoxical result obtained in Example 3 can now be explained. The condition expressed by Equation 45 was not satisfied in Example 3 since $k_2/k_1 = 100$ and $1 + h/t_1 = 21$. Therefore, it was not appropriate to use Equation 39 in Example 3.

When the condition expressed by Equation 45 is not satisfied, the lower layer of the two-layer system is not saturated, and the pore pressure in the lower layer of the two-layer system is the atmospheric pressure. As a result, only the upper layer should be considered in flow rate calculations. Accordingly, the flow rate through the two-layer system should then be calculated using Equation 37 instead of Equation 39.

Example 4. The same case as in Example 3 is considered. Calculate the flow rate through the two-layer system.

First, the condition expressed by Equation 45 should be checked:

$$k_2 / k_1 = 1 \times 10^{-9} / 1 \times 10^{-11} = 100$$

$$1 + h / t_1 = 1 + 100 / 5 = 21$$

Since 100 is greater than 21, the condition is not satisfied. Therefore, neither Equation 39 nor Equation 50 can be used. Consequently, the lower layer should be ignored and the flow rate is given by Equation 37 as follows:

$$q_{syst} = q_1 = 1 \times 10^{-11} \left(\frac{100 + 5}{5} \right) = 2.1 \times 10^{-10} \text{ m / s}$$

It should be noted that, if Equation 50 had been used, it would have given:

$$\frac{q_{syst}}{q_1} = 1 - \frac{(600 / 5)(1 + 100 / 5 - 100)}{(1 + 100 / 5)(100 + 600 / 5)} = 3.05$$

This is identical to the value obtained in Example 3. Again, this value is paradoxical for the reasons explained above.

END OF EXAMPLE 4

4 FLOW THROUGH COMPOSITE LINERS

4.1 Introduction

As indicated in Section 2.8, GCLs used in landfills are always used as the low-permeability soil component of composite liners. In other words, GCLs used in landfills are always associated with a geomembrane. The cases discussed in Section 3 were only relevant to the extreme design scenario where the geomembrane is ignored, and to other containment structures where GCLs may be used without a geomembrane.

In Section 4, the geomembrane is not ignored and the effectiveness of composite liners constructed with CCLs and GCLs is compared.

4.2 Rate of Leachate Migration Through Composite Liners With CCL and GCL

Development of Equation. As indicated by Giroud and Bonaparte (1989), liquid migration through a composite liner occurs essentially through defects of the geomembrane. According to Giroud (1997), the rate of liquid migration through a defect in the geomembrane component of a composite liner is given by the following semi-empirical equation:

$$Q = 0.21\,[1 + 0.1(h/t)^{0.95}]\,a^{0.1}\,h^{0.9}\,k^{0.74} \tag{51}$$

where: Q = flow rate through one geomembrane defect; h = head of liquid above the geomembrane; t = thickness of the soil component of the composite liner; a = defect area; and k = hydraulic conductivity of the soil component of the composite liner. It is important to note that Equation 51 can only be used with the following units: a (m^2), h (m), t (m), k (m/s).

As discussed in Sections 2.5 and 2.6, there are cases where it is prescribed by regulations, or simply envisioned by design engineers, to place a GCL on a layer of soil with a low hydraulic conductivity such as 1×10^{-8} or 1×10^{-7} m/s. An important conclusion from Section 3, is that, if a GCL is placed on a soil layer (even a soil layer with low permeability), the soil layer has no influence on leachate advective flow and only the GCL should be considered in leachate flow calculations. The same conclusion applies to the soil component of a composite liner. Accordingly, if, in a composite liner, a GCL is placed on a layer of low-permeability soil, only the GCL will be considered in Equation 51.

Using Equation 51, the ratio between the rate of leachate flow through a composite liner with a CCL and a composite liner with a GCL is as follows:

$$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = \frac{0.21N\,[1 + 0.1(h/t_{CCL})^{0.95}]\,a^{0.1}\,h^{0.9}\,k_{CCL}^{0.74}}{0.21N\,[1 + 0.1(h/t_{GCL})^{0.95}]\,a^{0.1}\,h^{0.9}\,k_{GCL}^{0.74}} \tag{52}$$

where: $q_{comp\ CCL}$ = unit rate of flow through a composite liner where the soil component is a CCL; $q_{comp\ GCL}$ = unit rate of flow through a composite liner where the soil component is a GCL; t_{CCL} = thickness of the CCL in the composite liner; t_{GCL} = thickness of the GCL

in the composite liner; and N = number of geomembrane defects per unit area. After simplification, Equation 52 becomes:

$$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = \left(\frac{k_{CCL}}{k_{GCL}}\right)^{0.74} \frac{1 + 0.1(h/t_{CCL})^{0.95}}{1 + 0.1(h/t_{GCL})^{0.95}} \tag{53}$$

Discussion. It appears that the leachate flow rate ratio expressed by Equation 53 does not depend on the number and the size of defects. Numerical values of $q_{comp\ CCL}/q_{comp\ GCL}$ calculated using Equation 53 are presented in Table 7. It appears that, for leachate heads typically encountered in landfills (i.e. heads smaller than 0.3 m, and generally smaller than 0.1 m), the calculated advective flow control performance of a composite liner which consists of a geomembrane on a GCL is significantly better than the calculated advective flow control performance of the standard composite liner which consists of a geomembrane on the standard CCL (i.e. a CCL with a thickness of 0.6 m and a hydraulic conductivity of 1×10^{-9} m/s). Table 7 also shows that a composite liner with a GCL outperforms the standard composite liner for leachate heads up to approximately 1 to 7 m depending on the GCL hydraulic conductivity; such large heads should be a very rare occurrence in a landfill since they would correspond to a major malfunction of the leachate collection and removal system.

Table 7. Ratio between rates of advective flow through a composite liner including a CCL and a composite liner including a GCL, $q_{comp\ CCL}/q_{comp\ GCL}$.

GCL characteristics: Thickness, t_{GCL} (mm) Hydraulic conductivity, k_{GCL} (m/s)			5 5×10^{-12}	7 1×10^{-11}	9 5×10^{-11}
	(m)	(mm)	$q_{comp\ CCL}/q_{comp\ GCL}$	$q_{comp\ CCL}/q_{comp\ GCL}$	$q_{comp\ CCL}/q_{comp\ GCL}$
Leachate head on top of the liner, h	0	0	50.44	30.20	9.18
	0.01	10	42.36	26.54	8.28
	0.05	50	26.92	18.50	6.14
	0.1	100	18.87	13.66	4.71
	0.3	300	9.01	6.98	2.54
	0.6	600	5.31	4.23	1.58
	1.0		3.59	2.89	1.09
	3.0		1.65	1.35	0.52
	5.0		1.23	1.01	0.39
	7.0		1.04	0.85	0.33
	10		0.90	0.74	0.28
	∞		0.53	0.44	0.17

Notes: The tabulated values of the advective flow rate ratio were calculated using Equation 53 with the following CCL characteristics: thickness, t_{CCL} = 0.6 m; and hydraulic conductivity, $k_{CCL} = 1 \times 10^{-9}$ m/s. (This is the standard CCL defined in Section 2.5.) The characteristics of the GCL are from Table 2.

It is interesting to compare Table 3 (for the case with no geomembrane) and Table 7 (for the case with a geomembrane, i.e. the case of a composite liner), as well as comparing the corresponding equations, Equations 6 and 53, respectively. Although Tables 3 and 7 show similar trends, the calculated flow rate ratios can be significantly different whether the geomembrane component of the composite liner is ignored (Table 3) or not (Table 7). Design engineers preparing equivalency demonstrations comparing a composite liner including a GCL and a composite liner including a CCL (see Sections 1, 2.8 and 2.9) often perform calculations where the presence of the geomembrane is ignored, i.e. they use equations such as Equation 6. It is recommended to use the new tool provided by Equation 51, which does not ignore the geomembrane component of the composite liner and provides a more accurate equivalency demonstration.

4.3 Travel Time for Composite Liner With CCL and GCL

Development of Equation. As indicated in Section 3.2, the steady-state travel time is the time required for leachate to advectively flow through a barrier under steady-state conditions, and it is expressed by Equation 16. Combining Equations 1 and 16 gives:

$$\bar{t}_{sst} = \frac{n\,t}{k\,i} \tag{54}$$

In the case of a composite liner, the average hydraulic gradient in the wetted area of the low-permeability soil component (i.e. the area of the low-permeability soil component where flow occurs) is given as follows by Giroud (1997):

$$i = 1 + 0.1\,(h/t)^{0.95} \tag{55}$$

Combining Equations 54 and 55 (and using the subscript *comp* for composite liner) gives:

$$\bar{t}_{sst\ comp} = \frac{n\,t}{k\left[1 + 0.1\,(h/t)^{0.95}\right]} \tag{56}$$

Discussion. It should be noted that there is a fundamental difference between the steady-state travel time for a composite liner (i.e. the steady-state travel time calculated using Equation 56) and the steady-state travel time for a soil layer used without a geomembrane (i.e. the steady-state travel time calculated using Equation 18). In the case of a soil layer used without a geomembrane, flow occurs over the entire soil layer surface area, whereas, in the case of a composite liner, flow occurs only in the "wetted area" associated with a geomembrane defect (Giroud 1997) which, in the case of a geomembrane-GCL composite liner, is the "hydrated area" (Giroud et al. 1997). In the case of a soil layer used without a geomembrane, the steady-state travel time is the same over the entire soil layer surface area, whereas, in the case of a composite liner, the steady-state travel time is infinite outside the wetted area and has the value calculated using Equation 56 in the wetted area. It should be noted that the steady-state travel time calculated using Equation 56 is an average steady-state travel time in the wetted area since the gradient expressed by Equation 55 is an average hydraulic gradient over the "wetted area" (whereas the case of a soil layer without a geomembrane is simpler, since the hydraulic gradient is uniform over the entire soil layer surface area).

The above discussion leads to another demonstration of Equation 56 which helps explain the flow mechanism in a composite liner. The apparent velocity, v, in Equation 16 that defines steady-state travel time is equal to the flow rate divided by the surface area of the wetted area, A_{wa} :

$$v = \frac{Q}{A_{wa}} \tag{57}$$

According to Giroud et al. (1997):

$$A_{wa} = 0.21\ a^{0.1}\ h^{0.9}\ k^{-0.26} \tag{58}$$

Then, combining Equations 16, 51, 57 and 58 indeed gives Equation 56.

It should be noted that the steady-state travel time is not always identical to the breakthrough time, which is the time it takes for leachate to first pass through a barrier. For example, in the case of a geomembrane-GCL composite liner, prior to the first occurrence of leakage, the bentonite of the GCL is quasi-dry and, therefore, highly permeable. It may then be inferred that the breakthrough time is very short. Indeed it takes only a few days (Giroud et al. 1997) for leachate flowing through a geomembrane defect to hydrate the bentonite over a certain area associated with the considered defect. Therefore, in this case, the breakthrough time is short, whereas the steady-state travel time (which corresponds to the very slow leachate flow through the hydrated bentonite) is very long. In contrast, in the case of the CCLs and in the case of GCLs that are hydrated with water before they are in contact with leachate, the breakthrough time and the steady-state travel times are virtually identical.

The use of the average hydraulic gradient in the wetted area (as noted above, before Equation 55) to calculate the steady-state travel time may appear to be questionable. In fact, one should remember that, as stated in Section 3.2, the steady-state travel time should be mostly regarded as a conventional way to evaluate a liner, which justifies the use of an average hydraulic gradient, whereas the calculation of the breakthrough time would require the use of the maximum gradient in the considered area.

Numerical Application. Equation 56 was used to establish Table 8. It appears in Table 8 that the steady-state travel time depends mostly on the leachate head on top of the geomembrane: the greater the leachate head, the smaller the steady-state travel time. For leachate heads less than 100 mm, the most common leachate heads in landfills, the calculated steady-state travel time is generally greater in the case of a composite liner with a GCL than in the case of a composite liner with a CCL. The opposite conclusion was drawn in Section 3.2 from steady-state travel time calculated for the soil layer without a geomembrane (Table 5). Therefore, design engineers who prepare equivalency demonstrations and have, to this time, calculated steady-state travel times of composite liners ignoring the beneficial effect of the geomembrane, should now use Equation 56 which provides a more accurate equivalency demonstration.

Table 8. Steady-state travel time (years) in the case of a composite liner, i.e. time required for leachate to pass through a geomembrane defect and through the underlying hydrated GCL under steady-state flow conditions.

Physical characteristics:			CCL	GCL		
Thickness, t (mm)			600	5	7	9
Porosity, n (%)			30-40	68	77	82
Hydraulic conductivity, k (m/s)			1×10^{-9}	5×10^{-12}	1×10^{-11}	5×10^{-11}
	(m)	(mm)	$\bar{t}_{sst\ comp\ CCL}$	$\bar{t}_{sst\ comp\ GCL}$	$\bar{t}_{sst\ comp\ GCL}$	$\bar{t}_{sst\ comp\ GCL}$
	0	0	5.71-7.61	21.56	17.09	4.68
	0.01	10	5.70-7.59	18.07	14.99	4.21
	0.05	50	5.65-7.54	11.40	10.37	3.10
Leachate head on top of the liner, h	0.1	100	5.61-7.47	7.92	7.59	2.36
	0.3	300	5.43-7.24	3.66	3.76	1.23
	0.6	600	5.19-6.92	2.06	2.17	0.73
	1.0		4.91-6.55	1.32	1.41	0.48
	10		2.33-3.11	0.16	0.17	0.06
	∞		0	0	0	0

Notes: The tabulated steady-state travel times were calculated using Equation 56, and then converted from seconds into years. The considered CCL is the "standard CCL" defined in Section 2.5; its characteristics are discussed in the footnote of Table 5. The characteristics of the GCL are from Table 2. It is interesting to note that the limit values of the steady-state travel times for $h = 0$ are identical to the limit values for $h = 0$ in the case where the CCL or GCL is used alone, i.e. without a geomembrane (see Table 5 for $h = 0$). The fact that greater steady-state travel times are obtained for the CCL with a porosity of 40% than with the GCL with a porosity of 30% is explained in the footnote of Table 5.

Example 5. A composite liner that consists of a geomembrane overlying a GCL is considered for a landfill. Tests have shown that, under the compressive stress applied on the geomembrane in the landfill, the GCL will have the following characteristics: thickness, $t_{GCL} = 6$ mm; porosity, $n_{GCL} = 0.7$; and hydraulic conductivity, $k_{GCL} = 8 \times 10^{-12}$ m/s. The applicable regulation prescribes a composite liner whose low-permeability soil component is a CCL with a thickness, $t_{CCL} = 0.6$ m, and a hydraulic conductivity, $k_{CCL} = 1 \times 10^{-9}$ m/s, and specifies that the maximum leachate head be 0.3 m. Determine if the considered composite liner can be considered equivalent to the composite liner prescribed by the regulation, from the viewpoint of advective flow.

The calculations will be performed using both the traditional method (presented in Section 3), which consists of ignoring the geomembrane, and the method proposed in Section 4, which accounts for the effect of the geomembrane. Furthermore, the calculations will be made for three values of the leachate head: 0.3 m, which is the maximum leachate head permitted by the applicable regulation; 0.05 m, which is a more realistic value of the head, i.e. a value that should rarely be exceeded during a normal year; and 1.0 m, which corresponds to a gross malfunction of the leachate collection and removal system. The detailed calculations are presented below only for the 0.3 m head. A porosity value of $n_{CCL} = 0.35$ is used for the CCL since porosities of CCLs are between 30 and 40%(see Table 5).

The flow rate ratio between a CCL and a GCL (both without a geomembrane) can be calculated using Equation 6 as follows:

$$\frac{q_{CCL}}{q_{GCL}} = \frac{(1\times10^{-9})(1+0.3/0.6)}{(8\times10^{-2})(1+0.3/0.006)} = 3.68$$

The flow rate ratio between a composite liner consisting of a geomembrane on a CCL and a composite liner consisting of a geomembrane on a GCL can be calculated using Equation 53 as follows:

$$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = \left(\frac{1\times10^{-9}}{8\times10^{-12}}\right)^{0.74} \frac{1+0.1\,(0.3/0.6)^{0.95}}{1+0.1(0.3/0.006)^{0.95}} = 7.33$$

The steady-state travel time when there is no geomembrane can be calculated using Equation 18 as follows for the CCL and the GCL, respectively:

$$\bar{t}_{sst\ CCL} = \frac{(0.35)\,(0.6)}{(1\times10^{-9})\,(1+0.3/0.6)} = 1.40\times10^{8}\ \text{s} = 4.44\ \text{yr}$$

$$\bar{t}_{sst\ GCL} = \frac{(0.70)\,(0.006)}{(8\times10^{-12})\,(1+0.3/0.006)} = 1.03\times10^{7}\ \text{s} = 0.33\ \text{yr}$$

The steady-state travel time when there is a geomembrane can be calculated using Equation 56 as follows for the geomembrane-CCL composite liner and the geomembrane-GCL composite liner, respectively:

$$\bar{t}_{sst\ comp\ CCL} = \frac{(0.35)\,(0.6)}{(1\times10^{-9})\left[1+0.1\,(0.3/0.6)^{0.95}\right]} = 2.00\times10^{8}\ \text{s} = 6.33\ \text{yr}$$

$$\bar{t}_{sst\ comp\ GCL} = \frac{(0.70)\,(0.006)}{(8\times10^{-12})\left[1+0.1\,(0.3/0.006)^{0.95}\right]} = 1.03\times10^{8}\ \text{s} = 3.26\ \text{yr}$$

The values calculated above as well as the corresponding values for the two other leachate heads, 0.05 m and 1.0 m, are given in Table 9. The following conclusions can be drawn from this design example:

- For any of the leachate heads considered in Table 9, the $q_{comp\ CCL}/q_{comp\ GCL}$ ratio is significantly greater than the q_{CCL}/q_{GCL} ratio. This shows that neglecting the presence of the geomembrane penalizes a geomembrane-GCL composite liner when it is compared to a geomembrane-CCL composite liner.
- For all of the leachate heads considered, $q_{comp\ CCL}$ is greater than $q_{comp\ GCL}$ which indicates that the typical geomembrane-GCL composite liner considered controls the leachate advective flow better than the standard geomembrane-CCL composite liner used as a reference.

Table 9. Results of Example 5.

Equivalency calculation	Leachate head (m)	Without geomembrane (soil liner)	With geomembrane (composite liner)
Flow rate ratio	0.05	$\frac{q_{CCL}}{q_{GCL}} = 14.51$	$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = 20.55$
	0.3	$\frac{q_{CCL}}{q_{GCL}} = 3.68$	$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = 7.33$
	1.0	$\frac{q_{CCL}}{q_{GCL}} = 1.99$	$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = 2.98$
Steady-state travel time	0.05	$\bar{t}_{sst\ CCL} = 6.15$ yr $\bar{t}_{sst\ GCL} = 1.78$ yr	$\bar{t}_{sst\ comp\ CCL} = 6.60$ yr $\bar{t}_{sst\ comp\ GCL} = 9.52$ yr
	0.3	$\bar{t}_{sst\ CCL} = 4.44$ yr $\bar{t}_{sst\ GCL} = 0.33$ yr	$\bar{t}_{sst\ comp\ CCL} = 6.33$ yr $\bar{t}_{sst\ comp\ GCL} = 3.26$ yr
	1.0	$\bar{t}_{sst\ CCL} = 2.50$ yr $\bar{t}_{sst\ GCL} = 0.10$ yr	$\bar{t}_{sst\ comp\ CCL} = 5.73$ yr $\bar{t}_{sst\ comp\ GCL} = 1.20$ yr

- The values of $\bar{t}_{sst\ comp\ GCL}$ are much larger than the value of $\bar{t}_{sst\ GCL}$ whereas the difference between $\bar{t}_{sst\ comp\ CCL}$ and $\bar{t}_{sst\ CCL}$ is smaller, which provides one more indication that neglecting the presence of the geomembrane penalizes a geomembrane-GCL composite liner when it is compared to a geomembrane-CCL composite liner.
- $\bar{t}_{sst\ comp\ GCL}$ is greater than $\bar{t}_{sst\ comp\ CCL}$ for small heads and smaller for large heads.

END OF EXAMPLE 5

5 CONCLUSIONS

As GCLs are increasingly accepted as replacement for CCLs in landfill liners, calculations comparing the effectiveness of liner systems incorporating GCLs versus liner systems incorporating CCLs are performed more and more often by design engineers. The calculations are called "equivalency demonstrations" when they are performed to demonstrate to a regulatory agency that a liner system incorporating a GCL is equivalent to a conventional liner system prescribed by a regulation.

In some cases, a GCL is placed on top of a layer of low-permeability soil. Some calculations intended to evaluate the effectiveness of such an association have given a paradoxical result: the addition of a layer of low-permeability soil under the GCL appears to lead to an increase in the calculated rate of leachate flow through the GCL. The analysis presented in this paper has explained this paradoxical result by showing that classical equations for liquid flow through systems that consist of several layers are generally used beyond their limits of validity. An important consequence of the analysis is to demonstrate that, from the viewpoint of controlling the advective flow of leachate, it is ineffective to place under a GCL a layer of material with a hydraulic conductivity greater

than that of the GCL, even if this material is a low-permeability soil. However, for other reasons such as puncture resistance and chemical attenuation, it may be necessary to place a soil layer below the GCL.

Whereas adding a low-permeability soil layer beneath a GCL provides no benefit regarding advective flow control when compared to the GCL alone, adding a GCL on top of a low-permeability soil provides a significant benefit regarding advective flow control when compared to the low-permeability soil alone. As a result, placing a GCL between the CCL and the geomembrane components of a composite liner is an effective method for improving the performance of a geomembrane-CCL composite liner. This has been done in several landfills in the United States, as an extra-precaution, in critical areas such as sumps.

An equation was presented that makes it possible to compare the effectiveness of a geomembrane-GCL composite liner and a geomembrane-CCL composite liner. A numerical application of this equation shows that typical geomembrane-GCL composite liners control leachate flow more effectively than a geomembrane-CCL composite liner where the CCL is 0.6 m thick and has a hydraulic conductivity of 1×10^{-9} m/s for heads of leachate on top of the geomembrane up to 1 to 7 m depending on the GCL hydraulic conductivity.

Another type of comparison, although less important, consists of comparing the steady-state travel time, i.e. the time required for leachate to advectively flow through the liner under steady-state flow conditions. This comparison is typically done by ignoring the presence of the geomembrane, i.e. by calculating the steady-state travel time as if the CCL or the GCL was used alone. A new equation to calculate the steady-state travel time of leachate through a composite liner has been developed and is presented in this paper.

Engineers performing equivalency demonstrations comparing a composite liner with a CCL and a composite liner with a GCL often ignore the presence of the geomembrane for the sake of simplicity. As shown in this paper, such comparisons penalize the composite liner with a GCL. The new equations presented in this paper make it possible to compare the effectiveness of a composite liner with a CCL and a composite liner with a GCL, by comparing flow rates and steady-state travel times without having to ignore the essential contribution of the geomembrane, thereby providing a method for a more accurate equivalency demonstration. (See Table 10 for a summary of the important equations developed in this paper.)

It is important to note that the comparisons between GCLs and CCLs presented in this paper only consider the ability of liner materials to control advective flow of leachate. Other mechanisms such as diffusion and attenuation were not considered this paper, although they may be important. In addition, the "standard CCL" considered in the comparisons presented in this paper is essentially consistent with US regulations for municipal solid waste landfills (i.e. the CCL is 0.6 m thick and has a hydraulic conductivity of 1×10^{-9} m/s). Consequently, the results of the comparisons presented herein are not necessarily valid for other CCLs, including other "standard CCLs" prescribed in other regulations. Of course, the equations presented in this paper can be used to perform similar comparisons considering any CCL.

Table 10. Summary of important equations.

Calculation	Without geomembrane (soil liner)	With geomembrane (composite liner)
Flow rate ratio	Equation 6 $\frac{q_{CCL}}{q_{GCL}} = \frac{k_{CCL}(1 + h/t_{CCL})}{k_{GCL}(1 + h/t_{GCL})}$	Equation 53 $\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = \left(\frac{k_{CCL}}{k_{GCL}}\right)^{0.74} \frac{1 + 0.1(h/t_{CCL})^{0.95}}{1 + 0.1(h/t_{GCL})^{0.95}}$
Steady-state travel time	Equation 18 $\bar{t}_{sst} = \frac{nt}{k(1+h/t)}$	Equation 56 $\bar{t}_{sst\ comp} = \frac{n\ t}{k\left[1+0.1\ (h/t)^{0.95}\right]}$

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NOTATIONS

Basic SI units are given in parentheses.

A	= surface area of soil layer (m^2)
A_{wa}	= surface area of wetted area, i.e. the area over which interface flow extends in the case of a composite liner (m^2)
a	= defect area (m^2)
g	= acceleration due to gravity (m/s^2)
h	= head of liquid (leachate) on top of the liner (m)
i	= hydraulic gradient (dimensionless)
k	= hydraulic conductivity of soil (m/s)
k_{CCL}	= hydraulic conductivity of CCL (m/s)
k_{GCL}	= hydraulic conductivity of GCL (m/s)
k_s	= hydraulic conductivity of a soil layer underlying a GCL (m/s)
k_{syst}	= hydraulic conductivity of a system of two or more soil layers (m/s)
k_1	= hydraulic conductivity of Layer 1 (m/s)
k_2	= hydraulic conductivity of Layer 2 (m/s)
Max (k_{GCL})	= maximum value of the hydraulic conductivity of a GCL for the GCL to be equivalent to a given CCL (m/s)
Max (k_s)	= maximum value of the hydraulic conductivity of a soil underlying a GCL to ensure that the soil is saturated and contributes to the effectiveness of the hydraulic barrier (m/s)

Max (k_s/k_{GCL}) = maximum value of the k_s/k_{GCL} ratio to ensure that the soil underlying the GCL is saturated and contributes to the effectiveness of the hydraulic barrier (dimensionless)

Min [Max (k_{GCL})] = minimum value of Max (k_{GCL}), i.e. maximum value of the hydraulic conductivity of a GCL for the GCL to be equivalent to a given CCL regardless of leachate head (m/s)

N = number of geomembrane defects per unit area (m^{-2})

n = soil porosity (dimensionless)

n_e = effective porosity (dimensionless)

n_{GCL} = porosity of bentonite in GCL (dimensionless)

p = liquid (leachate) pressure (Pa)

p_{min} = minimum liquid pressure in Layer 1 (Figure 8) (Pa)

Q = flow rate (m^3/s)

q = unit rate of flow (i.e. flow rate per unit area) (m/s)

q_{CCL} = unit rate of flow through a CCL (m/s)

$q_{comp\ CCL}$ = unit rate of flow through a composite liner where the soil component is a CCL (m/s)

$q_{comp\ GCL}$ = unit rate of flow through a composite liner where the soil component is a GCL (m/s)

q_{GCL} = unit rate of flow through a GCL (m/s)

q_{syst} = unit rate of flow through a two-layer system (m/s)

q_1 = unit rate of flow through Layer 1 when it is subjected to the same head loss Δh as the two-layer system (m)

q_2 = unit rate of flow through Layer 2 when it is subjected to the same head loss Δh as the two-layer system (m)

t = thickness of soil layer (m)

t_{CCL} = thickness of CCL (m)

t_{GCL} = thickness of GCL (m)

t_{syst} = thickness of a system of two or more soil layers (m)

t_1 = thickness of Layer 1 (m)

t_2 = thickness of Layer 2 (m)

$\bar{t}_{sst}$ = steady-state travel time, i.e. time required for liquid (leachate) to flow through a liner (s)

$\bar{t}_{sst\ CCL}$ = steady-state travel time in the case of a CCL (s)

$\bar{t}_{sst\ GCL}$ = steady-state travel time in the case of a GCL (s)

$\bar{t}_{sst\ comp}$ = steady-state travel time in the case of a composite liner (s)

$\bar{t}_{sst\ comp\ CCL}$ = steady-state travel time for a geomembrane-CCL composite liner (s)

$\bar{t}_{sst\ comp\ GCL}$ = steady-state travel time for a geomembrane-GCL composite liner (s)

v = apparent velocity of leachate flow (m/s)

v' = velocity of leachate flow (m/s)

z = altitude above the reference level used to define the head loss (m)

Δh = head loss (m)

Δh_1 = head loss through Layer 1 (m)

Δh_2 = head loss through Layer 2 (m)

ψ = permittivity of a soil layer (s^{-1})

ψ_{CCL} = permittivity of CCL (s^{-1})

ψ_{GCL} = permittivity of GCL (s^{-1})

ψ_{syst} = permittivity of a system of two or more soil layers (s^{-1})

ψ_1 = permittivity of Layer 1 (s^{-1})

ψ_2 = permittivity of Layer 2 (s^{-1})

ρ = liquid (leachate) density (kg/m^3)

ATTACHMENT 16

SECTION 03300

CAST-IN-PLACE CONCRETE

PART 1 GENERAL

1.01 SUMMARY

This Section includes cast-in-place concrete, including reinforcement, concrete materials, mix design, placement procedures, and finishes.

1.02 SUBMITTALS

A. Product Data: For each manufactured material and product indicated.

B. Design Mixes: For each concrete mix indicated.

C. Shop Drawings: Include deatails of steel reinforcement placement including materials, grade, bar schedules, strirup spacing, bent bar diagrams, arrangement, and supports.

1.03 QUALITY ASSURANCE

A. Manufacturer Qualification: A firm experienced in manufacturing ready-mixed concrete products complying with ASTM C 94 requirements for production facilities and equipment.

B. Comply with ACI 301, "Specification for Structural Concrete," including the following, unless modified by the requirements of the Contract Documents.

1. General requirements, including submittals, quality assurance, acceptance of structure, and protection of in-place concrete.
2. Formwork and form accessories.
3. Steel reinforcement and supports.
4. Concrete mixtures.
5. Handling, placing, and constructing concrete.
6. Lighweight concrete.

Part 2 PRODUCTS

2.01 MATERIALS

A. Formwork: Furnish formwork and form accessories according to ACI 301

B. Steel Reinforcement:

1. Reinforcing Bars: ASTM A 615/A 615M, Grade 60, deformed.
2. Plain-Steel Welded Wire Fabric: ASTM A 185, fabricated from as-drawn steel wire into flat sheets.

C. Concrete Materials:

1. Portland Cemet: ASTM C 150, Type I or II or I/II.
2. Normal-Weight Aggregate: ASTM C 33, uniformly graded, not exceeding 1-1/2 inch nominal size.
3. Water: Complying with ASTM C-94

D. Admixtures:

1. Air-Entraining Admixture: ASTM C 260
2. Water-Reducing Admixture: ASTM C 494, Type A.
3. Water-Reducing and Retarding Admixture: ASTM C 494, Type D.

E. Vapor Retarder: Polyethylene sheet, ASTM D 4397, not less than 10 mils thick.

F. Joint-Filler Strips: ASTM D 1751, asphalt-saturated cellulosic fiber.

G. Curing Materials:

1. Evaporation Retarder: Waterborne, monomelecular filem forming, manufactured for application to fresh concrete.
2. Absorptive Cover: AASHTO M 182, Class 2, burlap cloth made from jute or kenaf.
3. Moisture-Retaining Cover: ASTM C 171, polyethylene film or white burlap-polyethylene sheet.
4. Water: Potable.
5. Clear, Waterborne, Membrane-Forming Curing Compound: ASTM C 309, Type 1, Class B.
6. Clear, Waterborne, Membrane-Forming Curing and Sealing Compound: ASTm C 1315, Type 1, Class A.

2.02 CONCRETE MIXES

A. Comply with ACI 301 requirements for concrete mixtures.

B. Prepare design mixes, proportioned according to ACI 301, for normal-weight

concrete determined by either laboratory trail mix or field test data bases, as follows:

1. Compressive Strenght (28 Days): 3000 psi as shown on the drawing.

2. Slump: 4 inches±1 inch.
   a. Slump Limit for Concrete Containing High-Range Water-Reducing Admixture: Not more than 8 inches after adding admixture to plant-or-site-verified, 2-3-inch slump.

C. Add air-entraining admixture at manufacturer's prescribed rate to result in concrete at point of placement having an air content of 2.5 to 4.5 percent.

D. Maximum water/cemet ratio: 0.55.

2.03 CONCRETE MIXING

A. Ready-Mixed Concrete: Comply with ASTM C-94.

1. When air temperature is between 85 and 90 F, reduce mixing and delivery time from 1-1/2 hours to 75 minutes; when air temperature is above 90 deg F, reduce mixing and delivering time to 60 minutes.

B. Provide batch ticket for each batch discharged and used in the Work, indicating Project identification name and number, date, mix type, mix time, quality, and amount of water added. Record approximate location of final deposit in structure.

PART 3 EXECUTION

3.01 INSTALLATION, GENERAL

A. Prior to concreting obtain written directions from coating manufacturer for acceptable curing process and follow written directions.

B. Formwork: Design, construct, erect, shore, brace, and maintain formwork according to ACI 301.

C. Vapor Retarder: Install, protect and repair vapor-retarder sheets according to ASTM E 1643; place sheets in postion with longest dimension parallel with direction of pour.

1. Lap joints 6 inches and seal with manufactur's recommended tape.

D. Steel Reinforcement: Comply with CRSI's "Manual of Standard Pratice" for fabricating, placing and supporting reinforcement.

1. Do not cut or puncture vapor retarder. Repair damage and reseal vapor retarder before placing concrete.

E. Joints : Construct joints true to line with faces perpendicular to surface plane of concrete.

1. Construction Joints: Locate and install so as not to impair strength or appearance of concrete, at location indicated or as approved by Engineer.

2. Isolation Joints: Install joint-filler strips at junction with slabs-on-grade and vertical surfaces, such as column pedestals, foundation walls, grade beams, and other locations, as indicated.

   a. Extend joint fillers full width and depth of joint, terminating flush with finished concrete surface, unless otherwise indicated.

F. Tolerances: Comply with ACI 117, "Specifications for Tolerances for Concrete Construction and Materials."

3.02 CONCRETE PLACEMENT

A. Comply with recommendations in ACI 304R for Measuring,mixing,transporting, and placing concrete.

B. Consolidate concrete with mechancial vibrating equipment.

3.03 FINISHING FORMED SURFACES

A. Related Unformed Surfaces: At tops of walls, horizontal offsets, and similar unformed surfaces adjacent to formed surfaces, strike off smooth and finish with a texture matching adjacent formed surfaces. Continue final surface treatment of formed surfaces uniformly across adjacent unformed surfaces, unless otherwise indicated.

3.04 FINISHING UNFORMED SURFACES

A. General : Comply with ACI 302.1R for screeding, restraightening, and finishing operations for concrete surfaces. Do not wet concrete surfaces.

B. Screed surfaces with a straightedge and strike off. Begin initial floating using bull floats or darbies to form a uniform and open-textured surface plane before excess moisture or bleedwater appears on the surface.

1. Do not further disturb before starting finishing operations.

C. Scratch Finish: Apply scratch finish to surface to receive concrete floor topping or mortar setting beds for ceramic or quarry tile, Portland cement terrazzo, and other bonded cementitious floor finish, unless otherwise indicated.

D. Float Finish: Apply float finish to surface indicated and to floor and slab surfaces exposed to view or to be covered with fluid-applied or sheet waterproofing, built-up or membrance roofing , or sand-bed terrazzo.

E. Trowel Finish: Apply a hard trowel finish to surfaces indicated and to floor and slab surfaces exposed to view or to be covered with resilient flooring,carpet, ceramic or quarry tile set over a clevage membrane, paint, or another thin film-finish coating system.

F. Trowel and Fine-Broom Finish: Apply a partial trowel finish, stopping after second troweling, to surface indicated and to surfaces where cermic or quarry tile is to be installed by either thickset or thin-set methods. Immediately after second troweling, and when concrete is still plastic, slightly scarify surface with a fine broom.

G. Nonslip Broom Finish: Apply a nonslip broom finish to surface indicated and to exterior concrete platforms, steps, and ramps. Immediately after float finishing, slightly roughen trafficked surface by brooming with fiber-bristle broom perpendicular to main traffic route.

3.05 CONCRETE PROTECTION AND CURING

A. Prior to concreting, obtain written directions from coating manufacturer for acceptance during process and follow written directions.

B. General: Protect freshly placed concrete from premature drying and excessive cold or hot temperatures. Comply with ACI 306.1 for cold-weather protection, and follow recommendations in ACI 305R for hot-weather protection during curing.

C. Evaporation Retarder: Apply evaporation retarder to concrete surfaces if hot, dry, or windy conditions occur before and during finishing operations. Apply according to manufacturer's written instructions after placing, screeding, and bull floating or darbying concrete, but before float finishing.

D. Begin curing after finishing concrete, but not before free water has disappeared from concrete surface.

E. Cure formed and unformed concrete for at least seven days as follows:

1. Moisture Curing: Keep surfaces continuously moist with water or absorptive cover, water saturated and kept continuously wet.

2. Moisture-Retaining-Cover Curing: Cover concrete surface with moisture-retaining cover for curing concrete, placed in widest practicable width, with sides and ends lapped at least 12 inches, and sealed by waterproof tape or adhesive. Immediately repair any holes or tears during curing peroid using cover material and waterproof tape.

3.06 FIELD QUALITY CONTROL

A. Testing Agency: Owner will engage a qualifed independent testing and inspecting agency to sample materials, perform tests, and submit test reports during concrete placement. Test will be performed according to ACI 301.

1. Testing Frquency: One test for each days's pour of each concrete mix exceeding 5 cu. yd., but less than 25cu. yd., plus one set for each additional 50 cu.yd. or fraction thereof.

2. Testing shall consist of the following:

   a. Slump test per ASTM C 143.
   b. Air entrainment test per ASTM C 173.
   c. Compressive strength test per ASTM C 38 (4 cylinders).

[END OF SECTION]

ATTACHMENT 17

SECTION 02200

EARTHWORK

PART 1 GENERAL

1.01 SCOPE

A. This section includes the requirements for site preparation, excavation, surface water control, excavation dewatering, stockpiling, subgrade preparation, general fill, and earthwork materials. This section also includes the requirements to maintain the prepared subbase surface until the geosynthetics installer has completed construction of the liner system.

1.02 RELATED SECTIONS AND PLANS

A. Section 02100 – Surveying

B. Section 02110 – Stripping

C. Section 02215 – Trenching and Backfilling

D. Section 02290 – Erosion and Sediment Control

E. Section 02930 – Vegetation

F. Construction Quality Assurance (CQA) Plan

1.03 REFERENCES

A. Latest version of American Society of Testing and Materials (ASTM) Standards.
   1. ASTM D 698. Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop.

2. ASTM D 2487 Standard Test Method for Classification of soils for Engineering Purposes.

1.04 SUBMITTALS

A. Within 15 calendar days from Notice to Proceed, submit to the Engineer for review an Earthwork Work Plan. The Earthwork Work Plan shall include, at a minimum:
   1. list of equipment proposed for the construction activities including earthwork and for scope of work specified in Sections 02215, ~~02230,~~ and 02235~~, and 02240~~;
   2. construction methods for each construction activity;
   3. dewatering methods and techniques;
   4. coordination of survey requirements for the earthwork;
   5. proposed locations of temporary soil stockpile areas;
   6. coordination of earthwork activities with surface water management and erosion and sediment control measures;
   7. schedule for earthwork activities; and
   8. dust control measures.

1.05 CONSTRUCTION QUALITY ASSURANCE

A. The earthwork will be monitored and tested by the CQA Consultant as required in the CQA Plan.

B. The CQA Consultant will perform soil conformance testing on general fill to establish compliance with this Section. The Contractor will provide equipment and labor to assist the CQA Consultant in obtaining conformance samples from excavations and stockpiles.

C. The CQA Consultant will perform soil performance testing on the subgrade surface and general fill lifts to evaluate compliance with this Section. The CQA Consultant will indicate any portion of the earthwork that does not meet the requirements of this Section and will delineate the extent of the nonconforming area.

D. The Contractor shall correct all deficiencies and nonconformances identified by the CQA Consultant at no additional cost to the Owner.

E. The Contractor shall be aware of the activities required of the CQA Consultant by the CQA Plan and shall account for these activities in the construction schedule.

1.06 EXISTING CONDITIONS

A. Existing site surface and subsurface conditions, based on available site data, are indicated on the Construction Drawings.

B. Contractor shall verify existing conditions as indicated in Section 02100.

PART 2 – PRODUCTS

2.01 MATERIALS

A. Obtain material for general fill from the existing on-site borrow sources or on-site stockpiles designated by the Engineer.

B. General fill material shall be free of debris, foreign objects, large rock fragments, organics, and other deleterious materials. General fill material shall classify as SW, SP, SC or SM according to the Unified Soil Classification System (per ASTM D 2487). General fill material having the indicated classification is expected to be available from designated borrow sources. Soils having other classifications are acceptable as general fill, if approved by the Engineer.

C. General fill material used in the top 6-inches of the prepared subgrade shall be free of sharp materials or any materials larger than one-half inch.

2.02 EQUIPMENT

A. Furnish compaction equipment to achieve the required minimum soil dry density within the range of acceptable moisture contents.

B. Furnish hand compaction equipment, such as a walk-behind compactor, hand tampers, or vibratory plate compactor, for compaction in areas inaccessible to large compaction equipment.

C. Furnish water trucks, pressure distributors, or other equipment designed to apply water uniformly and in controlled quantities to variable surface widths for required in-place moisture adjustment, to prevent drying of soil surfaces, and for dust control.

D. Furnish equipment such as excavators, scrapers, compactors, loaders, dozers, earth hauling equipment and all other equipment, as required for earthwork construction.

PART 3 EXECUTION

3.01 GENERAL

A. All general fill material to be compacted shall be at a moisture content that will readily facilitate effective compaction.

3.02 SITE PREPARATION

A. Install construction fence and barricades around open trenches and excavated areas.

B. Install erosion and sediment controls in relevant areas of construction and as required by Section 02290. Maintain the erosion and sediment controls for the duration of the Contract and until the contained areas are vegetated in accordance with Section 02930. Accumulated sediment behind silt fences and from drainage swales and structures shall be removed as required or as directed by the Engineer.

C. Prior to any earthwork activity, perform stripping as indicated on the Construction Drawings and in accordance with Section 02110.

3.03 SURAFCE WATER CONTROL

A. Installation of surface water and erosion controls shall be in accordance with approved Surface Water Management and Erosion Control Plan as specified in Section 02290.

B. Install surface water and erosion controls in and around work areas to control runoff and erosion and to prevent surface water runon into excavations. Perimeter controls may include shallow ditches, berms, or localized regrading.

3.04 EXCAVATION

A. Excavate designated areas to the subgrade elevations or excavation limits indicated on the Construction Drawings. Stockpile excavated material in areas designated by the Construction Manager for use in subsequent construction.

3.05 EXCAVATION DEWATERING

A. Anticipate seepage of groundwater into, and accumulation of surface water runoff in excavations. Manage groundwater and surface water in excavations in accordance with this section.

B. Prevent surface water run-on from adjacent areas from entering the excavation.

C. All fill operations, except hydraulic filling, shall be performed in the dry. Contractor shall expect that groundwater is at or near the existing ground surface and shall be prepared to lower the groundwater in local areas as required to construct sumps and drainage structures. Contractor shall expect that work areas may be inundated with water and be prepared to dewater locally as required to perform work.

3.06 STOCKPILING

A. Separate stockpiles by material type.

B. Stockpile excavated soils from on-site borrow sources at the areas indicated on the Construction Drawings or as designated by the Engineer.

C. Construct stockpiles no steeper than 3H:1V (horizontal:vertical), grade to drain, seal by tracking perpendicular to the slope contours with a dozer, and dress daily during periods when fill is taken from the stockpile.

D. Silt fence or berms shall be constructed at the base of stockpiles that will not be immediately used.

E. Restore all areas used for stockpiling when stockpiles are removed.

3.07 SUBGRADE PREPARATION

A. Subgrade material shall consist of soil relatively free of debris, foreign objects, organics and other deleterious materials.

B. Compact all subgrade within the limits of flexible leachate storage containers to a minimum 95 percent of the Standard Proctor (ASTM D 698) maximum dry density at a moisture content approved by the Engineer.

C. Perform subgrade proof rolling by driving a loaded dump truck (minimum weight of 10 tons per axle and minimum loaded weight of 20 tons) or other pneumatic-tired vehicle, back and forth across the area to confirm the firmness of subgrade surface. Overlap the passes such that one set of tires on each pass runs between the two sets of tire tracks from the previous pass. The surface ~~Soils~~ shall not exhibit pumping ~~or develop~~ and not contain loose stones or ruts more than ~~two~~ one inches in depth. Minor rutting, defined as less than one ~~two~~ inches in depth, shall be regraded or covered with general fill to match finish grade.

D. Subgrade for general fill shall be scarified to a depth of 2 inches using equipment identified in this section.

E. Unsuitable soils shall be removed and replaced with general fill to a minimum depth of 2 feet below the proposed subgrade elevation. Suitable soil exhibiting pumping or developing ruts more than one ~~two~~ inches in depth will be removed to a minimum depth of 1 foot or dried in place, if feasible. Compact the general fill and liner subbase materials to a minimum 95 percent of standard Proctor (ASTM D 698) maximum dry density at a moisture content approved by the Engineer.

F. In excavations or other areas where water accumulates, implement measures to remove the water in accordance with this section. Maintain the subgrade surface free of standing water and in firm condition to meet proof rolling requirements of this section. Maintain dewatered areas until overlying construction is complete.

G. Manage surface water as described in Section 02290.

3.08 PREPARED SUBGRADE SOILS

A. Use fill that meets the requirements of general fill listed in this Section. Place fill to the limits and grades shown on the Construction Drawings.

B. Place general fill material on surfaces that are free of debris, vegetation, or other deleterious material.

C. Place general fill material in loose lifts with a thickness of 8 inches ± 1 inch. In areas where compaction is to be performed using hand operated equipment, place the fill material in loose lifts with a loose thickness of 4 inches ± 1 inch.

D. Prior to placing a succeeding lift of material over a previously compacted lift, thoroughly scarify the previous lift to a depth of 2 inches by discing, raking, or tracking with a dozer. Moisture condition the preceding lift if not within the acceptable moisture range.

E. The trafficking of scarified surfaces by trucks or other equipment, except compaction equipment, is not permitted.

F. Except as specified in this section, compact general fill in each lift to at least 95 percent of its standard Proctor maximum dry density (ASTM D 698). Compact general fill at moisture content as required to attain the specified density or as approved by the Engineer.

G. Do not place fill during periods of precipitation. Placement may occur during periods of misting or drizzle, but only as authorized by the Engineer.

H. Dust shall be controlled by the application of water to the general fill surfaces.

I. CONTRACTOR shall coordinate the final surface of subbase general fill within the footprint of the flexible storage containers with the geosynthetics installer. CONTRACTOR is responsible for maintenance of the subbase until acceptance by the geosynthetics installer.

3.09 SURVEY CONTROL

A. Survey limits and elevations of subgrade, excavations, and top of prepared subgrade in accordance with Section 02100.

3.10 TOLERANCES

A. Perform the earthwork construction related to the composite liner system to within ±0.1 ft. of the elevations and within 10 percent of the slopes shown or indicated on the Construction Drawings.

B. Positively draining slopes shall be maintained in all cases.

[END OF SECTION]

ATTACHMENT 18

SECTION 02740

GEOCOMPOSITE

PART 1 GENERAL

1.01 SCOPE

A. This section includes requirements for geocomposite drainage layer product and installation.

1.02 RELATED SECTIONS AND PLANS

A. Section 02770 – Geomembranes

B. Section 02780 – Geosynthetic Clay Liner

C. Construction Quality Assurance (CQA) Plan

1.03 REFERENCES

A. Latest version of American Society for Testing and Materials (ASTM) standards:

1. ASTM D 1505. Standard Test Method for Density of Plastics by the Density-Gradient Technique.
2. ASTM D 1603. Standard Test Method for Carbon Black in Olefin Plastics.
3. ASTM D 1777. Standard Method for Measuring Thickness of Textile Materials.
4. ASTM D 3786. Standard Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabric - Diaphragm Bursting Strength Tester Method.
5. ASTM D 4491. Standard Test Method for Water Permeability of Geotextiles by the Permittivity Method.
6. ASTM D 4533. Standard Test Method for Trapezoid Tearing Strength of Geotextiles.
7. ASTM D 4632. Standard Test Method for Breaking Load and Elongation of Geotextiles (Grab Method).

8. ASTM D 4716. Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products.
9. ASTM D 4751. Standard Test Method for Determining Apparent Opening Size of a Geotextile.
10. ASTM D 4833. Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
11. ASTM D 5261. Standard Test Method for Measuring Mass Per Unit Area of Geotextiles.
12. ASTM F 904. Standard Test Method for Comparison of Bond Strength or Ply Adhesion of Similar Laminates Made from Flexible Materials.

B. Federal Standard No. 751a - Stitches, Seams, and Stitching.

1.04 SUBMITTALS

A. Submit the following to the Engineer for review at least 21 calendar days prior to use:
   1. geocomposite Manufacturer and product names;
   2. certification of minimum average roll values and the corresponding test procedures for all geocomposite properties listed in Table 02740-1; and
   3. projected geocomposite delivery dates.

~~A.~~ B. Submit to the Engineer for review at least 14 calendar days prior to geocomposite placement, manufacturing quality control certificates for each roll of geocomposite as specified in this section.

~~B.~~ C. For each proposed geocomposite material, the Contractor shall submit to the Engineer for review at least 14 calendar days prior to transporting the geocomposite to site the results of manufacturing quality control testing and certification that the geocomposite is manufactured to meet the minimum interface shear strength criteria when tested in compliance with requirements of this section.

1.05 CONSTRUCTION QUALITY ASSURANCE

A. The installation of the geocomposite drainage layers will be monitored by the CQA Consultant as required by the CQA Plan.

B. The CQA Consultant will perform material conformance testing of the geocomposite as required by the CQA Plan.

C. The Contractor shall be aware of the activities required of the CQA Consultant by the CQA Plan and shall account for these activities in the installation schedule.

D. The Contractor shall correct all deficiencies and nonconformances identified by the CQA Consultant at no additional cost to the Owner.

PART 2 PRODUCT

2.01 GEOCOMPOSITE

A. Furnish geocomposite drainage layer materials consisting of a polyethylene geonet core with a needle-punched nonwoven geotextile heat laminated to each side of the geonet core.

B. Furnish geocomposite having properties meeting the required property values shown in Table 02740-1. Required geocomposites properties shall be considered minimum average roll values (95 percent lower confidence limit).

C. Furnish geocomposite that are stock products.

D. In addition to the property values listed in Table 02740-1, the geocomposite shall:
   1. retain their structure during handling, placement, and long-term service; and
   2. be capable of withstanding outdoor exposure for a minimum of 30 days with no measurable deterioration.

E. Furnish polymeric threads for stitching that are ultra-violet (UV) light stabilized to at least the same requirements as the geotextile to be sewn. Furnish polyester or polypropylene threads that have a minimum size of 2,000 denier.

2.02 MANUFACTURING QUALITY CONTROL

A. Sample and test the geotextile and geonet components of the geocomposite to demonstrate that these materials conform to the requirements of this section.

B. Perform manufacturing quality control tests to demonstrate that the geotextile properties conform to the values specified in Table 02740-1. Perform as a minimum, the following manufacturing quality control tests at a minimum frequency of once per 50,000 square feet:

Test	Procedure
Mass per unit area	ASTM D 5261
Grab strength	ASTM D 4632
Tear strength	ASTM D 4533
Puncture strength	ASTM D 4833
Burst strength	ASTM D 3786

C. Perform additional manufacturing quality control tests on the geotextile, at a minimum frequency of once per 50,000 square feet, to demonstrate that its apparent opening size (per ASTM D 4751) and permittivity (per ASTM D 4491) conform to the values specified in Table 02740-1.

D. Perform manufacturing quality control tests to demonstrate that the geonet drainage core properties conform to the values specified in Table 02740-1. Perform as a minimum, the following manufacturing quality control tests at a minimum frequency of once per 50,000 square feet:

Test	Procedure
Polymer density	ASTM D 1505
Carbon black	ASTM D 1603
Thickness	ASTM D 1777

E. Perform additional manufacturing quality control tests, at a minimum frequency of once per 50,000 square feet, to demonstrate that the geocomposite drainage layers conform to the hydraulic transmissivity (per ASTM D 4716) and peel strength (per ASTM F 904) requirements of Table 02740-1.

F. Submit quality control test certificates signed by the geotextile, geonet, and geocomposite manufacturer quality control manager. The quality control certificates shall include:
   1. lot, batch, and roll number and identification; and
   2. results of manufacturing quality control tests including description of test methods used.

G. Do not supply any geocomposite roll that does not comply with the manufacturing quality control requirements.

H. If a geotextile, geonet, or geocomposite sample fails to meet the quality control requirements of this section, sample and test rolls manufactured at the same time or in the same lot as the failing roll. Continue to sample and test the rolls until the extent of the failing rolls are bracketed by passing rolls. Do not supply failing rolls.

2.03 PACKING AND LABELING

A. The geocomposite shall be supplied in rolls wrapped in relatively impermeable and opaque protective covers.

B. Geocomposite rolls shall be labeled with the following information.
   1. Fabricator's name;
   2. product identification;
   3. lot or batch number;
   4. roll number; and
   5. roll dimensions.

C. Geocomposite rolls not labeled in accordance with this section or on which labels are illegible upon delivery to the site shall be rejected and replaced with properly labeled rolls at no additional cost to the Owner.

D. If any special handling is required, it shall be so marked on the geotextile component e.g., "This Side Up" or "This Side Against Soil To Be Retained".

2.04 TRANSPORTATION

A. Geocomposite shall be delivered to the site at least 21 days prior to the planned deployment date to allow the CQA Consultant adequate time to

perform conformance testing on the geocomposite samples as required by the CQA Plan.

2.05 HANDLING AND STORAGE

A. The Contractor shall be responsible for storage of the geocomposite at the site.

B. Handling and care of the geocomposite prior to and following installation at the site, is the responsibility of the Contractor. The Contractor shall be liable for all damage to the materials incurred prior to final acceptance by the Owner.

C. The geocomposite shall be stored off the ground and out of direct sunlight, and shall be protected from excessive heat or cold, mud, dirt, and dust. Any additional storage procedures required by the manufacturer shall be the Contractor's responsibility.

D. Protective wrappings shall be removed less than one hour prior to unrolling the geocomposite. After unrolling, a geocomposite shall not be exposed to ultraviolet light for more than 30 calendar days. Outdoor storage of geocomposite rolls shall not exceed the Manufacturer's recommendations or longer than 6 months whichever is less. For storage periods longer than 6 months a temporary enclosure shall be placed over the rolls, or they shall be moved to an enclosed facility. The location of the temporary field storage shall not be in areas where water can accumulate.

PART 3 EXECUTION

3.01 PLACEMENT

A. The Contractor shall not commence geocomposite installation until the CQA Consultant completes conformance evaluation of the geocomposite and quality assurance evaluation of previous work, including evaluation of Contractor's survey results for previous work.

B. The Contractor shall handle the geocomposite in such a manner as to ensure the geocomposite is not damaged in any way.

C. The Contractor shall take any necessary precautions to prevent damage to underlying layers during placement of the geocomposite.

D. The geocomposite shall only be cut using manufacturer's recommended procedures.

E. In the presence of wind, all geocomposite panels shall be weighted with sandbags or the equivalent. Such sandbags shall be installed during placement and shall remain until replaced with cover material.

F. Care shall be taken during placement of geocomposite not to entrap dirt or excessive dust in the geocomposite that could cause clogging of the drainage system, and/or stones that could damage the adjacent geomembrane. Care shall be exercised when handling sandbags, to prevent rupture or damage of the sandbags.

G. If necessary, the geocomposite shall be positioned by hand after being unrolled over a smooth rub sheet.

H Tools shall not be left on, in, or under the geocomposite.

I. After unwrapping the geocomposite from its opaque cover, the geocomposite shall not be left exposed for a period in excess of 30 days.

J. If white colored geotextile is used in the geocomposite, precautions shall be taken against "snowblindness" of personnel.

3.02 SEAMS AND OVERLAPS

A. The components of the geocomposite (i.e., geotextile, geonet, and geotextile) are not bonded together at the ends and edges of the rolls. Each component will be secured or seamed to the like component of adjoining panels.

B. Geotextile Components:
   1. The bottom layers of geotextile shall be overlapped a minimum of 4 inches. The top layers of geotextiles shall be continuously sewn (i.e., spot sewing is not allowed)~~. Geotextiles~~ and shall be overlapped a minimum of 6 inches prior to seaming.
   2. No horizontal seams shall be allowed higher than one-third the slope height on slopes steeper than 10 horizontal to 1 vertical.

3. Polymeric thread, with chemical resistance properties equal to or exceeding those of the geotextile component, shall be used for all sewing. The seams shall be sewn using Stitch Type 401 per Federal Standard No. 751a. The seam type shall be Federal Standard Type SSN-1.

C. The geonet component of adjacent geocomposite panels shall be overlapped a minimum of 4-inches along the geocomposite panels and 12-inches across end (butt) seams. The geonet component shall be fastened together using nylon or plastic fasteners approved by the Manufacturer. The geonet shall be fastened at a minimum spacing of 10-ft on surfaces 10:1 or flatter, every 5-ft on surfaces steeper than 10:1, and every 1-ft along end (butt) seams.

3.03 REPAIR

A. Any holes or tears in the geocomposite shall be repaired by placing a patch extending 2 ft beyond the edges of the hole or tear. The patch shall be secured by tying fasteners through the bottom geotextile and the geonet of the patch, and through the top geotextile and geonet on the slope. The patch shall be secured every 6 inches with approved tying devices. The top geotextile component of the patch shall be heat sealed to the top geotextile of the geocomposite needing repair. If the hole or tear width across the panel is more than 50 percent of the width of the panel, the damaged area shall be cut out and the two portions of the geonet shall be joined in accordance with this section.

B. All repairs shall be performed at no additional cost to the Owner.

3.04 PLACEMENT OF SOIL MATERIALS

A. The Contractor shall place all soil materials in such a manner as to ensure that:
   1. the geocomposite and underlying geosynthetic materials are not damaged;
   2. minimal slippage occurs between the geocomposite and underlying layers; and
   3. excess tensile stresses are not produced in the geocomposite.

B. Spread soil on top of the geocomposite from the bottom of slopes upward to cause the soil to cascade over the geocomposite rather than be shoved across the geocomposite.

C. For geocomposite overlying the geomembrane, do not place overlying soil material at ambient temperatures below 40 degrees Fahrenheit (F) or above 104°F, unless authorized in writing by the Engineer. For cold (<40°F) and hot (>104°F) weather placement operations, use the additional procedures authorized in writing by the Engineer.

D. Do not drive equipment directly on the geocomposite. Only use equipment above a geocomposite overlying a geomembrane that meets the following ground pressure requirements above the geomembrane:

Maximum Allowable Equipment Ground Pressure (pounds per square inch)	Minimum Thickness of Overlying Soil (inches)
<5	12
<10	18
<20	24
>20	36

TABLE 02740-1
GEOCOMPOSITE PROPERTY VALUES

PROPERTIES	QUALIFIER	UNITS	SPECIFIED VALUES (1)	TEST METHOD
Geonet Component:				
Polymer composition	Minimum	%	95 polyethylene by weight	--
Polymer density	Minimum	g/cm^3	0.93	ASTM D 1505
Carbon black content	Range	%	2 - 3	ASTM D 1603
Nominal thickness	Minimum	mil	200	ASTM D 1777 or ASTM D 5199
Geotextile Component:				
Type	None	none	needlepunched nonwoven	--
Polymer composition	Minimum	%	95 polyester or polypropylene	
Mass per unit area	Minimum	oz/yd^2	8	ASTM D 5261
Apparent opening size	Maximum	mm	$O_{95} \leq 0.21$ mm	ASTM D 4751
Permittivity	Minimum	sec^{-1}	0.5	ASTM D 4491
Grab strength	Minimum	lb	180	ASTM D 4632 (2)
Tear strength	Minimum	lb	75	ASTM D 4533 (2)
Puncture strength	Minimum	lb	75	ASTM D 4833 (3)
Burst Strength	Minimum	psi	350	ASTM D 3786
Geocomposite:				
Transmissivity	Minimum	m^2/s	5×10^{-4}	ASTM D 4716
Peel strength	Minimum	lb/in.	1.0	ASTM F 904 or GRI GC-7

Notes:
1. All values represent minimum average roll values.
2. Minimum value measured in machine and cross-machine direction.
3. Tension testing machine with a 1.75-inch diameter ring clamp, the steel ball being replaced with 0.31-inch diameter solid steel cylinder with flat tip centered within the ring clamp.
4. Transmissivity of geocomposite shall be tested with geocomposite sandwiched between geomembranes using water at 68°F with a gradient of 0.1 under compressive stress of 500 psf for 24 hours.

[END OF SECTION]

ATTACHMENT 19

SECTION 16651

CONTROL PANEL FABRICATION

PART 1 GENERAL

1.01 SCOPE

A. This specification identifies the minimum requirements for the design, fabrication and testing of the Pump Control Panels located at the impacted stormwater and leachate flexible leachate storage containers and at the impacted stormwater pumping location as indicated on the Construction Drawings. The Contractor is responsible for the functional operation of panel wiring from the main power drop to the panel and from the panel to the leachate/impacted stormwater sump pumps and various instrumentation. Panel general arrangement and construction shall be as shown on the contract drawings and indicated in the specifications. Follow the panel manufacturers written requirements and recommendations for mounting and space allocation, wiring and grounding of all equipment contained in the pump control panel. It is the intent of this specification to provide a fully operational and ready-to-use system.

B. The control panel shall be designed in accordance with the requirements of this specification, and the design drawings. No change orders will be accepted unless a specific change of scope is requested in writing by the Engineer, fully approved and executed.

C. This specification describes the functional requirements of the control panel and all internal components necessary to provide a complete and operating system.

D. The Contractor shall provide overall system integration of existing pumping equipment with the Pump Control Panel. The Contractor shall be responsible for coordination of control wiring and communications between the Pump Control Panel, pumps, level transducers, flow meters, and any other instrumentation, equipment or control panels that require communication or input/output capabilities.

1.02 RELATED SPECIFICATIONS

A. Section 16010 General Electrical Requirements.

B. Section 16170 Grounding and Bonding

C. Section 16652 Instrumentation

1.03 REFERENCES

The enclosures, wiring, and component parts of this system shall conform to the latest revision of the following codes and regulations:

A. National Electric Code (NEC), ANSI/NFPA 70

B. National Electric Safety Code (NESC), ANSI C2

C. American National Standards Institute (ANSI)

D. National Electrical Manufacturing Association (NEMA)

E. Electronics Industry Association / Telecommunications Industry Association (EIA/TIA)

F. All applicable federal, state, and local codes.

1.04 SUBMITTALS

A. The control panel manufacturer shall provide a copy of the panel design to the Engineer prior to beginning assembly of the panel. The Engineer shall review and provide written approval or required modifications prior to assembly.

B. The control panel manufacturer shall provide written documentation of functionality testing of the control panel and all instrumentation interfacing with the control panel.

PART 2 PRODUCTS

2.01 GENERAL

A. All wiring for control panel shall be provided by the contractor. Requirements shall comply with Section 16010.
B. The control panel shall be assembled by Sligo Systems, Inc. of Ormond Beach, Florida. All control panel components shall be provided by Sligo Systems, Inc.

2.02 PANEL COMPONENTS

Two Control Panels will be used for the flexible leachate storage container facility. The leachate Control Panel is intended to remotely operate the two sump pumps located in the two flexible leachate storage containers cells which are to be used for leachate. The impacted stormwater Control Panel is intended to remotely operate the two sump pumps located in the two flexible leachate storage containers~~,~~ cells which are to be used for

impacted stormwater. The pumps will consist of 2-HP submersible pumps. The Control Panels will also monitor leachate levels in the sumps and flow rates in the piping during sump pump operation.

A. All pump controls will be housed in a painted NEMA Type 4 Cabinet. Cabinet Size will be be determined by the panel manufacturer.

B. Controls will be protected from weather by placing them behind the outer door of the cabinet.

C. The Cabinet will be equipped with an appropriately sized service disconnect switch capable of de-energizing all equipment in the cabinet and all external equipment serviced by the cabinet. The service disconnect shall be accessible from the outside of the cabinet when the outer door is closed.

D. The primary sump pumps (PS-1104-01 and PS-1104-02) will be controlled by a level transducer located inside the sump. The level transducer will monitor the depth of leachate in the primary sump and will start and stop the primary leachate pumps at specific set points.

E. The two primary pumps will alternate as lead and lag with each pump activation. Set points will be such that the lead pump will start when the first high level set point (SPH-1104-01) is reached. If leachate levels in the sump continue to rise then the lag pump will start when the second high level set point (SPH-1104-02) is reached. Both primary sump pumps will be set to turn off at the same set point (SPL-1104-01).

F. The secondary sump pump will be turned on by high level set point, SPH-1104-03 and turned off by low level set point, SPH-1104-03.

G. A high-high level alarm (LAHH-1104-01) will be activated if either of the high-high level set points (SPHH-1104-01 or SPHH-1104-02) are activated. This alarm will activate a flashing strobe light on top of the control panel to notify the operator that leachate levels in the sump risers are too high.

H. A separate fused disconnect will be provided to house an external power receptacle. This separate fused disconnect shall be referred to as the booster pump disconnect. The fused disconnect and receptacle shall meet the following requirements:

   a) The booster pump disconnect and receptacle shall be housed in a NEMA 4 cabinet;

   b) The booster pump disconnect shall have an auxiliary pole to provide a control signal to the sump pump control panel. This control signal shall disable the sump pumps when the booster pump disconnect is activated.

   c) The receptacle shall be capable of handling the power loads from the following 20 hp motor:

      i) 3-phase 460 power;

      ii) Minimum efficiency 40 percent;

iii) Power factor 0.89

iv) Service factor 1.15.

d) The receptacle shall be a watertight pin and sleeve style connector.

e) The receptacle shall be accessible without opening the booster pump disconnect panel.

I. The Leachate Sump Control Panel shall be capable of communicating with the Leachate Storage Area Control Panel such that:

a) All pumps will be shut down in the event a signal is received indicating that all four leachate pump-in valves in the leachate storage area are closed.

b) All sump pumps will be shut down in the event any one of the high-high level switches in the leachate storage area has been activated.

c) A separate control panel, to be added in the future, will be able to monitor sump pump operation and limit the number of pumps operating to 4.

J. Operation of the sump pumps and the control panel shall conform to the operational notes set forth on the Process and Instrumentation Diagram.

K. Communication between the Sump Pump Control Panel and the Leachate Storage Area Control Panel shall use a 2 wire signal processing system.

a) The panel manufacturer shall be responsible for determining the appropriate number of nodes required for operation.

b) The 2 wire system installed shall be such that it can be modified in the future for radio telemetry.

L. All pumps will operate on 460VAC 3 phase power.

M. A 110VAC, 20 amp convenience outlet shall be provided at the control cabinet location.

N. A convenience light fixture shall also be provided at the control cabinet. The light shall be sufficient to illuminate the sump area and the control cabinet area. An externally mounted light switch rated for exterior installation shall be installed at the control cabinet location.

O. The control cabinets shall be shielded from direct sunlight to the extent possible by installing a fiberglass or plastic backing and roof to the control panel mounting posts.

P. Three-position switches capable of overriding the level switch operation will be provided for each pump. The each switch will be equipped with a legend plate identifying the switch position. The switch positions shall be labeled as Hand, Off and Auto corresponding to the operation of the pump at that position. The Hand position

will allow an operator to turn on the pump motor independent of the water level in the sump. The hand position will be spring loaded to prevent the switch from being left in the hand position. The Off position will allow an operator to turn off the pump motor independent of the water level in the sump. The Auto position will return the pump to control by the level switches.

Q. Each pump shall be protected by a Type E-1 current/voltage monitor. The monitor shall be set by the contractor to detect stuck impeller and no flow conditions.

R. Each pump shall have a pilot light mounted on the front of the panel. The pump control panel will be configured such that the pilot light will light when the associated pump is operating.

S. The pump control panel will be equipped with three beacon lights mounted on top of the panel box.

   a) A steady lit amber colored light shall be configured to indicate power is available to the panel. The amber light shall be lit when the main disconnect switch on the pump control panel is in the on position.

   b) A flashing red light shall be configured to indicate operational problems associated with:

      i) High or low voltage

      ii) High or low current

      iii) Water level has activated the High High Level switch (HHL-01).

   c) A flashing blue light shall be configured to indicate operational problems associated with the active level transducer.

   d) Flashing lights shall be strobe activated types. Mechanical rotating lights shall not be used.

T. Panel Wiring

   a) Wire PLC inputs and outputs to terminal blocks for field wiring connection.

U. Wireway

   a) Provide ventilated plastic wireways inside the panels for separating and organizing the wiring.

   b) Electric signals carried in one Wireway will be of similar types and voltage levels. Provide separate wireways for AC and DC wiring. Route internal wiring in separate wireway from space allowed for external field wiring. Provide each signal type with its own terminal strip.

V. Terminal Blocks

a) All fabricator wiring shall be limited to one side of the terminal strips. The other side of the terminal is reserved for field wiring connections.

W. Wire Marking

a) Permanently identify each wire at both ends with a permanent identification tag. Identify wiring according to wire identifiers on the control panel design plans provided. Wire from terminal block to terminal block without splicing.

2.03 LIGHTNING PROTECTION

A. The control cabinet location shall be protected from incoming voltage surges by an appropriately sized service entrance Transient Voltage Surge Supression (TVSS) unit. The TVSS shall be manufactured by Erico, Inc.

B. An additional TVSS unit shall be installed on the incoming communication conductors.

C. Two Lightning protection devices (lightning rods) shall be installed above the control panel to protect the system from lightning strikes.

D. Lightning protection and TVSS units shall be designed and installed by a qualified lighting protection specialist.

E. Grounding and bonding shall be accomplished in accordance with Section 16170, Grounding and Bonding.

PART 3 EXECUTION

3.01 GENERAL

A. The Control Panel shall provide system control for the proposed system as discussed in Part 2 of this specification and as depicted on the design drawings.

3.02 TESTING

A. The Control Panel will be given a complete visual inspection and fully powered point-to-point by the Contractor before notifying the Engineer that the system is ready for testing.

B. Testing will be conducted in accordance with the manufacturer's requirements. Written documentation of the field-testing shall be provided before the system is accepted by the Engineer.

C. The Contractor shall have the control cabinet installation inspected and verified by Sligo Systems, Inc. Sligo Systems shall prepare an inspection report on the cabinet installation. The inspection report shall be provided to the Engineer prior to acceptance of the panel.

D. Electrical power shall be checked by the Contractor and written documentation shall be provided indicating that the incoming power is within the limits required by the control panel, pump, and instrumentation manufacturers

3.03 FINAL INSPECTION AND COMMISSIONING

The Engineer shall inspect the panels after installation to ensure that each has been installed in accordance with this section and the contract drawings. The Contractor shall demonstrate the operation of the completed panel system to the Engineer to show that it operates as intended by the design. If system components fail or are inoperative during the testing and/or operational demonstration, they shall be repaired or replaced by the Contractor.

(END OF SECTION 16651)

ATTACHMENT 20

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1. INTRODUCTION

1.1 Overview

This Construction Quality Assurance (CQA) Plan describes the quality assurance and construction quality control (CQC) activities that will be undertaken during construction of the flexible leachate storage containers (FLSCs) at the Central County Solid Waste Disposal Complex (CCSWDC) located in Sarasota, Florida. The CCSWDC facility is owned and operated by Sarasota County Solid Waste Operations (Sarasota County). The purpose of this document is to define the scope, formal organization, and procedures necessary to achieve a high level of quality and assure that the construction of the CCSWDC FLSC facility is constructed in compliance with the approved design as shown or indicated in the Construction Drawings and Technical Specifications. This plan addresses the CQA and CQC activities to be performed during construction.

1.2 Project Description

The design and construction of FLSC facility is presented in the Construction Drawings and Technical Specifications. The project includes the following:

- construction of a double-composite liner system;
- construction of the leachate collection, removal, transmission and storage systems;
- construction of a gas management system;
- construction of the final cover system components above the FLSC surfaces;
- construction of surface water management system; and
- general site work including FLSC grading and general earthwork.

1.3 CQA Plan Scope

The CQA Plan establishes the quality assurance and quality control monitoring and testing activities to be implemented during construction of the FLSC facility. The CQA Plan was developed in consideration of the current Florida Department of Environmental Protection (FDEP) guidelines and regulations. The scope of the CQA Plan includes:

- defining the responsibilities of parties involved with the construction of the FLSC facility;
- providing guidance in the proper construction of FLSC facility components;
- establishing testing protocols for the evaluation of FLSC facility components;
- establishing procedures for construction documentation; and
- providing the means for assuring that the overall construction conforms to the Construction Drawings and Technical Specifications.

The CQA Plan is intended to establish procedures for the CQA Consultant and to inform the Contractor of CQA activities during the construction of the FLSC facility. The CQA Plan is considered a supplement to the Technical Specifications and a part of the construction contract. In the case of any conflict between the CQA procedures described in this plan and the requirements of the Technical Specifications, the Technical Specifications will govern.

1.4 CQA Plan Organization

The remainder of this CQA Plan is organized as follows:

- definitions of key terms are presented in Section 2;
- project organization and descriptions, responsibilities, and qualifications of key parties involved with the construction of the FLSC facility are presented in Section 3;
- requirements for CQA documentation are described in Section 4;
- CQA activities for the soil components of the FLSC facility, to include fill placement, liner system, final cover system, and general earthwork, are presented in Section 5;
- CQA activities for geomembranes, geosynthetic clay liner, geotextiles, and geocomposites are presented in Sections 6 through 9, respectively;
- CQA activities for piping and fittings are covered in Section 10;
- CQA activities for mechanical and electrical components are described in Section 11;
- CQA activities for concrete associated work are outlined in Section 12; and

- CQA activities for road construction and general civil site work are presented in Sections 13 and 14, respectively.

2. CQA PLAN DEFINITIONS

2.1 Construction Quality Assurance and Construction Quality Control

In the context of this document, construction quality assurance and construction quality control are defined as follows:

- Construction Quality Assurance (CQA) - The planned and systematic means and actions designed to assure adequate confidence that materials and/or services meet contractual and regulatory requirements and will perform satisfactorily in service.

- Construction Quality Control (CQC) - Those actions which provide a means to measure and regulate the characteristics of an item or service in relation to contractual and regulatory requirements.

- In the context of this document:

- CQA refers to means and actions employed by the CQA Consultant, Engineer, or Sarasota County to assure conformity of the various components of the FLSC facility construction project with the requirements of the Construction Drawings and Technical Specifications.

- CQC refers to those actions taken by the CQA Consultant, Contractor, Manufacturers, or Installers to ensure that the materials and the workmanship of the various components of the FLSC facility construction project meet the requirements of the Construction Drawings and Technical Specifications. In the case of the geosynthetic components of these systems, CQC is provided by the CQA Consultant and/or Manufacturers and Installers of the various geosynthetics.

2.2 Plans and Specifications

In this CQA Plan, reference to Construction Drawings and Technical Specifications is understood to mean those plans and specifications issued as a part of a specific contract for construction of a component or phase at the FLSC facility. In all cases, it is expected that this CQA Plan will conform to the Construction Drawings and Technical Specifications. In case of conflict, the approved Construction Drawings and Technical Specifications will govern.

2.3 Geosynthetics

Geosynthetics is the generic term for all synthetic materials used in geotechnical engineering applications; the term includes geotextiles, geogrids, geonets, geomembranes, geosynthetic clay liners (GCL), and geocomposites. There are four types of geosynthetic products referenced in this CQA Plan that are included in the FLSC facility construction. These geosynthetics include: (i) high density polyethylene (HDPE) and polyethylene (PE) geomembranes used in the liner and final cover systems, respectively; (ii) GCL used in the double-composite liner system; (iii) geotextiles used as filters or separators; and (iv) geocomposite drainage layers used in the liner and the final cover systems.

2.4 Construction Activities

In the context of this CQA Plan, the FLSC facility construction is understood to include:

- geosynthetic and soil components of the liner system;
- leachate collection, removal, transmission, and storage systems;
- geosynthetic and soil components of the final cover system above the FLSC surfaces;
- gas management system;
- surface-water management system components;
- other site work including grading and general earthwork;
- road work; and
- other construction activities as assigned by Sarasota County.

2.5 CQA Lines of Communications

Successful execution of this CQA Plan is dependent on open and continuous communication between all parties having a role in the project. The lines of communication between Sarasota County, Engineer of Record, Design Engineer,

Construction Manager, Contractor, and CQA Consultant are defined in the organization charts included in Section 3 of this CQA Plan.

3. PROJECT ORGANIZATION AND PERSONNEL

3.1 Overview

The FLSC facility construction organization chart is shown in Figure 3-1. It is understood that the Project Manager will act on behalf of the Sarasota County in all matters relating to the construction of the FLSC facility. Day-to-day construction activities at the FLSC facility will be managed through the direct interaction of several parties below Project Manager level including but not limited to the Construction Manager, Design Engineer, Contractor, and CQA Consultant. The organization chart for the FLSC facility CQA Consultant is presented in Figure 3-2. The description, qualifications, and responsibilities of the parties responsible for construction and CQA at the FLSC facility project are described below.

3.2 Construction Manager

The Construction Manager shall be an individual employed by the Project Manager and who is responsible for overall management of the construction project at the site. In this CQA plan the term "Construction Manager" shall refer specifically to an authorized representative of the Project Manager at the FLSC facility. The Construction Manager will hold a baccalaureate degree in construction management, engineering, or related field or have 10 years of construction management experience. The Construction Manager will also have 3 years of FLSC construction experience. The Construction Manager shall be responsible for coordination and oversight of all construction activities including: (i) contract administration; (ii) construction management; (iii) review of any modifications or changes to the construction contract documents; and (iv) final approval authority for contract or shop drawings and submittals.

3.3 Design Engineer

The Design Engineer is the individual representing the firm having responsibility for FLSC facility design. The Design Engineer will hold a minimum of a baccalaureate degree in engineering, be a Professional Engineer registered in the state of Florida, and have 10 years experience in construction management, engineering, or related fields. The Design Engineer shall have expertise which demonstrates significant familiarity with geosynthetics and soils, as appropriate, including design and construction experience related to FLSC liner system, and final cover system. The Design Engineer is responsible for approving all design and specification changes and making design clarifications that may be required during construction at the FLSC facility. The Design

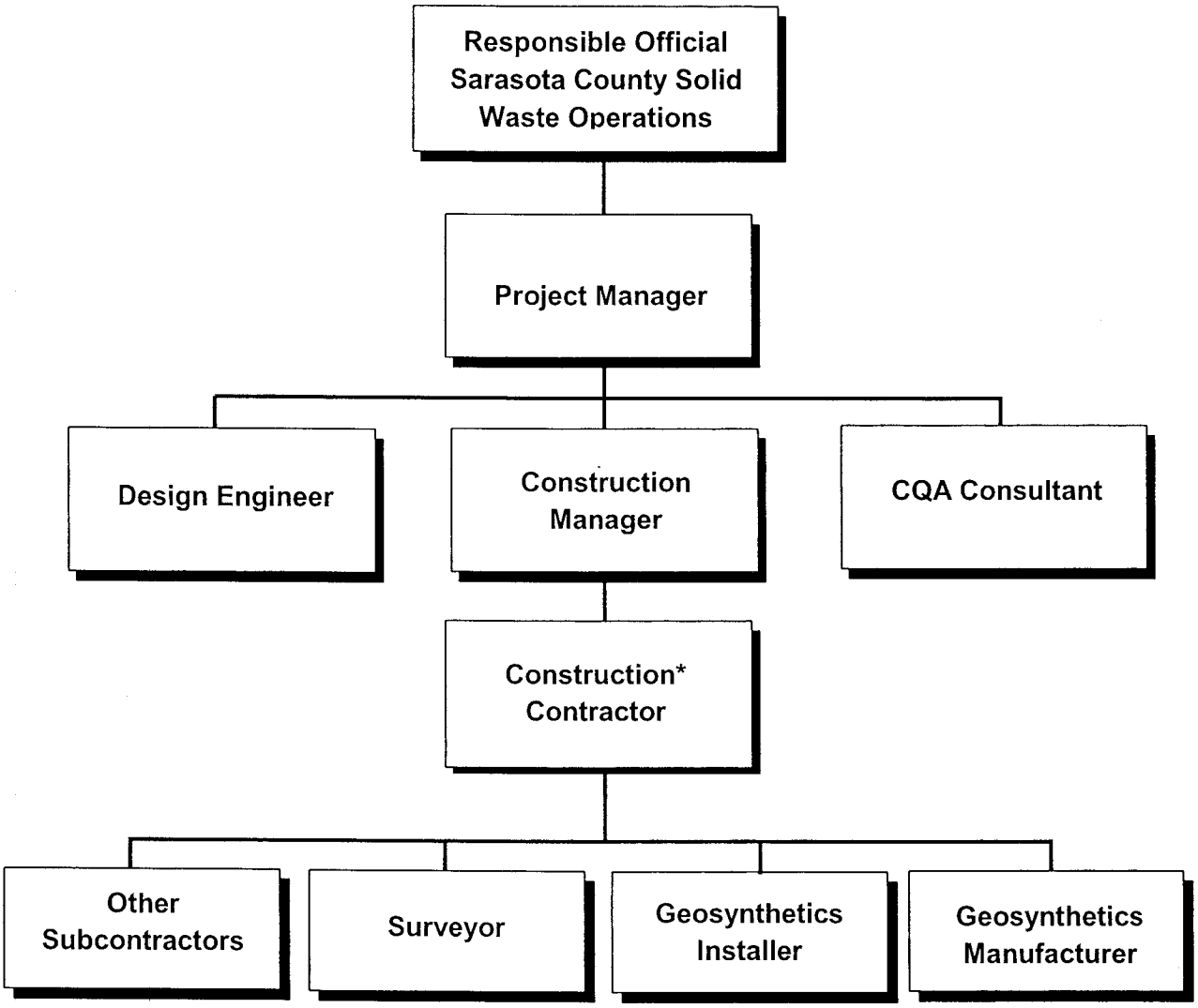
Engineer shall assist the Construction Manager in reviewing and approving the Contractor's shop drawings and submittals as necessary. The Design Engineer will not be present on-site but will visit the project during construction and attend the project coordination meetings as required to assure conformance with plans and specifications. The Design Engineer will be capable of discussing and interpreting all elements of the FLSC facility design. The Design Engineer shall have the authority to recommend changes or modifications to the Construction Drawings and Technical Specifications for approval by Sarasota County and FDEP, as required.

3.4 Contractor

The Contractor is the firm or corporation having a legally binding agreement to construct components of the FLSC facility construction, or shall be qualified construction personnel hired directly by Sarasota County and working under the direct supervision of a construction foreman and superintendent. The Contractor is represented on-site by a qualified individual who is authorized to act on behalf of the Contractor in all matters pertaining to the construction at the FLSC facility. The Contractor shall be qualified as required by the contract to perform all aspects of work required to successfully construct the project. The Contractor shall be registered in accordance with applicable local, state, and federal requirements and shall demonstrate significant prior related experience. The Contractor's field representative shall be a qualified individual who is able to perform all tasks associated with FLSC facility construction activities. The Contractor's field representative shall demonstrate experience similar to the Construction Manager. The Contractor's field representative shall have the authority to direct and instruct the Contractor's crews and its subcontractors.

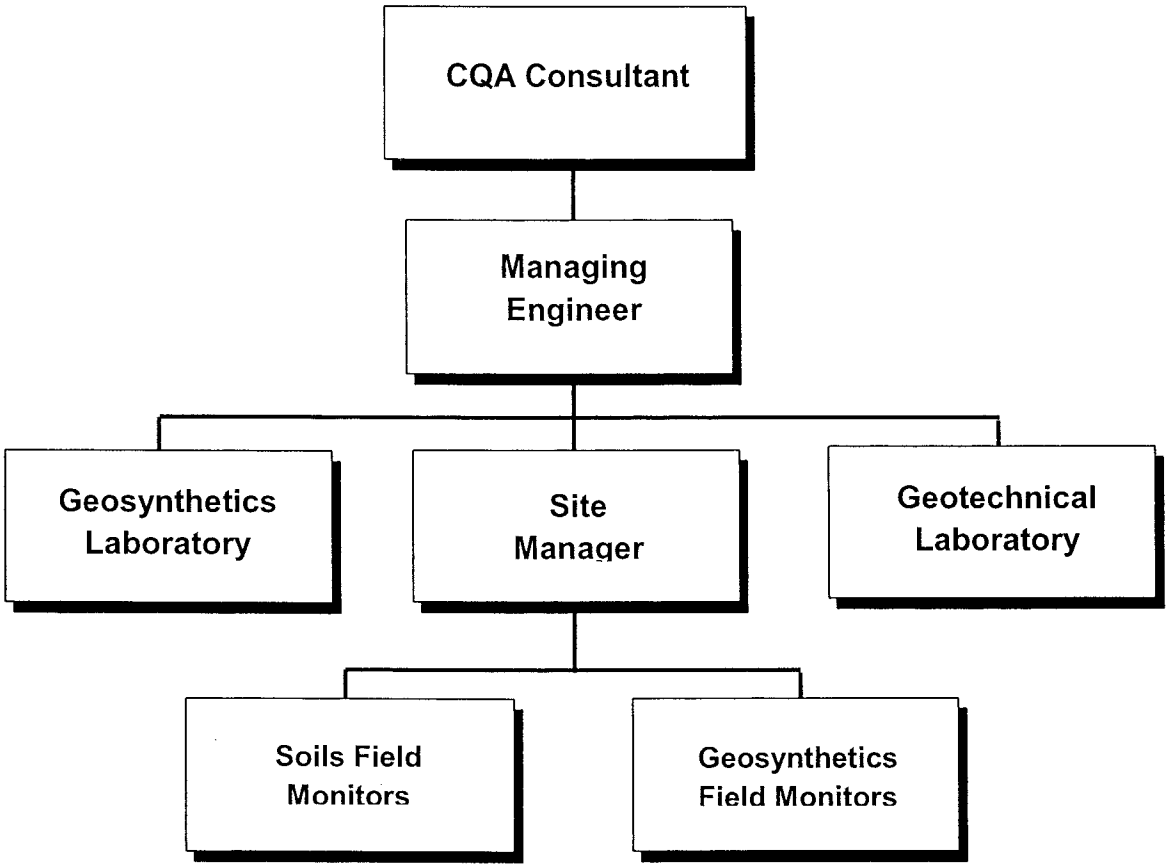
The Contractor is responsible for all construction materials and activities. The Contractor is also responsible for scheduling and coordination of the required work with its subcontractors to complete the project within the construction schedule approved by the Construction Manager. The Contractor shall provide an experienced supervisory representative at all times during any construction activity on-site. The Contractor is responsible for furnishing as-built record drawings and a copy of all documentation required during the construction at the FLSC facility. The Contractor is also responsible for updating all construction drawings for any deviations from the original plans and specifications on a regular basis.

Figure 3-1
FLSC Facility Construction Organization Chart



*The Construction Contractor is assumed to have earthwork capabilities as an integral part of the firm. Otherwise, the earthwork subcontractor is a major entity in this chart under the prime contractor.

Figure 3-2
FLSC Facility CQA Organization Chart



The Contractor's field representative is responsible for coordinating and supervising the work of all subcontractors on site. At a minimum, the Contractor's field representative will be responsible for the following:

- informing the Construction Manager of any discrepancies between the plans and specifications and the field conditions;
- submitting all documentation required by the Construction Drawings and Technical Specifications in a timely manner;
- attending all project coordination meetings held on site;
- scheduling all phases of the construction;
- maintaining a daily log of all construction activities on site;
- implementing and verifying all QC procedures required of the Contractor and/or subcontractors; and
- submitting proposed alternative materials or construction methods to the Construction Manager for approval prior to acquisition and use.

3.5 CQA Consultant

3.5.1 Definition

The CQA Consultant is the party, independent from Sarasota County and the Contractor, responsible for observing, testing, and documenting activities related to the CQA and CQC of the soil and geosynthetic components and other activities related to the construction at the FLSC facility as described in this CQA Plan.

3.5.2 Qualifications

The CQA Consultant shall be a well-established firm specializing in geotechnical engineering, liner and final cover system design, construction management, and CQA. The CQA Consultant shall possess the equipment, personnel, and licenses necessary to conduct the monitoring and testing activities required by this CQA Plan and the FLSC facility Construction Drawings and Technical Specifications. The CQA Consultant

shall also be experienced in the installation and CQA of soil and geosynthetic materials similar to those materials to be used for the FLSC facility construction. The CQA Consultant will be experienced in the preparation of CQA documentation including CQA plans, field documentation, field testing procedures, laboratory testing procedures, construction specifications for construction, construction plans, and CQA certification reports. The CQA Consultant shall provide qualified staff for the project.

In addition, the CQA Consultant shall provide the following, in writing, to Sarasota County as required:

- corporate background and information;
- a detailed summary of the firm's CQA capabilities;
- a detailed summary of the firm's CQA experience; and
- a representative list of at least 10 completed facilities for which the CQA Consultant has provided CQA monitoring services for the installation of the corresponding geosynthetic material; for each facility, the following information will be provided:
    - name and purpose of facility, its location, and date of installation;
    - name of owner;
    - surface area of each geosynthetic material installed; and
    - telephone number of person familiar with the project.

The CQA Consultant shall provide resumes of personnel to be involved in the project including:

- the CQA Managing Engineer, who operates from the office of the CQA Consultant and who conducts periodic visits to the site as required;
- the CQA Site Manager, who is located at the site; and
- the CQA Field Monitors, who will be located at the site.

The CQA Consultant organization will be led by the CQA Managing Engineer, who will hold a baccalaureate degree in engineering and be a Professional Engineer registered to practice in the state of Florida. The CQA Site Manager will be the representative of the CQA Consultant on site and will have experience in similar construction and be specifically familiar with the construction of soil and geosynthetic components of the FLSC.

3.5.3 Responsibilities

The CQA Consultant shall be responsible for monitoring and documenting the activities of the Contractor relative to the installation of the liner and final cover system components as well as various appurtenances related to the construction at the FLSC facility. The CQA Consultant will be responsible for monitoring the compliance of construction materials delivered to the site with the submittals and/or shop drawings previously reviewed and approved by the Construction Manager. The CQA Consultant shall assure that the Contractor's construction methods and workmanship are performed in accordance with the Construction Drawings and Technical Specifications. The CQA Consultant shall be responsible for obtaining and testing samples of the various construction materials in accordance with the testing frequencies identified in this plan. The CQA Consultant shall also be responsible for obtaining, labeling, and shipping samples for off-site laboratory testing in accordance with the requirements of this plan and appropriate specifications.

The CQA Consultant shall be responsible for soils quality control testing to be performed by both the on-site and off-site testing laboratories. The CQA Consultant shall be responsible for staffing and operating the on-site soils laboratory, if required. Test results from the on-site and off-site laboratories shall be submitted to the Construction Manager within a time frame that will not impede or delay construction activities.

The on-site soils laboratory, if required, shall be equipped to perform routine index testing including, but not limited to:

- standard Proctor (ASTM D 698);
- particle-size analysis (ASTM D 422 and ASTM C 136);
- Atterberg limits (ASTM D 4318);

- moisture content (ASTM D 2216 and ASTM D 4643);
- soils classification (ASTM D 2487); and
- percent passing No. 200 sieve (ASTM D 1140).

The CQA Consultant shall also be responsible for conducting routine field tests during construction of the FLSC facility, which shall include:

- moisture content by nuclear methods (ASTM D 3017);
- in-place density by nuclear methods (ASTM D 2922);
- lift thickness by direct measurement;
- sand cone (ASTM D 1556); and
- drive cylinder (ASTM D 2937).

The CQA Consultant will be responsible for the quality control of its on-site laboratory testing program and for documenting the calibration of the soils laboratory testing equipment. Equipment calibration certificates shall be maintained in the CQA Consultant's on-site project file. All tests will be conducted in accordance with ASTM or other applicable state or federal standards. Test results shall be submitted to the Construction Manager within a time frame that will not impede or delay construction of activities.

The duties of the CQA Personnel are discussed in the following subsections.

3.5.3.1 CQA Managing Engineer

The CQA Managing Engineer:

- reviews the FLSC Construction Drawings and Technical Specifications;
- reviews soils and geosynthetics-related documents (such reviews are for familiarization and for evaluation of constructibility only);
- attends project meetings related to construction quality activities;
- administers the CQA program (i.e., assigns and manages all on-site CQA

personnel, reviews all field reports, and provides engineering review of all CQA-related activities);

- provides quality control of CQA documentation;
- reviews changes to the construction design, and assures any major changes are submitted to FDEP for approval prior to incorporation into the Construction Drawing and Technical Specifications; and
- with the CQA Site Manager, prepares the final certification report.

3.5.3.2 CQA Site Manager

The CQA Site Manager:

- acts as the on-site representative of the CQA Consultant;
- familiarizes all CQA Field Monitors with the site, project documents, and the CQA requirements;
- manages the daily activities of the CQA Field Monitors;
- attends regularly scheduled CQA-related meetings on-site;
- reviews the ongoing preparation of the construction record drawings;
- reviews test results provided by the Contractor;
- verifies the calibration and condition of on-site testing equipment;
- reviews the CQA Field Monitors' daily reports and logs;
- provides reports to the Construction Manager, and documents in a daily report any reported relevant observations by the CQA Field Monitors;
- prepares a daily report for the project;
- oversees the collection and shipping of all laboratory test samples;
- reviews results of laboratory testing and makes appropriate recommendations;

- reports any unresolved deviations from the CQA Plan and Construction Drawings and Technical Specifications to the Construction Manager;
- assists with the preparation of the final certification report;
- reviews appropriate certifications and documentation from the Contractor and the Geosynthetics Manufacturer and Installer, and makes appropriate recommendations;
- reviews the Geosynthetics Manufacturer's QC documentation;
- reviews the geosynthetics Installer's personnel qualifications for conformance with those required by the Technical Specifications; and
- performs duties of CQA Field Monitor as needed.

3.5.3.3 CQA Field Monitors

The duties of the CQA Field Monitors are monitoring and documenting construction of all soils and geosynthetics components of the FLSCs and other CCSWDC facility activities, as assigned by the CQA Site Manager.

The duties of the CQA Field Monitors will include:

- monitoring material stockpiles for any deterioration of materials;
- monitoring surface-water drainage in the areas of soil and geosynthetic material stockpiles;
- preparing daily field reports;
- recording CQA and CQC activities on field logs;
- reporting problems to the CQA Site Manager;
- assisting with collection of samples from the constructed soil components in accordance with the CQA Plan;
- monitoring soil placement and compaction operations;

- monitoring the unloading and on-site handling and storage of the geosynthetics;
- monitoring geosynthetic repair operations;
- monitoring geosynthetic material deployment and installation operations; and
- collecting conformance samples for testing by CQA laboratories.

In addition to these specific duties, all CQA Field Monitors will document any on-site activities that could result in damage to the soils or geosynthetic components of the FLSC. This is particularly true during the placement and compaction of the initial lift of soil on top of the underlying geosynthetic material. Any observations so noted by the CQA Field Monitors shall be reported immediately to the CQA Site Manager.

3.6 Soils CQA Laboratory

3.6.1 Definition

The Soils CQA Laboratory is the party, independent from Sarasota County and Contractor, responsible for conducting geotechnical laboratory tests in accordance with standards referenced in the Construction Drawings and Technical Specifications and this CQA Plan. The testing results generated by the Soils CQA Laboratory shall be used by the CQA Consultant to verify compliance of the soils construction materials with the plans and specifications and submittals previously approved by the Construction Manager.

It is anticipated that the on-site Soils CQA Laboratory will be utilized to perform the conformance evaluation testing of the various soils components at the FLSC facility. The off-site soils CQA Laboratory will be for more sensitive performance testing required during construction such as hydraulic conductivity testing which require tightly controlled laboratory conditions.

3.6.2 Qualifications

The Soils CQA Laboratory will be experienced in testing of soils similar to those proposed for use in the construction at the FLSC facility in accordance with ASTM and other applicable soil test standards. The Soils CQA Laboratory will be capable of providing test results within a maximum of 7 working days of receipt of samples and will maintain that capability throughout the duration of the earthwork construction.

Prior to construction, the Soils CQA Laboratory, if different from the CQA Consultant, shall submit their qualifications and QA/QC procedures to the Construction Manager for review and approval. The qualifications presented by the Soils CQA Laboratory shall, as a minimum, include:

- corporate background and statement of qualifications;
- list of testing capabilities including reference to ASTM test methods;
- a laboratory QA/QC plan;
- information on staff size and experience; and
- information regarding test result turnaround time.

3.6.3 Responsibilities

The Soils CQA Laboratory will be responsible for testing various soils components at the FLSC facility. These tests shall include, but not be limited to, material qualification (conformance) tests and material construction quality control (performance) tests as described in Construction Drawings and Technical Specifications. The CQA Consultant will be responsible for coordinating the Soils CQA Laboratory testing.

3.7 Geosynthetics CQA Laboratory

3.7.1 Definition

The Geosynthetics CQA Laboratory is the party, independent from Sarasota County, Contractor, and geosynthetics Manufacturer and Installer, responsible for conducting tests on samples of geosynthetic materials used in construction of the FLSC in accordance with standards referenced in the Construction Drawings and Technical Specifications and this CQA Plan. The testing results generated by the Geosynthetics CQA Laboratory shall be used by the CQA Consultant to verify compliance of the geosynthetic materials with plans and specifications and submittals previously approved by the Construction Manager.

3.7.2 Qualifications

The Geosynthetics CQA Laboratory shall hold current accreditation by Geosynthetic Research Institute (GRI) or be approved by the Design Engineer and have experience in testing geosynthetics similar to those proposed for use during construction at the FLSC facility. The Geosynthetics CQA Laboratory shall be familiar with ASTM and other applicable geosynthetic test standards. The Geosynthetics CQA Laboratory will be capable of providing destructive test results for geomembrane field seams within 24 hours of receipt of samples and will maintain that capability throughout the duration of geosynthetic material installation.

Prior to construction, the Geosynthetics CQA Laboratory, if different from the CQA Consultant, shall submit their qualifications to the Construction Manager for review and approval. The qualifications presented by the Geosynthetics CQA Laboratory shall, as a minimum, include:

- corporate background and statement of qualifications;
- listing of testing capabilities including reference to ASTM or other applicable test methods;
- a laboratory QA/QC plan;
- information on staff size and experience; and
- information regarding test result turnaround time.

3.7.3 Responsibilities

The Geosynthetics CQA Laboratory will be responsible for testing various geosynthetic components of the FLSC. These tests shall include, but not be limited to, geosynthetic conformance and performance tests and destructive testing of the geomembrane field seams as described in the Construction Drawings and Technical Specifications. The CQA Consultant will be responsible for coordinating the Geosynthetics CQA Laboratory testing.

3.8 Geosynthetics Manufacturers

The geosynthetics Manufacturers are the firms or corporations responsible for production of the geosynthetic materials to be used in construction at the FLSC facility. The geosynthetics Manufacturers shall be able to provide sufficient production capacity and qualified personnel to meet the demands of the project schedule. Prior to shipment of any material to the site, each geosynthetics Manufacturer shall be pre-qualified and approved by the Construction Manager. The geotextile, geomembrane, geocomposite and GCL Manufacturers shall meet the qualifications outlined in the Technical Specifications, respectively.

Each geosynthetics Manufacturer is responsible for the production and quality control of its respective geosynthetic product. In addition, each geosynthetics Manufacturer is responsible for the condition of the geosynthetic until the material is accepted by the Contractor. Each geosynthetics Manufacturer shall produce a consistent high quality product that shall meet all the requirements of the Technical Specifications. Each geosynthetics Manufacturer shall submit quality control documentation to the Construction Manager for its respective products as required by the Technical Specifications.

3.9 Geosynthetics Installers

The geosynthetics Installers will be experienced and qualified to install the geosynthetic materials of the type specified for this project. The geosynthetics Installers will be approved and/or licensed by the geosynthetics Manufacturers. A copy of the approval letter or license will be submitted by the Contractor to the Construction Manager as required by the Technical Specifications. The geosynthetics Installers shall meet the qualifications outlined in the Technical Specifications. The geosynthetics Installers will designate one representative as its supervisor, who will be responsible for acting as the geosynthetics Installer's spokesman on site. The geosynthetics Installers will provide the Construction Manager with a list of proposed seaming personnel and their qualifications. This document will be reviewed by the CQA Consultant. Final approval of the geosynthetic Installer's geomembrane seaming personnel will be the responsibility of the Construction Manager. Any proposed seaming personnel deemed insufficiently experienced will not be accepted. The most experienced seamer, the "master seamer", shall provide direct supervision, as required, over less experienced seamers. No field seaming shall take place without the master seamer being present.

The geosynthetics Installer's supervisor will be responsible for installation of the geosynthetics used in construction at the FLSC facility and for providing supervision and guidance to the installation crew. The geosynthetics Installer's supervisor is also responsible for the following: (i) obtaining samples, as required by the CQA Plan and the specifications, under the supervision of CQA personnel; (ii) field testing; (iii) documenting quality control testing activities; and (iv) coordinating the geosynthetics installation activities with the Construction Manager. The geosynthetics Installer's supervisor will be responsible for documenting the geosynthetics installation activities, including, but not limited to, on-site personnel, material inventories, production figures, test results, installation deficiencies, and resolution of construction problems.

3.10 Surveyor

The Surveyor is responsible for lines and grades required for control of the work on an ongoing basis during all phases of the FLSC facility construction. Close interaction between the Surveyor, Contractor, and the CQA Consultant is essential to ensure that construction at the FLSC facility is completed in accordance with the Construction Drawings and Technical Specifications. The project Surveyor shall be a state of Florida licensed Professional Land Surveyor or registered Professional Engineer who shall sign and seal all construction survey record drawings. All surveying personnel shall be experienced in the provision of surveying services, including detailed accurate documentation as required in the Technical Specifications. The Surveyor is responsible for all surveying activities and products in accordance with the Technical Specifications.

4. DOCUMENTATION

4.1 Overview

An effective CQA Plan depends largely on recognition of all construction activities that should be monitored and the assignment of responsibilities for the monitoring of each activity. This is most effectively accomplished and verified by the documentation of quality assurance and quality control activities. The CQA Consultant shall be responsible for assuring that the Contractor's quality control requirements have been addressed and satisfied.

The CQA Site Manager shall provide the Construction Manager descriptive daily field reports, data sheets, and logs, as requested, which document that monitoring activities have been accomplished. Examples of some of the forms that will be used to document CQA activities are included in Attachment A. The CQA Site Manager shall also maintain at the job site a complete file of Construction Drawings and Technical Specifications, this CQA Plan, the Contractor's Quality Control Plan(s), checklists, test procedures, daily logs, and other pertinent construction and CQA documents.

4.2 Daily Record Keeping

The CQA Consultant's daily reporting procedures shall include: (i) daily summary report; (ii) monitoring logs; (iii) testing data sheets; and (iv) when appropriate, problem identification and corrective measures reports.

4.2.1 Daily Summary Reports

The CQA Consultant's daily summary reports shall include the following information as applicable:

- an identifying sheet number for cross referencing and document control;
- date, project name, location, and other pertinent project identification;
- data on weather conditions;
- summary on meetings held and their results;

- process description(s) and location(s) of construction activities underway during the time frame of report;
- descriptions and specific locations of areas, or units, of work being tested and/or observed and documented;
- description of locations where tests and samples were taken;
- a narrative summary of field test results;
- off-site materials received, including quality control documentation;
- decisions made regarding acceptance of units of work, and/or corrective actions to be taken in instances of substandard testing results;
- identifying sheet numbers of data sheets and/or problem reporting and corrective measures reports used to substantiate the decisions described above; and
- signature of the respective CQA Site Manager and/or the CQA Field Monitor.

4.2.2 CQA Monitoring Logs and Test Data Sheets

Monitoring observations, sampling information, and test results shall be recorded on the appropriate monitoring logs and test data sheets. The CQA Consultant shall use the monitoring logs and test data sheets to ensure completeness of the required CQA activities. Any corrections to the monitoring logs and test data sheets shall be single line crossed out, initialed by the CQA personnel responsible for the correction and dated. Examples of relevant monitoring logs are presented in Attachment A.

The CQA Consultant's monitoring logs and test data sheets shall include the following information as applicable:

- project specific information such as project name, location;
- the date the CQA activity was performed;
- a unique identifying sheet number for cross-referencing and document control;

- description or title of the CQA activity or test procedure;
- location of the CQA activity or location from which the sample was obtained;
- type of CQA activity or procedure used (reference to standard method when appropriate);
- recorded observation or test data, with all necessary calculations;
- results of the CQA activity and comparison with specification requirements (pass/fail); and
- the initials or signature of personnel involved in CQA inspection activity.

4.2.3 Nonconformance Identification and Reporting

A nonconformance is defined herein as material or workmanship that does not meet the specified requirement(s). Nonconformance identification and corrective measures reports should be cross-referenced to specific summary reports, logs, or test data sheets where the nonconformance was identified. The reports should include the following information as applicable:

- a unique identifying sheet number for cross-referencing and document control;
- detailed description of the problem;
- location of the problem;
- probable cause;
- how and when the problem was located;
- estimation of how long problem has existed;
- suggested corrective measures;
- documentation of corrections (reference to inspection data sheets);
- suggested methods to prevent similar problems; and

- signature of the appropriate CQA Field Monitor and concurrence by the CQA Site Manager.

In some cases, not all of the above information will be available or obtainable. However, when available, such efforts to document nonconformances could help to avoid similar nonconformances in the future. The CQA Site Manager shall distribute copies of the report to the Construction Manager for further actions.

4.3 Photographic Documentation

The CQA Site Manager will be responsible for obtaining photographic documentation of the Contractor's activities, materials installation methods, and testing procedures. Photographs will serve as a pictorial record of work progress, problems, and corrective measures. Photographic reporting data sheets should be utilized to organize and document photographs taken during construction at the FLSC facility. Such data sheets could be cross-referenced or appended to summary reports, CQA monitoring logs, or test data sheets and/or problem identification and corrective measures reports. At a minimum, photographic reporting data sheets should include the following information:

- a unique identifying number on data sheets and photographs for cross-referencing and document control;
- person responsible for photograph;
- the date and location where the photograph was taken; and
- location and description of the work;

These photographs will serve as a pictorial record of work progress, problems, and corrective measures. Copies of the photographs referenced in this section will be part of the Final Certification Report. Color prints shall be organized chronologically and kept in a permanent protective file. Negatives and/or digital files shall be stored in a separate protective file.

4.4 Design and/or Specifications Changes

Design and/or specifications changes may be required during construction. In cases of Contractor initiated changes, the Contractor must submit written requests for such

changes to the Construction Manager. The Design Engineer shall review and respond to these requests in a timely manner. All design and/or specifications changes will be made only with the approval of the Engineer of Record and Design Engineer and approval by FDEP if required. Such changes will take the form of a change order to the contract if required.

4.5 Nonconformances

The Construction Manager will be informed in writing of any significant recurring nonconformance with the Construction Drawings, Technical Specifications, or CQA Plan by the CQA Consultant. The cause of the nonconformance will be determined by the CQA Consultant. The Contractor will be directed by the Construction Manager to make appropriate changes in materials or procedures in order to correct the nonconformance. When this type of evaluation is made, the results will be documented, and any revision to procedures or specifications must be approved by the Design Engineer.

4.6 CQA Certification Report

At the completion of construction phases, the CQA Consultant will provide Sarasota County with a construction phase final certification report for submittal to FDEP. This report will acknowledge: (i) that the work has been performed in compliance with the approved Construction Drawings, Technical Specifications, and approved modifications; (ii) physical sampling and testing has been conducted at the appropriate frequencies; and (iii) that the summary documentation provides the necessary supporting information.

At a minimum, this report will include:

- summary of CQA activities;
- CQA monitoring logs and testing data sheets including sample location plans;
- laboratory test results;
- problem identification and reports of corrective measures reports;
- copies of the photograph documentation referenced in Section 4.3 will be part of the Final Certification Report,

- a descriptive summary of any changes to the Construction Drawings or Technical Specifications; and

- a summary statement indicating compliance with the Construction Drawings or Technical Specifications and any approved changes that are signed and sealed by the CQA Managing Engineer.

The record drawings, which include scale drawings depicting the location of the construction and details pertaining to the extent of construction (e.g., depths, plan dimensions, elevations, soil component thicknesses, etc.), and a geomembrane panel drawing prepared by the CQA Consultant will also be included as part of the final certification report.

4.7 Storage of Records

The CQA Site Manager will be responsible for all CQA document storage during the construction at the FLSC facility. This includes the CQA Consultant's copy of the Construction Drawings and Technical Specifications, the CQA Plan, and the originals of all the data sheets and reports. When the FLSC facility construction is complete and upon issuance of the final certification report, the CQA document originals will be organized and retained by the CQA Consultant until requested by Sarasota County. Required records shall include, but not be limited to, field logbooks, other data collections forms, equipment calibration records, costs data, drawings, maintenance records, and all associated reports.

5. SOILS CONSTRUCTION

5.1 Introduction

CQA monitoring and testing shall be performed during installation of the liner system, the final cover system, and other earthwork components. Criteria to be used for determination of acceptability of the various soil components are identified in the Construction Drawings and Technical Specifications and this CQA Plan.

5.2 Soil Components

General fill is the principal soil component used in the FLSC facility construction. A varying thickness of compacted general fill will be constructed below the FLSC facility liner system. In addition, general fill material is also used for earthwork related to perimeter berm construction. All general fill placement, grading, and compaction will be monitored and tested in accordance with the Construction Drawings, Technical Specifications, and this CQA Plan.

5.3 Record Drawings and As-Built Surveys

During construction of the soil components at the FLSC facility, the CQA Consultant shall routinely review record drawings submitted by the Contractor. The drawings are used to verify location of work, percent of work completed, layer thickness, or final grades. Prior to the placement of successive soil or geosynthetic layers the CQA Consultant shall review as-built surveys that indicate compliance of the preceding layer thickness, lines, and grades. Once an as-built survey has been received, it will be the responsibility of the CQA Consultant to review the information in a timely manner and notify the Contractor of any noncompliance.

5.4 Related Construction Drawings and Technical Specifications

Several sections of the Technical Specifications should be referenced by the CQA Consultant for pertinent soil materials physical properties and construction requirements. Related specifications include the following:

- Section 02100 - Surveying;
- Section 02110 - Clearing, Grubbing & Stripping;

- Section 02200 - Earthwork;
- Section 02215 - Trenching and Backfilling;
- Section 02235 - Drainage Gravel;
- Section 02245 - Riprap;
- Section 02290 - Erosion & Sediment Control; and
- Section 02920 - Vegetative Layer.

Prior to the start of soils construction, the CQA Consultant shall review the information required by the Technical Specifications listed above. Compliance of the submittals with the Technical Specifications shall be determined by the Construction Manager.

5.5 Subgrades

During construction, monitoring of the subgrade preparation shall be performed by the CQA Consultant. The CQA Consultant shall monitor to assure a firm and smooth surface that is free of vegetation and other deleterious materials is achieved. Material placed to achieve grades indicated on the Construction Drawings shall be monitored by the CQA Consultant to verify that the subgrade material and fill placement, grading, and compaction complies with the Technical Specifications. Areas that do not meet the Technical Specifications will be delineated, and nonconforming areas will be reworked by the Contractor. This process will be repeated until acceptable results are achieved.

The CQA Consultant shall monitor the repair and rework of fill material that is damaged by excess moisture (causing softening). If such conditions are found to exist, the CQA Consultant shall evaluate the suitability of the subgrade by the following methods as applicable:

- moisture/density testing; and/or
- continuous visual inspection during proof-rolling.

5.6 Conformance Testing

It will be necessary for the CQA Consultant to observe and test the soil components to ensure they are uniform and conform to the requirements of the Technical Specifications. For

soil materials obtained from on-site sources, visual inspections and conformance tests shall be performed by the CQA Consultant prior to the materials being used. If soil materials are obtained from off site borrow sources, visual inspection and conformance tests shall be performed at the source location or as the materials arrive at the FLSC site. Borrow area inspections may also be utilized by the CQA Consultant to ensure that only suitable soil materials are transported to the FLSC site. For off-site borrow areas containing non-uniform materials, it shall be necessary for the Contractor and the CQA personnel to coordinate excavation and monitoring of the segregation of substandard materials. All materials failing to comply with conformance standards shall be rejected for use at the FLSC facility.

Initial evaluation of various soil types by CQA personnel during construction shall be largely visual; therefore, the CQA personnel must be experienced with visual-manual soil classification procedures. CQA personnel shall be aware that changes in color or texture can be indicative of a change in soil type. CQA personnel shall observe soils for deleterious materials (e.g., roots, stumps, and large objects). When necessary, the visual-manual procedure for the description and identification of soils shall be conducted by the CQA Consultant in accordance with test method ASTM D 2488.

5.6.1 Test Methods

Conformance tests used to evaluate the suitability of soil materials during construction shall be performed in accordance with the current ASTM or other applicable test procedures indicated in Table 5-1. Documentation and reporting of the test results shall be the responsibility of the CQA Consultant.

The standard Proctor test (ASTM D 698) shall be used for the evaluation of moisture/density relationships unless otherwise indicated. Any conflict regarding acceptance of test results shall be resolved by the Design Engineer.

5.6.2 Test Frequency

The frequency of conformance tests shall conform to the minimum frequencies presented in Table 5-1. The frequency of testing may be increased at the discretion of the CQA Consultant or if variability of the materials is observed. The testing frequencies described herein for general fill shall also apply to materials used by the Contractor in areas outside the limits of the liner and final cover systems at the FLSC facility.

5.7 Construction Monitoring

During installation of the various soil components, the CQA Consultant shall visually observe and document the Contractor's earthwork activities for the following:

- changes in the soil consistency;
- the thickness of lifts as loosely placed and as compacted;
- soil conditioning prior to placement including general observations regarding moisture distribution, clod size, etc.;
- placement method which may damage or cause displacement or wrinkling of geosynthetics;
- the action of the compaction and heavy hauling equipment on the construction surface (sheepsfoot penetration, pumping, cracking, etc.);
- the number of passes used to compact each lift;
- desiccation cracks or the presence of ponded water; and
- final lift or layer thickness.

5.8 Performance Testing

During construction, the CQA Consultant shall observe and test all soil components to ensure they are installed in accordance with the requirements of the Construction Drawings and Technical Specifications. The CQA Consultant shall also evaluate the procedures, methods, and equipment used by the Contractor to install the various soil components.

5.8.1 Test Methods

All performance testing shall be conducted in accordance with the Technical Specifications or as directed by the Design Engineer. The field testing methods, used to evaluate the suitability of soils during their installation, shall be performed by the CQA Consultant in accordance with current ASTM test procedures indicated in Table 5-2. Documentation and reporting of the test results shall be the responsibility of the CQA Consultant.

The standard Proctor test (ASTM D 698) shall be used for the evaluation of moisture/density relationships unless otherwise indicated. In-place surface moisture/density by nuclear test methods (ASTM D 3017 and D 2922) shall be used for in-situ field testing. The sand cone test method (ASTM D 1556) or drive cylinder test method (ASTM D 2937) shall be used to establish correlations of moisture and density in cases of uncertainty, and as a check of the nuclear surface moisture/density gauge calibration. Any conflict regarding acceptance of test results shall be resolved by the Design Engineer.

5.8.2 Test Frequency

Performance testing shall be conducted during the course of the work. The minimum construction performance testing frequencies are presented in Table 5-2. The frequency may be increased at the discretion of the CQA Consultant or if variability of the materials is observed by the CQA Consultant. Sampling locations shall be selected by the CQA Consultant. If necessary, the location of routine in-place density tests shall be selected using a non-biased sampling approach.

A special testing frequency shall be used at the discretion of the CQA Consultant when visual observations of construction performance indicate a potential problem. Additional testing for suspected areas shall be considered when:

- rollers slip during rolling operations;
- lift thickness is greater than specified;
- material is at improper and/or variable moisture content;
- it is suspected that less than the specified number of roller passes are made;
- dirt-clogged rollers are used to compact the material;
- rollers may not have used optimum ballast;
- there is change to subgrade condition since subgrade approval;
- fill materials differ substantially from those specified;
- the degree of compaction is doubtful; and
- as directed by the Design Engineer or the Construction Manager.

During construction, the frequency of testing may also be increased in the following situations:

- adverse weather conditions;
- breakdown of equipment;
- at the start and finish of grading;
- material fails to meet specifications; and
- the work area is reduced.

5.9 Deficiencies

If a defect is discovered in the soils construction, the CQA Consultant shall immediately determine the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Consultant shall determine the extent of the deficient area by additional tests, observations, a review of records, or other means that the CQA Consultant deems appropriate. If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the CQA Consultant shall define the limits and nature of the defect and the appropriate remedy.

As soon as possible, after determining the extent and nature of substandard materials, noncompliant construction practice, or other such deficiency in materials or workmanship which cannot be immediately resolved on-the-spot, the CQA Consultant shall notify the Construction Manager and Contractor and schedule appropriate retests when the work deficiency is to be corrected.

The CQA Consultant shall verify that the Contractor has corrected all noted deficiencies. If a specified criterion cannot be met, or unusual weather conditions hinder work, the Contractor shall submit suggested solutions or alternatives to the Construction Manager for review.

At locations where the field testing indicates in-situ conditions which do not comply with the requirements of the Technical Specifications, the failing area shall be reworked to the satisfaction of the CQA Consultant. Alternatively, at the CQA Consultant's option, undisturbed samples of in-place material shall be obtained for appropriate testing. All retests performed by the CQA Consultant must verify that the deficiency has been corrected before any additional work is performed by the Contractor in the area of the deficiency.

5.10 Documentation

The documentation of soils CQA testing activities is an important factor in assuring the successful construction, performance, and approval of the soil components of the FLSC facility. The CQA monitoring observations, sample location descriptions, field test results, and on-site laboratory test results shall be documented by the CQA Consultant on forms specifically designed for their purpose. Reports and forms shall be submitted to the Construction Manager as requested.

TABLE 5-1

MINIMUM CONFORMANCE TESTING FREQUENCIES FOR FLSC SOIL COMPONENTS

TEST NAME/ TEST METHOD	GENERAL FILL
SPECIFICATION SECTION	02200
Particle Size Analysis/ASTM D 422	1 test per 5,000 yd^3
Particle Size Analysis/ASTM C 136	N/A
Soil Classification/ASTM D 2487	1 test per 5,000 yd^3
Standard Proctor/ASTM D 698	1 test per 5,000 yd^3

N/A = Not Applicable

TABLE 5-2

MINIMUM PERFORMANCE TESTING FREQUENCIES FOR FLSC SOIL COMPONENTS

TEST NAME/ TEST METHOD	SOIL TYPE GENERAL FILL/ MISC. SOILS
SPECIFICATION SECTION	02200
In-Situ Moisture/ASTM D 3017	5 tests per acre per lift[1] or 1 test per 200 lf per lift
In-Situ Density/ASTM D 2922	5 tests per acre per lift[1] or 1 test per 200 lf per lift and 1 test per 500 lf of pipeline trench
Sand Cone/ASTM D 1556 or Drive Cylinder/ASTM D 2937	1 test per 25 nuclear tests or 1 test per 200 lf per lift

N/A = Not Applicable

NOTE: 1. A minimum of two nuclear moisture and density tests each day of active soils construction

6. GEOMEMBRANE

6.1 Introduction

The CQA Consultant shall perform conformance and destructive seam testing and shall monitor the installation of geomembranes as required by Section 02770 of the Technical Specifications and this CQA Plan. The testing used to evaluate the conformance of the geomembrane sheet and seams with the requirements of the Technical Specifications shall be carried out by the CQA Consultant in accordance with the current versions of the ASTM or other applicable test procedure indicated in Tables 6-1 and 6-2.

6.2 Manufacturing Plant Visit

At the request of Sarasota County, the CQA Consultant, or authorized representative, shall visit the plant of the geomembrane Manufacturer for the purpose of collecting conformance samples and verifying that manufacturing quality control procedures are in conformance with Section 02770 of the Technical Specifications. If possible, such a visit shall be performed prior to or during the manufacturing of the geomembrane rolls for the FLSC facility project. The CQA Consultant shall review the manufacturing process, quality control procedures, laboratory facilities, and testing procedures.

During the project specific plant visit, the CQA Consultant shall:

- verify that properties guaranteed by the geomembrane Manufacturer meet all specifications;
- verify that the measurements of properties by the geomembrane Manufacturer are properly documented and test methods used are acceptable;
- spot inspect the rolls and verify that they are free of holes, blisters, or any sign of contamination by foreign matter;
- review packaging and transportation procedures to verify that these procedures are not damaging the geomembrane;
- verify that all rolls are properly labeled; and

- verify that extrusion rods and/or beads manufactured for the field seaming of the geomembrane are derived from the same base resin type as the geomembrane.

Upon completion of the manufacturing plant visit, a report describing the findings and observations shall be completed by the CQA Consultant and shall be included as an attachment to the final certification report.

6.3 Transportation, Handling and Storage

The CQA Consultant shall monitor the transportation, handling, and storage of the geomembrane on-site. The Construction Manager shall designate a geomembrane storage location. It will be the responsibility of the Contractor to protect the geomembrane stored on site from theft, vandalism, and damage.

Upon delivery at the site, the Contractor, Installer, and CQA Consultant shall conduct an inspection of the rolls for defects and damage. This inspection shall be conducted without unrolling the materials unless defects or damages are found or suspected. The CQA Consultant shall indicate to the Construction Manager:

- rolls, or portions thereof, which should be rejected and removed from the site because they have severe or nonrepairable flaws which may compromise geomembrane quality; and

- rolls that include minor and repairable flaws that do not compromise geomembrane quality.

The CQA Consultant shall also monitor that equipment used to handle the geomembrane on-site is adequate and does not pose any risk of damage to the geomembrane when used properly.

6.4 Conformance Testing

6.4.1 Sampling Procedures

Upon delivery of the geomembrane rolls to the FLSC facility, the CQA Consultant shall ensure that representative geomembrane conformance samples are obtained at the specified frequency and forwarded to the Geosynthetics CQA Laboratory for testing.

Geomembrane conformance samples shall be taken across the entire width of the roll and shall not include the first 3 ft of material. Unless otherwise directed by the Design Engineer, samples shall be 3 ft long by the roll width. The required minimum geomembrane conformance sampling frequencies are provided in Table 6-1. The CQA Consultant shall mark the machine direction on the samples with an arrow and affix a label, tag, or otherwise mark each sample with the following information:

- date sampled;
- project number;
- lot/batch number and roll number;
- conformance sample number; and
- CQA personnel identification.

6.4.2 Testing Procedures

Conformance testing of the geomembrane materials delivered to the site will be conducted to ensure compliance with both the Technical Specifications and the Manufacturer's list of minimum average roll values. As a minimum, the geomembrane conformance test procedures listed in Table 6-1 shall be performed by the Geosynthetics CQA Laboratory.

6.4.3 Test Results

All conformance test results shall be reviewed, accepted, and reported by the CQA Consultant before deployment of the geomembrane. Any non-conformance of the material's properties with the requirements of the Technical Specifications shall be reported to the Construction Manager. In all cases, the test results shall meet, or exceed, the property values listed in Attachment B.

6.4.4 Conformance Test Failure

In the case of failing test results, the Contractor may request that another sample from the failing roll be retested by the Geosynthetics CQA Laboratory with the

Manufacturer's technical representative present during the test procedure. If the retest fails or if the option to retest is not exercised, then two isolation conformance samples shall be obtained by the CQA Consultant. These isolation samples shall be taken from rolls, which have been determined by correlation with the manufacturer's roll number, to have been manufactured prior to and after the failing roll. This method for choosing isolation rolls for testing should continue until passing tests are achieved. All rolls that fall numerically between the passing roll numbers shall be rejected. The CQA Consultant will verify that the Contractor has replaced all rejected rolls. The CQA Consultant shall document all actions taken in conjunction with geomembrane conformance failures.

6.5 Anchor Trench

The CQA Consultant shall verify and document that the anchor trench has been constructed as indicated in the Construction Drawings. The amount of anchor trench open at any time shall be limited to one day of geomembrane installation capacity. The anchor trench shall be constructed with proper drainage to prevent ponding.

Geosynthetic materials in the anchor trench shall be temporarily anchored with sand bags or other suitable methods approved by the CQA Consultant. The anchor trench shall be backfilled with suitable material as indicated in the Construction Drawings and Technical Specifications as soon as possible after all geosynthetics are installed. In-place moisture/density by nuclear methods testing of the compacted anchor trench backfill shall be performed at a frequency of one per 100 lineal feet of anchor trench.

The anchor trench shall be constructed with a slightly rounded corner where the geosynthetics enter the trench. No loose soil shall be allowed to underlie the geosynthetics in the anchor trench. The CQA Consultant shall verify that all temporary ballast (i.e., sandbags) and deleterious materials are removed from the anchor trench prior to backfilling. Backfilling of the anchor trench shall be performed when the geomembrane is in its most contracted state to prevent stress inducement and using extreme care to prevent any damage to the geosynthetic materials.

6.6 Geomembrane Placement

6.6.1 Field Panel Identification

A field panel is a piece of geomembrane larger than approximately 10 ft^2, 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. The CQA Consultant shall assure that each field panel is given an "identification code" (number or letter-number) consistent with the as-built layout plan. This identification code shall be agreed upon by the Installer and CQA Consultant. This field panel identification code shall be as simple and logical as possible. The geosynthetic Manufacturer's roll numbers shall be traceable to the field panel identification code.

The CQA Consultant shall document the correspondence between roll numbers, factory panels, and field panel identification codes. The field panel identification code shall be used for all quality assurance/quality control records.

6.6.2 Field Panel Placement

The CQA Consultant shall monitor that field panels are installed substantially at the location indicated in the Installer's layout plan, as approved or modified. The CQA Consultant shall record the field panel identification code, Manufacturer's roll number, location, date of installation, time of installation, and dimensions of each field panel.

Geomembrane placement shall not proceed at an ambient temperature below 40°F or above 104°F unless authorized by the Design Engineer. Geomembrane placement shall not proceed 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. The CQA Consultant shall monitor that the above conditions are fulfilled and that the supporting soil has not been damaged by adverse weather conditions.

The CQA Consultant shall monitor geomembrane deployment for the following:

- any equipment used does not damage the geomembrane by handling, trafficking, excessive heat, leakage of hydrocarbons or other means;
- the prepared surface underlying the geomembrane has not deteriorated since previous acceptance, and is still acceptable immediately prior to geomembrane placement;

- any geosynthetic elements immediately underlying the geomembrane are clean and free of foreign objects or debris;

- all personnel working on the geomembrane do not smoke, wear damaging shoes, or engage in other activities which could damage the geomembrane;

- the method used to unroll the panels does not cause scratches or crimps in the geomembrane and does not damage the supporting soil;

- the method used to place the panels minimizes wrinkles (especially differential wrinkles between adjacent panels);

- 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 adjacent sand bags, is recommended along edges of panels to minimize risk of wind flow under the panels); and

- direct contact with the geomembrane is minimized; i.e., the geomembrane is protected by geotextiles, extra geomembrane, or other suitable materials, in areas where excessive traffic may be expected.

The CQA Consultant shall observe the geomembrane panels, after placement and prior to seaming, for damage. The CQA Site Manager shall advise the Construction Manager which panels, or portions of panels, should be rejected, repaired, or accepted. Damaged panels or portions of damaged panels that have been rejected shall be marked and their removal from the work area recorded by the CQA Consultant. Repairs shall be made according to procedures described in this Section.

6.7 Field Panel Seaming

6.7.1 Panel Layout

The CQA Consultant shall review the panel layout drawing previously submitted to the Construction Manager by the Installer and verify that it is consistent with accepted state of practice. 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 ft beyond the toe or shoulder of the slope, or areas of potential stress

concentrations, unless otherwise authorized by the Design Engineer. A seam numbering system compatible with the field panel identification numbering system shall be agreed upon prior to any seaming.

6.7.2 Seaming Equipment and Products

Approved processes for field seaming are extrusion welding and fusion welding. ~~Proposed alternate processes shall be documented and submitted to the Construction Manager for approval.~~ Only equipment which have been specifically recommended by the geosynthetics Manufacturer by make and model shall be used. All seaming equipment shall be permanently marked with an identification number.

6.7.2.1 Fusion Process

The fusion-welding apparatus must be automated, self-propelled devices. The fusion-welding apparatus shall be equipped with gauges giving the applicable temperatures and welding speed. The CQA Consultant shall monitor ambient temperatures, geomembrane surface temperatures, apparatus speed, and apparatus temperatures at appropriate intervals.

The CQA Consultant shall also monitor that:

- the number of spare operable seaming apparatus agreed by the Construction Manager are maintained on site;
- equipment used for seaming will not damage the geomembrane;
- the seaming zone is dry and clean;
- there is sufficient overlap between panels;
- the electric generator is placed on a smooth base such that no damage occurs to the geomembrane;
- for cross seams, the edge of the cross seam is ground to a smooth incline (top and bottom) prior to welding;
- an insulating material is placed beneath the hot welding apparatus after usage; and

- a movable protective layer is used, as necessary, directly below each overlap of geomembrane that is to be seamed to prevent build-up of moisture between the sheets.

6.7.2.2 Extrusion Process

The extrusion-welding apparatus shall be equipped with gauges giving the temperature in the apparatus and at the nozzle. The CQA Consultant shall verify that the extrudate is comprised of the same resin as the geomembrane sheeting. The CQA Consultant shall monitor extrudate temperatures, ambient temperatures, and geomembrane surface temperatures at appropriate intervals.

The CQA Consultant shall also monitor that:

- the number of spare operable seaming apparatus agreed by the Construction Manager are maintained on site;
- equipment used for seaming is not likely to damage the geomembrane;
- the seaming zone is dry and clean;
- the extruder is purged prior to beginning a seam until all heat-degraded extrudate has been removed from the barrel;
- the electric generator is placed on a smooth base such that no damage occurs to the geomembrane; and
- an insulating material is placed beneath the hot welding apparatus after usage.

6.7.3 Seam Preparation

The CQA Consultant shall monitor that:

- prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris of any kind, and foreign material;
- seams are overlapped a minimum of 4 inches;
- if seam overlap grinding is required, the process is completed according to the

geosynthetics Manufacturer's instructions or Section 02770 of the Technical Specifications, whichever is the more stringent, prior to the seaming operation, and in a way that does not damage the geomembrane;

- the grind depth shall not exceed 10 percent of the geomembrane thickness;
- grinding marks shall not appear beyond the extrudate after it is placed; and
- seams are aligned with the fewest possible number of wrinkles and "fishmouths".

6.7.4 Weather Conditions for Seaming

The normally required weather conditions for seaming are as follows:

- Unless authorized by the Design Engineer, no seaming shall be attempted at an ambient temperature below 40°F or above 104°F.
- Between ambient temperatures of 40°F and 50°F, seaming is possible if the geomembrane is preheated by either sun or hot air device, and if there is no cooling of the geomembrane to below 50°F resulting from wind.
- In all cases, the geomembrane seam areas shall be dry and protected from rain and wind.

The CQA Consultant shall verify that methods used by the Installer for seaming at ambient temperatures below 40°F or above 104°F will produce seams that are entirely equivalent to seams produced at ambient temperatures between 40°F and 104°F and protect the overall quality of the geomembrane. The CQA Consultant shall monitor that seaming conducted during abnormal weather conditions is performed in accordance with the methods approved by the Design Engineer.

6.7.5 Overlapping and Temporary Bonding

The CQA Consultant shall monitor that:

- the panels of geomembrane have a finished overlap of a minimum of 4 in. for both extrusion and fusion welding, but in any event sufficient overlap shall be provided to allow peel tests to be performed on the seam;

- no solvent or adhesive is used; and

- 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 spot welding apparatus is controlled such that the geomembrane is not damaged.

6.7.6 Trial Seams

The CQA Consultant shall verify that the Installer performs trial seam tests in accordance with Section 02770 of the Technical Specifications. The CQA Consultant shall observe and document the Installer's trial seam testing procedures. The trial seam samples shall be assigned an identification number and marked accordingly by the CQA Consultant. Each sample shall be marked with the date, time, machine temperature(s) and setting(s), number of seaming unit, and name of seaming technician. Trial seam samples shall be maintained until destructive seam testing of the applicable seams are tested and pass.

6.7.7 General Seaming Procedures

No geomembrane seaming shall be performed unless the CQA Consultant is on-site. The CQA Consultant shall monitor the general seaming procedure used by the installer as follows:

- If required for fusion welding, a movable protective layer of plastic will be placed directly below each overlap of geomembrane that is to be seamed. This is to prevent any moisture build-up between the sheets to be welded.

- 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.

- 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 in. beyond the cut in all directions.

- ~~If seaming operations are carried out at night, adequate illumination shall be provided by the Contractor/Installer to the satisfaction of the CQA Consultant.~~ Geomembrane seaming shall not occur during non-daylight hours.

- Seaming shall extend to the outside edge of panels to be placed in the anchor trench.

6.7.8 Nondestructive Seam Continuity Testing

The CQA Consultant shall monitor that the Installer shall nondestructively test all field seams over their full length using a vacuum test unit or air pressure test (for double fusion seams only). ~~Spark testing will be performed if the seam cannot be tested using the vacuum or air pressure test methods.~~ The purpose of nondestructive tests is to check the continuity of seams. Continuity testing shall be carried out as the seaming work progresses, not at the completion of all field seaming. The CQA Consultant shall:

- monitor nondestructive testing;

- document the results of the nondestructive testing; and

- inform the Contractor and Construction Manager of any noncompliance.

Any required seam repairs shall be made in accordance with the Technical Specifications. The CQA Consultant shall:

- observe the repair procedures;

- observe the retesting procedures; and

- document the results.

The seam number, date of observation, dimensions and/or descriptive location of the seam length tested, name of person performing the test, and outcome of the test shall be recorded by the CQA Consultant.

6.7.9 Destructive Testing

Destructive seam testing shall be performed during the geomembrane installation. The purpose of this testing is to evaluate seam strength. Destructive seam testing shall be done as the seaming work progresses, not at the completion of all field seaming.

6.7.9.1 Location and Frequency

The CQA Consultant shall select all destructive seam test sample locations. Sample locations shall be established as follows.

- A minimum frequency of one test location per 200 ft of seam length. This minimum frequency is to be determined as an average taken throughout the entire facility. This minimum frequency will be decreased for seams made outside the normal ambient temperature range of 40°F to 104°F.

- Test locations shall be determined during seaming at the CQA Consultant's discretion. Selection of such locations may be prompted by suspicion of excess crystallinity, 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.

6.7.9.2 Sampling Procedures

Destructive seam testing shall be performed as the seaming progresses in order to obtain the Geosynthetic CQA Laboratory test results before the geomembrane is covered by overlying materials. The CQA Consultant shall:

- observe sample cutting;

- assign a number to each sample, and mark it accordingly; and

- record sample location on geomembrane panel layout drawing.

All holes in the geomembrane resulting from destructive seam test sampling shall be immediately repaired in accordance with repair procedures described in Section 02770 of the Technical Specifications. The continuity of the new seams in the repaired area shall be nondestructively tested as described in this Section.

6.7.9.3 Size of Samples

At a given sampling location, two types of samples (field test samples and laboratory test samples) shall be taken. First, a minimum of two field samples or test strips should be taken for field testing. Each of these test strips shall be 1 in. wide by 12 in. long, with the seam centered parallel to the width. The distance between these two specimens shall be 42 in. If both specimens pass the field test described in this Section, a second full laboratory destructive sample shall be taken for testing by the Geosynthetics CQA Laboratory.

The full destructive sample shall be located between the two field test strips. The sample shall be 12 in. wide by 42 in. long with the seam centered lengthwise. The sample shall be cut into three parts and distributed as follows:

- one 12 in. by 12 in. portion to the Installer;
- one 12 in. by 12 in. portion to the Construction Manager for archive storage; and
- one 12 in. by 18 in. portion for Geosynthetics CQA Laboratory testing.

6.7.9.4 Field Testing

The test strips shall be tested in the field, for peel adhesion, using a gauged tensiometer. In addition to meeting the strength requirements outlined in Attachment B, all specimens shall exhibit a Film Tear Bond and shall not fail in the weld. If any field test sample fails to meet these requirements, the destructive sample has failed.

The CQA Consultant shall witness all field tests and mark all samples and portions with their number. The CQA Consultant shall also log the date, number of seaming unit, seaming technician identification, destructive sampling, and pass or fail description.

6.7.9.5 Geosynthetics CQA Laboratory Testing

Destructive test samples shall be tested by the Geosynthetics CQA Laboratory. Testing shall include "Bonded Seam Strength" and "Peel Adhesion" (ASTM D 6932). The minimum acceptable values to be obtained in these tests are presented in

Attachment B. At least five specimens shall be tested for each test method. Specimens shall be selected alternately by test from the samples (i.e., peel, shear, peel, shear...). Both the inside and outside tracks of the double track fusion seams shall be tested for peel adhesion. A passing test shall meet or exceed the minimum required values in at least four out of five specimens, and the fifth specimen shall meet or exceed 80% of the minimum required values. In the event that the CQA destructive testing sample fails, the archived sample may be tested following the above procedure described in this section.

The Geosynthetics CQA Laboratory shall provide test results no more than 24 hours after they receive the samples. The CQA Site Manager shall review laboratory test results as soon as they become available, and make appropriate recommendations to the Construction Manager.

6.7.9.6 Procedures for Destructive Test Failure

The following procedures shall apply whenever a sample fails a destructive test, whether that test was conducted in the field or by the Geosynthetics CQA Laboratory. The CQA Consultant will monitor that the Installer follows one of the two options below:

- The Installer can reconstruct the seam (e.g., remove the old seam and reseam) between any two passed destructive test locations or between points judged by the CQA Consultant to represent conditions of the failed seam (e.g., a tie-in seam or a seam made by the apparatus and/or operator used in the failing seam);or

- The Installer can trace the welding path to an intermediate location a minimum of 10 ft from the point of the failed test in each direction and take a small sample for additional field testing in accordance with the destructive test procedure at each location. If these additional isolation samples pass the field test, then full laboratory samples are taken at both locations. If these laboratory samples meet the specified strength criteria, then the seam is reconstructed between these locations. If either sample fails, then the process is repeated to establish the zone in which the seam should be reconstructed or repaired.

All failed seams must be bounded by two locations from which samples passing laboratory destructive tests have been taken or the entire seam is reconstructed and

retested. In cases exceeding 150 ft of reconstructed seam, a sample taken from the zone in which the seam has been reconstructed must pass destructive testing. Repairs shall be made in accordance with this section. The CQA Consultant shall document all actions taken in conjunction with destructive test failures.

6.8 Defects and Repairs

6.8.1 Identification

All seams and non-seam areas of the geomembrane shall be examined by the CQA Consultant 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 Construction Manager shall require the geomembrane surface to be broomed or washed by the Contractor if the amount of dust or mud inhibits examination.

6.9 Repair Procedures

Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, shall be repaired by the geosynthetics Installer in accordance with Section 02770 of the Technical Specifications. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure shall be agreed upon between the Installer and CQA Consultant.

In addition, the following conditions shall be monitored by the CQA Consultant:

- surfaces of the geomembrane which are to be repaired shall be abraded no more than one hour prior to the repair;
- all surfaces must be clean and dry at the time of the repair;
- all seaming equipment used in repairing procedures must be approved;
- the repair procedures, materials, and techniques shall be approved by the CQA Consultant in advance of the specific repair;
- patches or caps shall extend at least 6 in. beyond the edge of the defect, and all

corners of patches shall be rounded with a radius of at least 3 in.; and

- the geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

6.9.1 Verification of Repairs

Each repair shall be numbered and logged. Each repair shall be non-destructively tested using approved methods. Repairs which pass the non-destructive test shall be taken as an indication of an adequate repair. Large caps may be of sufficient extent to require destructive test sampling, at the discretion of the CQA Consultant or as specified in Table 6-2. The CQA Consultant shall observe all non-destructive testing of repairs and shall record the number of each repair, date, and test outcome.

6.10 Liner System Acceptance

The Contractor shall retain all responsibility for the geosynthetics until acceptance by the Construction Manager. The terms for the liner system acceptance are described in Section 02770 of the Technical Specifications.

6.11 Materials in Contact with the Geomembrane

The procedures outlined in this section are intended to assure that the installation of materials in contact with the geomembrane do not cause damage. Additional quality assurance and quality control procedures are necessary to assure that systems built with these materials will be constructed in such a way to ensure proper performance.

6.11.1 Soils

The CQA Consultant shall monitor that the Contractor takes all necessary precautions to ensure that the geomembrane is not damaged during its installation, during the installation of other components of the liner system, or by other construction activities. The CQA Consultant shall monitor the following:

- placement of protective soil materials above the geomembrane which shall not proceed at an ambient temperature below 40°F or above 104°F unless otherwise

approved by the Construction Manager;

- soil placement operations above the geomembrane shall be performed by the Contractor to minimize wrinkles in the geomembrane;
- equipment used for placing soil shall not be driven directly on the geomembrane;
- a minimum soil thickness of 1 ft is maintained between a light, track-mounted dozer (e.g., having a maximum ground pressure of 5 psi) and the geomembrane;
- a minimum soil thickness of 3 ft is maintained between rubber-tired vehicles and the geomembrane; and
- soil thickness shall be greater than 3 ft in heavily trafficked areas such as access ramps.

6.11.2 Appurtenances

The CQA Consultant shall monitor that:

- installation of the geomembrane in appurtenant areas, and connection of geomembrane to appurtenances have been made in accordance with the Construction Drawings and Technical Specifications;
- extreme care is taken by the Installer when seaming around appurtenances since neither non-destructive nor destructive testing may be feasible in these areas; and
- the geomembrane has not been visibly damaged when making connections to appurtenances.

TABLE 6-1

GEOMEMBRANE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM TESTING FREQUENCY[1]
Specific Gravity	ASTM D 792 Method A or ASTM D 1505	1 test per 100,000 ft^2
Thickness	ASTM D 5199 or ASTM D 5994 GRI GM13	1 test per 100,000 ft^2
Tensile Strength at Yield	ASTM D 638	1 test per 100,000 ft^2
Tensile Strength at Break	ASTM D 638	1 test per 100,000 ft^2
Elongation at Yield	ASTM D 638	1 test per 100,000 ft^2
Elongation at Break	ASTM D 638	1 test per 100,000 ft^2
Carbon Black Content	ASTM D 1603	1 test per 100,000 ft^2
Carbon Dispersion	ASTM D 5596	1 test per 100,000 ft^2
Oxidative Induction Time	ASTM D 3895 or ASTM D 5885	1 test per 100,000 ft^2
Interface Friction	ASTM D 5321	1 test

TABLE 6-2

GEOMEMBRANE SEAM TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM TESTING FREQUENCY
Peel Adhesion of Seam	ASTM D 6392[1,3]	1 test every 200 ft
Bonded Seam Strength	ASTM D 6392[2,3]	1 test every 200 ft
Vacuum Testing Welded Seams	—	100 percent of extrusion welds
Air Pressure Testing Welded Seams	—	100 percent of fusion welds

Notes:

1. For peel adhesion, seam separation shall not extend more than 10 percent into the seam interface. Testing shall be discontinued when the sample has visually yielded.
2. For shear tests, the sheet shall yield before failure of the seam.
3. For either test, sample failure shall be a Film Tear Bond (FTB) as outlined in NSF 54, Attachment A.

7. GEOSYNTHETIC CLAY LINER

7.1 Introduction

The CQA Consultant shall perform conformance testing and shall monitor the installation of the geosynthetic clay liner (GCL) as required by Section 02780 of the Technical Specifications and this CQA Plan. The testing used to evaluate the conformance of the GCL with the requirements of the Technical Specifications shall be performed by the CQA Consultant in accordance with the current versions of the ASTM or other applicable test procedure indicated in Table 7-1.

7.2 Transportation, Handling, and Storage

The CQA Consultant shall monitor the transportation, handling, and storage of the GCL on-site. The Construction Manager shall designate a GCL storage location. Handling of the rolls shall be performed in a competent manner such that damage does not occur to the GCL or its protective wrapping. Any protective wrapping that is damaged or stripped off the rolls shall be repaired immediately to the satisfaction of the CQA Consultant. During transportation, handling, and storage the GCL rolls will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions.

Upon delivery of the GCL at the site, the Contractor, Installer, and CQA Consultant shall conduct an inspection of the rolls for defects and damage. This inspection shall be conducted without unrolling the materials unless defects or damages are found or suspected. The CQA Consultant shall indicate to the Construction Manager:

- rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and
- rolls which include minor repairable flaws.

The CQA Consultant shall also monitor that equipment used to handle the GCL on-site is adequate and does not pose any risk of damage to the GCL when used properly.

7.3 Conformance Testing

7.3.1 Sampling Procedures

Upon delivery of the rolls of GCL, the CQA Consultant will assure that samples are removed and forwarded to the Geosynthetic CQA Laboratory for testing of conformance to both the Technical Specifications and the list of guaranteed properties provided by the Manufacturer. Conformance samples will be 3 ft long by the roll width. The CQA Consultant will mark the machine direction on the samples with a waterproof marker, and tape or otherwise secure the cut edges of the sample to eliminate the loss of the granular bentonite. The required minimum sampling frequencies are provided in Table 7-1. The rolls shall be immediately re-wrapped and replaced in their shipping trailers or in the temporary field storage area. The CQA Consultant shall mark the machine direction on the samples with an arrow and affix a label, tag, or otherwise mark each sample with the following information:

- date sampled;
- project number;
- lot/batch number and roll number;
- conformance sample number; and
- CQA personnel identification.

7.3.2 Testing Procedure

Conformance testing of the GCL materials delivered to the site will be conducted to ensure compliance with both the Technical Specifications and the Manufacturer's list of minimum average roll values. As a minimum, the GCL conformance test procedures listed in Table 7-1 shall be performed by the Geosynthetics CQA Laboratory.

7.3.3 Test Results

The CQA Consultant will examine all results from laboratory conformance testing and will report any non-conformance to the Construction Manager. The GCL

conformance test results shall meet or exceed the minimum property values presented in Attachment C.

7.3.4 Conformance Test Failure

In the case of failing test results, the Contractor may request that another sample from the failing roll be retested by the Geosynthetics CQA laboratory with the Manufacturer's technical representative present during the test procedure. If the retest fails or if the option to retest is not exercised, then two isolation conformance samples shall be obtained by the CQA Consultant. These isolation samples shall be taken from rolls, which have been determined by correlation with the manufacturer's roll number, to have been manufactured prior to and after the failing roll. This method for choosing isolation rolls for testing should continue until passing tests are achieved. All rolls that fall numerically between the passing roll numbers shall be rejected. The CQA Consultant will verify that the Contractor has replaced all rejected rolls. The CQA Consultant shall document all actions taken in conjunction with GCL conformance failures.

7.4 Surface Preparation

The GCL shall not be placed on surfaces which are softened due to high water content or cracked due to desiccation. The CQA Consultant and the Installer will jointly verify that the surface on which the GCL will be installed is acceptable. The Contractor shall comply with the surface preparation and acceptance requirements identified in Section 02200 of the Technical Specifications. Additionally, the surface shall contain no loose stones and no ruts greater than 1-in. depth. The CQA Consultant shall notify the Contractor of any observed change in the supporting soil condition that may require repair work and verify that compacted soil repair work is completed in accordance with the requirements of the Technical Specifications of this CQA Plan.

7.5 Placement

The CQA Consultant shall verify that the Installer has taken all necessary precautions to protect the underlying subgrade during GCL deployment operations. The CQA Consultant shall verify that all GCL is handled in such a manner as to ensure they are not damaged in any way, and the following conditions are met:

- in the present of wind, all GCL are weighted with sandbags or the equivalent;

- GCL is kept continually under tension to minimize the presence of wrinkles;
- GCL is cut using a utility blade in a manner recommended by the Manufacturer;
- during placement, care is taken not to entrap fugitive stones or other debris under the GCL;
- the exposed GCL is protected from damage in heavily trafficked areas;
- a visual examination of the GCL is carried out over the entire surface, after installation, to assure that damaged areas, if any, are identified and repaired; and
- if a white colored GCL is used, precautions are taken against "snowblindness" of personnel.

7.6 Overlaps

The CQA Consultant shall monitor and verify the GCL overlapping procedures conform to the requirements of Section 02780 of the Technical Specifications. GCL panels shall be overlapped at a minimum of 6 inches along panel sides and a minimum of 12 inches along panel ends. Dry bentonite powder shall be applied, at a minimum rate of one pound per lineal foot, around pipe penetrations or other perforations of GCL which may be required.

7.7 Repair

The CQA Consultant shall monitor the repair of any holes or tears in the GCL or the geotextile backing. Repairs shall be made by placing a patch made from the same type GCL over the damaged area. On slopes greater than 5 percent, the patch shall overlap the edges of the hole or tear by a minimum of 2 ft in all directions. On slopes, 5 percent or flatter, the patch shall overlap the edges of the hole or tear by a minimum of 1 ft in all directions. The patch shall be secured to the satisfaction of the CQA Consultant to avoid shifting during soil placement or covering with another geosynthetic.

TABLE 7-1

GCL CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM TESTING FREQUENCY
Hydraulic Conductivity	ASTM D 5887	1 test per 100,000 ft^2

8. GEOTEXTILES

8.1 Introduction

The CQA Consultant shall perform conformance testing and shall monitor the installation of geotextile filters, and separators as required by Section 02720 of the Technical Specifications and this CQA Plan. The testing used to evaluate the conformance of the geotextiles with the requirements of the Technical Specifications shall be performed by the CQA Consultant in accordance with the current versions of the ASTM or other applicable test procedure indicated in Table 8-1.

8.2 Transportation, Handling, and Storage

The CQA Consultant shall monitor the transportation, handling, and storage of the geotextile on-site. The Construction Manager shall designate a geotextile storage location. During transportation, handling, 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.

Handling of the geotextile rolls shall be performed in a competent manner such that damage does not occur to the geotextile or to its protective wrapping. Rolls of geotextiles shall not be stacked upon one another to the extent that deformation of the core occurs or to the point where accessibility can cause damage in handling. Furthermore, geotextile rolls shall be stacked in such a way that access for conformance sampling is possible. Protective wrappings shall be removed less than one hour prior to unrolling the geotextile. After unrolling, a geotextile shall not be exposed to ultraviolet light for more than 30 calendar days.

Outdoor storage of geotextile rolls shall not exceed the Manufacturers recommendations or longer than 6 months whichever is less. For storage periods longer than 6 months a temporary enclosure shall be placed over the rolls, or they shall be moved to an enclosed facility. The location of temporary field storage shall not be in areas where water can accumulate. The rolls shall be elevated off the ground to prevent contact with ponded water.

Upon delivery at the site, the Contractor, Installer, and CQA Consultant shall conduct an inspection of the rolls for defects and damage. This inspection shall be conducted without unrolling the materials unless defects or damages are found or suspected. The CQA Consultant shall indicate to the Construction Manager:

- rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and
- rolls which include minor repairable flaws.

The CQA Consultant shall also monitor that equipment used to handle the geotextiles on-site is adequate and does not pose any risk of damage to the geotextiles when used properly.

8.3 Conformance Testing

8.3.1 Sampling Procedures

Samples shall be taken across the entire width of the roll and shall not include the first 3 feet. Unless otherwise specified, samples shall be 3 feet long by the roll width. The required minimum geotextile conformance sampling frequencies are provided in Table 8-1. The CQA Consultant shall mark the machine direction on the samples with an arrow and affix a label, tag, or otherwise mark each sample with the following information:

- date sampled;
- project number;
- lot/batch number and roll number;
- conformance sample number; and
- CQA personnel identification.

The geotextile rolls which are sampled shall be immediately rewrapped in their protective coverings to the satisfaction of the CQA Consultant.

8.3.2 Testing Procedure

Conformance testing of the geotextile materials delivered to the site will be conducted to ensure compliance with both the Technical Specifications and the Manufacturer's list of minimum average roll values. As a minimum, the geotextile conformance test procedures listed in Table 8-1 shall be performed by the Geosynthetics CQA Laboratory.

8.3.3 Test Results

The CQA Consultant shall review all laboratory conformance test results and verify compliance of the test results with the specification shown in Attachment D prior to deployment of the geotextiles. Any non-conformance shall be reported to the Construction Manager.

8.3.4 Conformance Test Failure

In the case of failing test results, the Contractor may request that another sample from the failing roll be retested by the Geosynthetics CQA Laboratory with the Manufacturer's technical representative present during the test procedure. If the retest fails or if the option to retest is not exercised, then two isolation conformance samples shall be obtained by the CQA Consultant. These isolation samples shall be taken from rolls, which have been determined by correlation with the Manufacturer's roll number, to have been manufactured prior to and after the failing roll. This method for choosing isolation rolls for testing should continue until passing tests are achieved. All rolls that fall numerically between the passing roll numbers shall be rejected. The CQA Consultant will verify that the Contractor has replaced all rejected rolls. The CQA Consultant shall document all actions taken in conjunction with geotextile conformance failures.

8.3.5 Placement

The CQA Consultant shall monitor the placement of all geotextiles to assure they are not damaged in any way, and the following conditions are met.

- ~~On slopes, the geotextiles shall be securely anchored in the anchor trench and then deployed down the slope in such a manner as to continually keep the geotextile in tension.~~

- In the presence of wind, all geotextiles shall be weighted with sandbags or the equivalent. Such sandbags shall be installed during placement and shall remain until replaced with earth cover material.

- Trimming of the geotextiles shall be performed using only a upward cutting hook blade. Special care must be taken to protect other materials from damage which could be caused by the cutting of the geotextiles.

- The CQA Consultant shall monitor that the Installer is taking necessary precautions to prevent damage to underlying layers during placement of the geotextile.

- During placement of geotextiles, care shall be taken not to entrap stones, excessive dust, or moisture that could generate clogging of drains or filters.

- A visual examination of the geotextile shall be carried out over the entire surface, after installation, to ensure that no potentially harmful foreign objects, (e.g., stones, sharp objects, small tools, sandbags, etc.) are present.

8.4 Seams and Overlaps

All geotextile filters shall be continuously sewn (i.e., spot sewing is not allowed). Geotextiles shall be overlapped 6 in. prior to seaming. No horizontal seams shall be allowed on side slopes that are steeper than 10 horizontal to 1 vertical (i.e. seams shall be along, not across, the slope), except as part of a patch.

Sewing shall be done using polymeric thread with chemical and ultraviolet resistance properties equal to or exceeding those of the geotextile. The seams shall be sewn using a single row type "401" two-thread chainstitch. The CQA Consultant shall monitor the geotextile seaming procedures to verify that seams and overlaps are in accordance with Section 02720 of the Technical Specifications.

Geotextile separators may be overlapped a minimum of 2 feet in lieu of sewing.

8.5 Repair

The CQA Consultant shall monitor that any holes or tears in the geotextile are repaired as follows:

- On-slopes: A patch made from the same geotextile is double seamed into place (with each seam 1/4 in. to 3/4 in. apart and no closer than 1 in. from any edge) with a minimum 12-in. overlap. Should any tear exceed 50 percent of the width of the roll, that roll shall be removed from the slope and replaced.

- Non-slopes: A patch made from the same geotextile is sewn in place with a minimum of 12 in. overlap in all directions away from the repair area.

Care shall be taken to remove any soil or other material which may have penetrated the torn geotextile. The CQA Consultant shall observe all repairs and assure that any non-compliance with the above requirements is corrected.

8.6 Placement of Soil Materials

The CQA Consultant shall monitor the Contractor's placement of all materials located on top of a geotextile, to verify:

- that no damage occurs to the geotextile;

- that no shifting of the geotextile from its intended position occurs and underlying materials are not exposed or damaged;

- that excess tensile stress does not occur in the geotextile; and

- that equipment ground pressure on geotextiles ~~overlying geomembranes~~ does not exceed those specified in Section 02720 of the Technical Specifications.

Soil backfilling or covering of the geotextile with another geosynthetic shall be completed within 30 days. ~~On side slopes, soil layers shall be placed over the geotextile from the bottom of the slope upward.~~

TABLE 8-1

GEOTEXTILE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM TESTING FREQUENCY
Mass per Unit Area	ASTM D 5261	1 test per 100,000 ft^2
Grab Strength	ASTM D 4632(1)	1 test per 100,000 ft^2
Trapezoidal Tear Strength	ASTM D 4533(2)	1 test per 100,000 ft^2
Puncture Resistance	ASTM D 4833(3)	1 test per 100,000 ft^2
Burst Strength	ASTM D 3786	1 test per 100,000 ft^2
Apparent Opening Size(~~5~~ 4)	ASTM D 4751	1 test per 100,000 ft^2
Permittivity(~~5~~ 4)	ASTM D 4491	1 test per 100,000 ft^2

Notes:

1. Minimum of values measured in machine and cross machine directions with 1 inch clamp on Constant Rate of Extension (CRE) machine.
2. Minimum value measured in machine and cross machine direction.
3. Tension testing machine with a 1.75-inch diameter ring clamp, the steel ball being replaced with 0.31-inch diameter solid steel cylinder with a flat tip centered within the ring clamp.
4. Apparent opening size and permittivity testing to be performed on filter geotextiles only.

9. GEOCOMPOSITES

9.1 Introduction

The CQA Consultant shall perform conformance testing and shall monitor the installation of the geocomposite drainage layers as required by Section 02740 of the Technical Specifications and this CQA Plan. The testing used to evaluate the conformance of the geocomposite drainage layers with the requirements of the Technical Specifications shall be performed by the CQA Consultant in accordance with the current versions of the ASTM or other applicable test procedure indicated in Table 9-1.

9.2 Transportation, Handling and Storage

The CQA Consultant shall monitor the transportation, handling, and storage of the geocomposite on-site. The Construction Manager shall designate a geocomposite storage location. During transportation, handling, and storage, the geocomposite shall be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions.

Handling of the geocomposite rolls shall be performed in a competent manner such that damage does not occur to the geocomposite or to its protective wrapping. Rolls of geocomposite shall not be stacked upon one another to the extent that deformation of the roll occurs or to the point where accessibility can cause damage in handling. Furthermore, geocomposite rolls shall be stacked in such a way that access for conformance sampling is possible. Protective wrappings shall be removed less than one hour prior to unrolling the geocomposite. After unrolling, a geocomposite shall not be exposed to ultraviolet light for more than 30 calendar days.

Outdoor storage of geocomposite rolls shall not exceed the Manufacturer's recommendations or longer than 6 months whichever is less. For storage periods longer than 6 months a temporary enclosure shall be placed over the rolls, or they shall be moved to an enclosed facility. The location of temporary field storage shall not be in areas where water can accumulate. The rolls shall be elevated off the ground to prevent contact with ponded water.

Upon delivery at the site, the Contractor, Installer, and CQA Consultant shall conduct an inspection of the rolls for defects and damage. This inspection shall be

conducted without unrolling the materials unless defects or damages are found or suspected. The CQA Consultant shall indicate to the Construction Manager:

- rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and
- rolls which include minor repairable flaws.

The CQA Consultant shall also monitor that equipment used to handle the geocomposites on-site is adequate and does not pose any risk of damage to the geocomposites when used properly.

9.3 Conformance Testing

9.3.1 Sampling Procedures

Samples shall be taken across the entire width of the roll and shall not include the first 3 feet. Unless otherwise specified, samples shall consist of one section 3 feet long by the roll width for geonet and geocomposite testing and one section 10 feet long cut 1 foot from the edge of the geonet for testing of the unbonded geotextiles. The required minimum geocomposite conformance sampling frequencies are provided in Table 9-1. The CQA Consultant shall mark the machine direction on the samples with an arrow and affix a label, tag, or otherwise mark each sample with the following information:

- date sampled;
- project number;
- lot/batch number and roll number;
- conformance sample number; and
- CQA personnel identification.

The geocomposite rolls which are sampled shall be immediately rewrapped in their protective coverings to the satisfaction of the CQA Consultant.

9.3.2 Testing Procedure

Conformance testing of the geocomposite materials delivered to the site will be conducted to ensure compliance with both the Technical Specifications and the manufacturer's list of minimum average roll values. As a minimum, the geotextile, geonet, and geocomposite conformance test procedures listed in Table 9-1 shall be performed by the Geosynthetics CQA Laboratory.

9.3.3 Test Results

The CQA Consultant shall review all laboratory conformance test results and verify compliance of the test results with the specification shown in Attachment E prior to deployment of the geocomposites. Any non-conformance shall be reported to the Construction Manager.

9.3.4 Conformance Test Failure

In the case of failing test results, the Contractor may request that another sample from the failing roll be retested by the Geosynthetics CQA laboratory with the manufacturer's technical representative present during the test procedure. If the retest fails or if the option to retest is not exercised, then two isolation conformance samples shall be obtained by the CQA Consultant. These isolation samples shall be taken from rolls, which have been determined by correlation with the manufacturer's roll number, to have been manufactured prior to and after the failing roll. This method for choosing isolation rolls for testing should continue until passing tests are achieved. All rolls which fail numerically between the passing roll numbers shall be rejected. The CQA Consultant will verify that the Contractor has replaced all rejected rolls. The CQA Consultant shall document all actions taken in conjunction with geocomposite conformance failures.

9.4 Placement

The CQA Consultant shall monitor the placement of all geocomposites to assure they are not damaged in any way, and the following conditions are met.

- ~~On slopes, the geocomposites shall be securely anchored in the anchor trench and then deployed down the slope in such a manner as to continually keep the geocomposites in tension.~~

- In the presence of wind, all geocomposites shall be weighted with sandbags or the equivalent. Such sandbags shall be installed during placement and shall remain until replaced with earth cover material.

- Trimming of the geocomposites shall be performed using only a upward cutting hook blade. Special care must be taken to protect other materials from damage which could be caused by the cutting of the geocomposites.

- The CQA Consultant shall monitor that the Installer is taking necessary precautions to prevent damage to underlying layers during placement of the geocomposite.

- During placement of geocomposites, care shall be taken not to entrap stones, soil, excessive dust, or moisture that could damage the geomembrane, generate clogging of drains or filters, or hamper subsequent drainage operations.

- A visual examination of the geocomposite shall be carried out over the entire surface, after installation, to ensure that no potentially harmful foreign objects, (e.g., stones, sharp objects, small tools, sandbags, etc.) are present.

9.5 Joining, Seams, and Overlaps

The components of the geocomposite (e.g., geotextile, geotextile) shall be seamed, joined, and overlapped to like components in adjacent geocomposites. Lower geotextile components of the geocomposites shall be overlapped such that the component has a minimum overlap of four inches. Adjacent edges of geonet component along the length of the geocomposite should be overlapped a minimum ~~2-3~~ 4 inches and joined by tying the geonet together with white or yellow plastic fasteners or polymeric thread. Geonet for adjoining geocomposite panels (end to end) along the roll width should be shingled down in direction of slope and overlapped a minimum of 12 inches. Upper geotextile components of the geocomposites shall be continuously sewn (i.e., spot sewing is not allowed). Geotextiles shall be overlapped 6 in. prior to sewing. No horizontal seams shall be allowed higher than one-third the slope height on side slopes that are steeper

than 10 horizontal to 1 vertical (i.e. seams shall be along, not across, the slope), except as part of a patch.

Sewing of geotextiles shall be done using polymeric thread with chemical and ultraviolet resistance properties equal to or exceeding those of the geotextile. The seams shall be sewn using a single row type "401" two-thread chainstitch. The CQA Consultant shall monitor the geotextile seaming and geonet tying procedures to verify that joining, seams, and overlaps are in accordance with Section 02740 of the Technical Specifications.

9.6 Repair

The CQA Consultant shall monitor that any holes or tears in the geocomposite are repaired as follows:

- A patch made from the same geocomposite will be secured into place by tying fasteners through the bottom geotextile and the geonet of the patch, and through the top geotextile and geonet.
- The patch will extend 2 feet beyond the edges of the hole or tear.
- The patch will be secured every 6 inches and heat sealed to the top geotextile of the geocomposite needing repair.
- If the hole or tear is more than 50 percent of the width of the roll, the damaged area should be cut out and the two portions of the geocomposite will be joined.

Care will be taken to remove any soil or other material which may have penetrated the torn geocomposite component. The CQA Consultant shall observe any repair and assure that any non-compliance with the above requirements is corrected.

9.7 Placement of Soil Materials

The CQA Consultant shall monitor the Contractor's placement of all soil materials located on top of a geocomposite, to verify:

- that no damage occurs to the geocomposite;

- that no shifting of the geocomposite from its intended position occurs and underlying materials are not exposed or damaged;
- that excess tensile stress does not occur in the geocomposite; and
- that equipment ground pressure on geocomposites overlying geomembranes does not exceed those specified in Section 02740 of the Technical Specifications.

Soil backfilling or covering of the geocomposite shall be completed within 30 days. On side slopes soil layers shall be placed over the geocomposite from the bottom of the slope upward.

TABLE 9-1

GEOCOMPOSITE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM TESTING FREQUENCY[3]
Geotextile Components		
Mass per Unit Area	ASTM D 5261	1 test per 100,000 ft^2
Grab Strength	ASTM D 4632[1]	1 test per 100,000 ft^2
Trapezoidal Tear Strength	ASTM D 4533[2]	1 test per 100,000 ft^2
Apparent Opening Size	ASTM D 4751	1 test per 100,000 ft^2
Permittivity	ASTM D 4491	1 test per 100,000 ft^2
Geocomposite		
Transmissivity[3]	ASTM D 4716	1 test per 100,000 ft^2
Peel Strength	ASTM F 904	1 test per 100,000 ft^2

Notes:

1. Minimum of values measured in machine and cross machine directions with 1 inch clamp on Constant Rate of Extension (CRE) machine.
2. Minimum value measured in machine and cross machine direction.
3. The design transmissivity is the hydraulic transmissivity of the geocomposite measured using water at 68°F ± 3°F with a hydraulic gradient and compressive stress for geocomposites as described in the Technical Specifications. For the tests, the geocomposites shall be overlain by soil representative of the material that will be used on the project. The geocomposite shall be underlain by a textured geomembrane. The minimum test duration shall be 24 hours and the report for the test results shall include measurements at intervals over the entire test duration.

10. PIPES AND FITTINGS

10.1 Introduction

The CQA Consultant shall monitor the installation of ancillary materials such as pipes and fittings for the leachate collection and conveyance system ~~and FLSC gas management system~~ as required by Sections 02715 of the Technical Specifications, the Construction Drawings and this CQA Plan.

10.2 Butt-Fusion Welding Process

The CQA Consultant shall monitor the assembling of lengths of HDPE pipe into suitable installation lengths by the butt-fusion process. Butt-fusion means the butt-joining of the pipe by softening the aligned faces of the pipe ends in a suitable apparatus and pressing them together under controlled pressure. Butt-fusion welding of the HDPE pipes and fittings shall be performed by the Contractor in accordance with the pipe manufacturer's recommendations as to equipment and technique.

10.3 Transportation, Handling and Storage

The pipe is to be bundled together with plastic straps for bulk handling and shipment. The packing shall be such that either fork lifts or cranes equipped with slings can be used for safe handling. The pipe shall be segregated by wall thickness and diameter.

The CQA Consultant shall monitor the offloading of the pipe to assure that handling is done in a competent manner and that the pipes are not placed in areas where water can accumulate. The pipe shall not be stacked more then three high or in such a manner that could cause damage to the pipe. Furthermore, the pipe shall be stacked in such a manner that access for any conformance sampling is possible. Outdoor storage should be no longer than 12 months. For outdoor storage periods longer than 12 months a temporary covering shall be placed over the pipes, or they shall be moved to within an enclosed facility.

10.4 Installation

The CQA Consultant shall monitor that care is taken during installation of the pipes such that they will not be cut, kinked, or otherwise damaged. Ropes, fabric, or rubber-protected slings and straps shall be used by the Contractor when installing pipes. The use of chains, cables, or hooks inserted into the pipe ends shall not be allowed.

The Contractor shall install the pipe and fittings in such a manner that the materials are not damaged. Slings for handling the pipe shall not be positioned at butt-fused joints of HDPE pipes. Sections of the pipes with deep cuts and/or gouges shall be removed and the ends of the pipeline rejoined. Care shall be exercised when lowering pipe into the trench to prevent damage or twisting of the pipe.

10.5 Testing

The CQA Consultant shall monitor the testing of all pipes as required by the Technical Specifications and as necessary to assure workmanship conforming the state-of-practice.

11. MECHANICAL AND ELECTRICAL

11.1 Introduction

The CQA Consultant shall monitor the materials used in and installation of all mechanical and electrical systems to assure compliance with the Technical Specifications and approved submittals. The mechanical and electrical systems include, but are not limited to, the following:

- leachate sump pumps and associated connections and wiring;
- overhead/buried power distribution system, power wiring, including power circuit connections for pump motors, and equipment mounting boards; and
- temporary support facilities for electric, water, and sanitary sewer services.

11.2 Related Construction Drawings and Technical Specifications

The mechanical work performed by the Contractor shall comply with the Construction Drawings, Technical Specifications, and approved submittals. These specifications shall be referenced for specific details of the mechanical equipment requirements and installation. The electrical work performed by the Contractor shall comply with Construction Drawings, Technical Specifications, and approved submittals. These specifications shall be referenced for specific details of the electrical requirements and installation.

11.3 Codes, Rules, Inspections, and Workmanship

The CQA Consultant shall monitor the work of the Contractor in the installation of all mechanical and electrical appurtenances in accordance with national codes and other regulations or authorities having jurisdiction over the work. The CQA Consultant shall observe and document construction acceptance testing procedures performed by the Contractor.

11.4 Record Drawings

The CQA Consultant shall monitor the maintenance by the Contractor of a set of prints on which the actual installation of all mechanical and electrical work shall be accurately shown, indicating any variation from Construction Drawings or approved submittals. Changes in layout or circuitry shall be clearly and completely indicated as the work progresses. These progress prints shall be inspected by the Design Engineer and Construction Manager and used to determine the progress of mechanical and electrical work.

At the completion each phase of the work, the CQA consultant shall obtain from the Contractor a set of record drawings of the work to include marked-up prints showing the dimensioned location of all underground systems.

12. CONCRETE

12.1 Introduction

This CQA Consultant shall monitor the construction and perform conformance testing of all concrete materials and finished products to assure compliance with Construction Drawings and Technical Specifications.

12.2 Inspections

The CQA Consultant shall monitor concrete workmanship to assure that the Contractor does not place concrete until foundations, forms, reinforcing steel, pipes, conduits, sleeves, anchors, hangers, inserts, and other work required to be built into concrete has been inspected and approved by the Construction Manager. The Contractor is required to notify the Construction Manager and CQA Consultant at least 24 hours in advance of concrete placement activities for scheduling of the inspection activities described above.

12.3 Field Quality Control Testing

Conformance testing of placed concrete shall be the responsibility of the CQA Consultant. The concrete test program shall meet the following requirements:

- Concrete samples will be obtained by the CQA Consultant at a frequency of one set of standard cylindrical test specimens for the first 5 cubic yards and every 25 cubic yards of concrete or any portion of thereafter for each structure. For each work shift, when concrete is delivered, at least one set of specimens will be made. A set of test specimens will consist of at least three standard cylinders. Each set of test specimens will be tested for 2-day, 7-day, and 28-day compressive strength, and a fourth cylinder will be held in reserve.

- Compressive strengths shall be determined from the standard test specimens taken according to ASTM C 31 and ASTM C 172, and cured and tested in accordance with ASTM C 39. Core drilling, if required, and testing will be in accordance with ASTM C 94.

- If required by the Engineer, slump and air content shall be determined with no less frequency than that of casting strength specimen sets. Air content and

slump shall be determined in accordance with ASTM C 231 and ASTM C 143, respectively.

The CQA Consultant shall be responsible for reporting all test results to the Contractor and the Construction Manager. Materials determined by the Construction Manager to fail the requirements of the Construction Drawings and Technical Specifications shall be rejected.

14. GENERAL SITE WORK

14.1 Introduction

The CQA Consultant shall monitor the activities that are to be performed for various general site work items including, but not limited to riprap, erosion and sediment control, culverts, fences and gates, and vegetation for compliance with Construction Drawings and Technical Specifications.

14.2 Conformance Testing

Conformance testing of materials to ensure compliance with the Construction Drawings and Technical Specifications shall be performed by the CQA Consultant at the discretion of the Construction Manager. If nonconformances or other deficiencies are found by the CQA Consultant in the Contractors materials or completed work, the Contractor will be required to repair or replace the deficiency at no cost. Any noncompliant items shall be reported to the Construction Manager.

ATTACHMENT 21

SECTION 02720

GEOTEXTILES

PART 1 GENERAL

1.01 SCOPE

A. This section includes the requirements for geotextile products and installation.

1.02 RELATED SECTIONS AND PLANS

A. Section 02215 - Trenching and Backfilling

B. Section 02235 - Granular Drainage Materials

C. Construction Quality Assurance (CQA) Plan

1.03 REFERENCES

A. Latest version of American Society for Testing and Materials (ASTM) Standards:

1. ASTM D 3786. Standard Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabric-Diaphragm Bursting Strength Test Method.
2. ASTM D 4355. Standard Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water.
3. ASTM D 4491. Standard Test Method for Water Permeability of Geotextiles by Permittivity.
4. ASTM D 4533. Standard Test Method for Trapezoid Tearing Strength of Geotextiles.
5. ASTM D 4632. Standard Test Method for Breaking Load and Elongation of Geotextiles (Grab Method).
6. ASTM D 4751. Standard Test Method for Determining Apparent Opening Size of a Geotextile.
7. ASTM D 4833. Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
8. ASTM D 4873. Standard Guide for Identification, Storage, and Handling of Geotextiles.
9. ASTM D 5261. Standard Test Method for Measuring Mass Per Unit Area of Geotextiles.

B. Federal Standard No. 751a - Stitches, Seams, and Stitching.

1.04 SUBMITTALS

A. Submit the following to the Engineer for review not less than 21 calendar days prior to use.
   1. geotextile Manufacturer and product name;
   2. certification of minimum average roll values and the corresponding test procedures for all geotextile properties listed in Table 02720-1; and
   3. projected geotextile delivery dates.

B. Submit to the Engineer for review at least 14 calendar days prior to geotextile placement, manufacturing quality control certificates for each roll of geotextile as specified in this section.

1.05 CONSTRUCTION QUALITY ASSURANCE

A. The installation of geotextiles will be monitored by the CQA Consultant as required in the CQA Plan.

B. The CQA Consultant will perform material conformance testing of the geotextiles as required in the CQA Plan.

C. The Contractor shall be aware of the activities required of the CQA Consultant by the CQA Plan and shall account for these activities in the construction schedule.

D. The Contractor shall correct all deficiencies and nonconformances identified by the CQA Consultant at no additional cost to the Owner.

PART 2 PRODUCTS

2.01 GEOTEXTILE

A. Furnish geotextile products with minimum average roll values (95 percent lower confidence limit) meeting or exceeding the required property values in Tables 02720-1.

B. Furnish geotextiles that are stock products.

C. Furnish geotextiles that are manufactured from first quality polymers, with not more than 20 percent reclaimed polymer used in production.

D. Furnish polymeric threads for stitching that are ultra-violet (UV) light stabilized to at least the same requirements as the geotextile to be sewn. Furnish polyester or polypropylene threads that have a minimum size of 2,000 denier.

2.02 MANUFACTURING QUALITY CONTROL

A. Sample and test the geotextile to demonstrate that the material conforms to the requirements of this section.

B. Perform manufacturing quality control tests to demonstrate that the geotextiles properties conform to the values specified in Table 02720-1. Perform as a minimum, the following manufacturing quality control tests at a minimum frequency of once per 100,000 square feet:

Test	Procedure
Mass per unit area	ASTM D 5261
Grab strength	ASTM D 4632
Tear strength	ASTM D 4533
Puncture strength	ASTM D 4833
Burst strength	ASTM D 3786

C. Perform additional manufacturing quality control tests on the geotextile filter at a minimum frequency of once per 250,000 square feet, to demonstrate that its apparent opening size (ASTM D 4751) and permittivity (ASTM D 4491) conform to the values specified in Table 02720-1.

D. Submit quality control certificates signed by the geotextile manufacturer quality control manager. The certificates shall state that the geotextiles are continuously inspected and are needle-free. The quality control certificates shall also include: lot, batch, and roll number and identification; and results of manufacturing quality control tests including description of test methods used.

E. Do not supply any geotextile roll that does not comply with the manufacturing quality control requirements.

F. If a geotextile sample fails to meet the quality control requirements of this section, sample and test rolls manufactured at the same time or in the same lot as the failing roll. Continue to sample and test the rolls until the extent of the failing rolls are bracketed by passing rolls. Do not supply failing rolls.

2.03 PACKAGING AND LABELING

A. Supply geotextiles in rolls wrapped in relatively impermeable and opaque protective wrapping. Wrapping which becomes torn or damaged shall be repaired with similar materials.

B. Mark or tag geotextile rolls in accordance with ASTM D 4873 with the following information:
   1. manufacturer's name;
   2. product identification;
   3. lot or batch number;
   4. roll number; and
   5. roll dimensions.

C. Geotextile rolls not labeled in accordance with this section or on which labels are illegible upon delivery to the site shall be rejected and replaced at no expense to the Owner.

2.04 TRANSPORTATION

A. Deliver geotextiles to the site at least 14 calendar days prior to the planned deployment date to allow the CQA Consultant adequate time to perform conformance testing on the geotextile samples as described in the CQA Plan.

2.05 HANDLING AND STORAGE

A. Protect geotextiles from sunlight, moisture, excessive heat or cold, puncture, mud, dirt, and dust or other damaging or deleterious conditions. Follow all geotextile manufacturer recommendations for handling and storage.

B. Store geotextile rolls on pallets or other elevated structures. Do not store geotextile rolls directly on the ground.

C. Outdoor storage of geotextile rolls shall not exceed the manufacturer's recommendation or longer than 6 months, whichever is less. For storage periods longer than 6 months a temporary enclosure shall be placed over the rolls, or they shall be moved to an enclosed facility. The location of temporary field storage shall not be in areas where water can accumulate. The rolls shall be elevated off the ground to prevent contact with ponded water.

PART 3 EXECUTION

3.01 PLACEMENT

A. Do not commence geotextile installation until the CQA Consultant completes conformance evaluation of the geotextiles and performance evaluation of previous work, including evaluation of Contractor's survey results for previous work.

B. Handle geotextiles so as to ensure they are not damaged in any way.

C. Take necessary precautions to prevent damage to underlying layers including rutting during placement of the geotextiles.

D. After unwrapping the geotextiles from its opaque cover, do not leave them exposed for a period in excess of 30 calendar days.

E. If white colored geotextiles are used, take precautions against "snowblindness" of personnel.

F. Examine the geotextile surface after installation to ensure that no potentially harmful foreign objects are present. Remove any such objects and replace any damaged geotextiles.

3.02 SEAMS AND OVERLAPS

A. Continuously overlap a minimum of 6 inches and sew filter geotextiles (i.e., spot sewing is not allowed) using a "single prayer" seam. Sew seams using Stitch Type 401 as per Federal Standard No. 751a. In lieu of sewing, geotextile filters may be overlapped a minimum of two feet.

B. Do not install horizontal seams on slopes that are steeper than 10 horizontal to 1 vertical. Seams shall be along, not across, the slopes.

C. Overlap separator geotextiles a minimum of 12 inches and ensure that the overlap is maintained.

3.03 REPAIR

A. Repair any holes or tears in the geotextiles using a patch made from the same geotextile material. Extend geotextile patches a minimum of 1 foot beyond the

damaged area. Sew geotextile patches into place no closer than 1 inch from any panel edge. Should any tear exceed 50 percent of the width of the roll, remove and replace that roll.

B. Remove any soil or other material that may have penetrated the torn geotextiles.

3.04 PLACEMENT OF SOIL MATERIALS

A. Place soil materials on top of geotextiles in such a manner as to ensure that:
   1. the geotextiles and the underlying materials are not damaged; and
   2. slippage does not occur between the geotextile and the underlying layers during placement.

B. Spread soil on top of the geotextile to cause the soil to cascade over the geotextile rather than be shoved across the geotextile.

C. Place aggregate over geotextile separators as indicated on the Construction Drawings prior to trafficking.

D. Place soil over geotextile filters as indicated on the Construction Drawings prior to trafficking.

E. Do not drive equipment directly on the geotextile. Only use equipment above the geotextile that meets the following ground pressure requirements.

Maximum Allowable Equipment Ground Pressure (pounds per square inches)	Minimum Thickness of Overlying Material (inches)
<5	12
<10	18
<20	24
>20	36

TABLE 02720-1

REQUIRED PROPERTY VALUES FOR GEOTEXTILE

PROPERTIES	QUALIFIER	UNITS	SPECIFIED[1] VALUES	TEST METHOD
Type				
nonwoven needlepunched				(-)
Polymer composition	minimum	%	95 polypropylene or polyester by weight	(-)
Mass per unit area	minimum	oz/yd^2	8`	ASTM D 5261
Filter Requirements				
Apparent opening size (O_{95})	maximum	mm	0.21	ASTM D 4751
Permittivity	minimum	sec^{-1}	0.5	ASTM D 4491
Mechanical Requirements				
Grab strength	minimum	lb	180	ASTM D 4632[2]
Tear strength	minimum	lb	75	ASTM D 4533[3]
Puncture strength	minimum	lb	75	ASTM D 4833[4]
Burst strength	minimum	psi	350	ASTM D 3786
Durability				
Ultraviolet Resistance	minimum	%	70	ASTM D 4355

Notes:

(1) All values represent minimum average roll values.
(2) Minimum of values measured in machine and cross machine directions with 1 inch clamp on Constant Rate of Extension (CRE) machine.
(3) Minimum value measured in machine and cross machine direction.
(4) Tension testing machine with a 1.75-inch diameter ring clamp, the steel ball being replaced with 0.31-inch diameter solid steel cylinder with flat tip centered within the ring clamp.
(5) mm = millimeter
% = percent
oz/yd^2 = ounce per square yard
sec = second
lb = pound
psi = pound per square inch

[END OF SECTION]

ATTACHMENT 22

SARASOTA COUNTY

"Dedicated to Quality Service"

March 14, 2007

Ayushman Gupta, P.E.
GeoSyntec Consultants
14055 Riveredge Drive, Suite 300
Tampa, FL 33637

Re: Central County Solid Waste Disposal Complex
Flexible Leachate Container

Dear Ayushman:

One of the requests from FDEP pertains to an indication of the groundwater gradient. Ardaman Associates have been conducting a hydrological study on behalf of HDR in connection with the planning of the development of our Phase II, MSW site. The ongoing data from October 06 to February 07 is enclosed for your use.

The south west groundwater gradient, in the direction of Cow Pen Slough, is consistent with gradient reports since 1998.

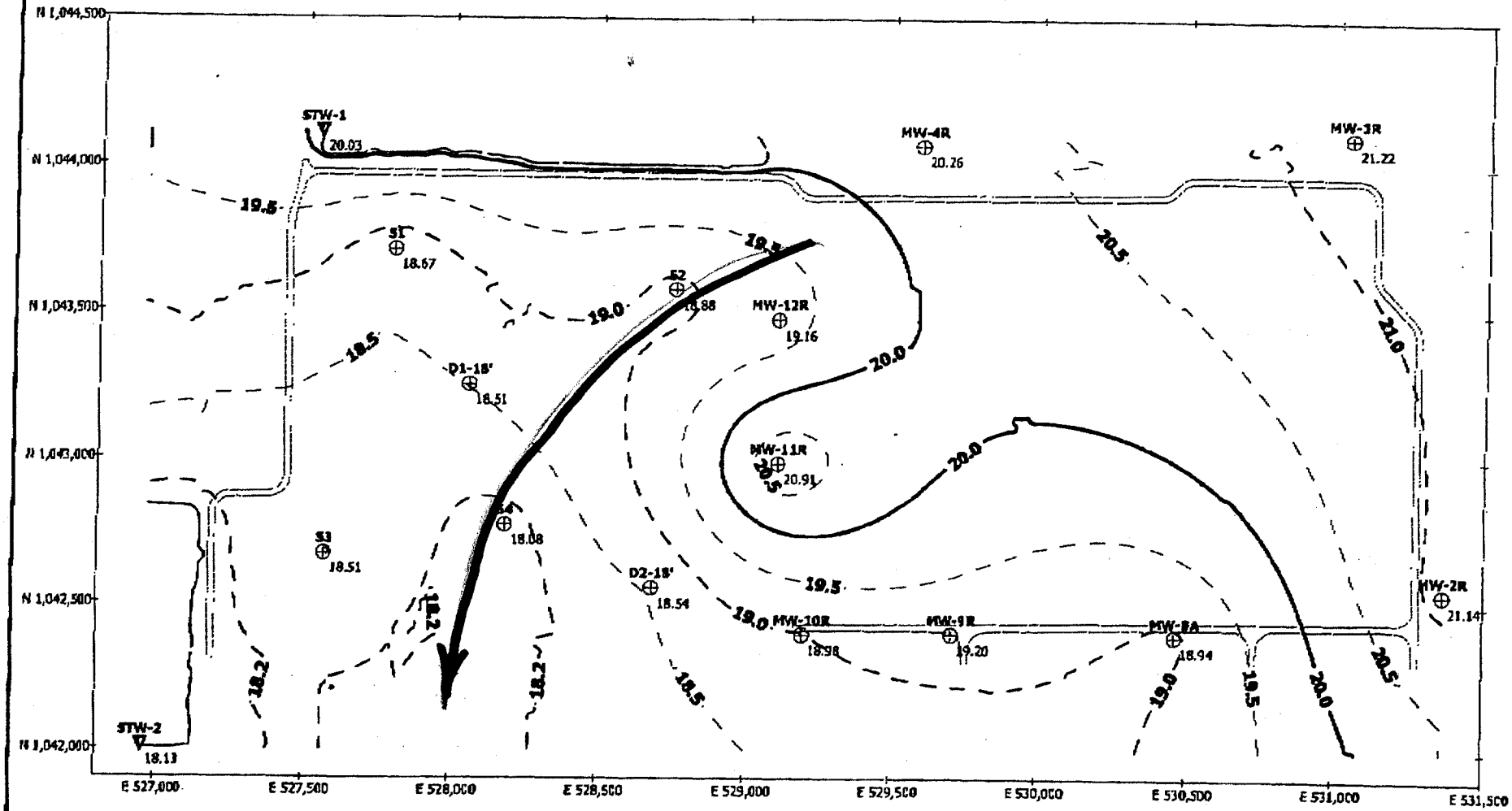
Please contact me if additional support is required.

Sincerely,

PA Wingler

Paul A. Wingler, P.E.
Project Manager

ENVIRONMENTAL SERVICES, Solid Waste Operations • 4000 Knights Trail Road, Nokomis, FL 34275
Tel 941-861-5000 • Fax 941-486-2620

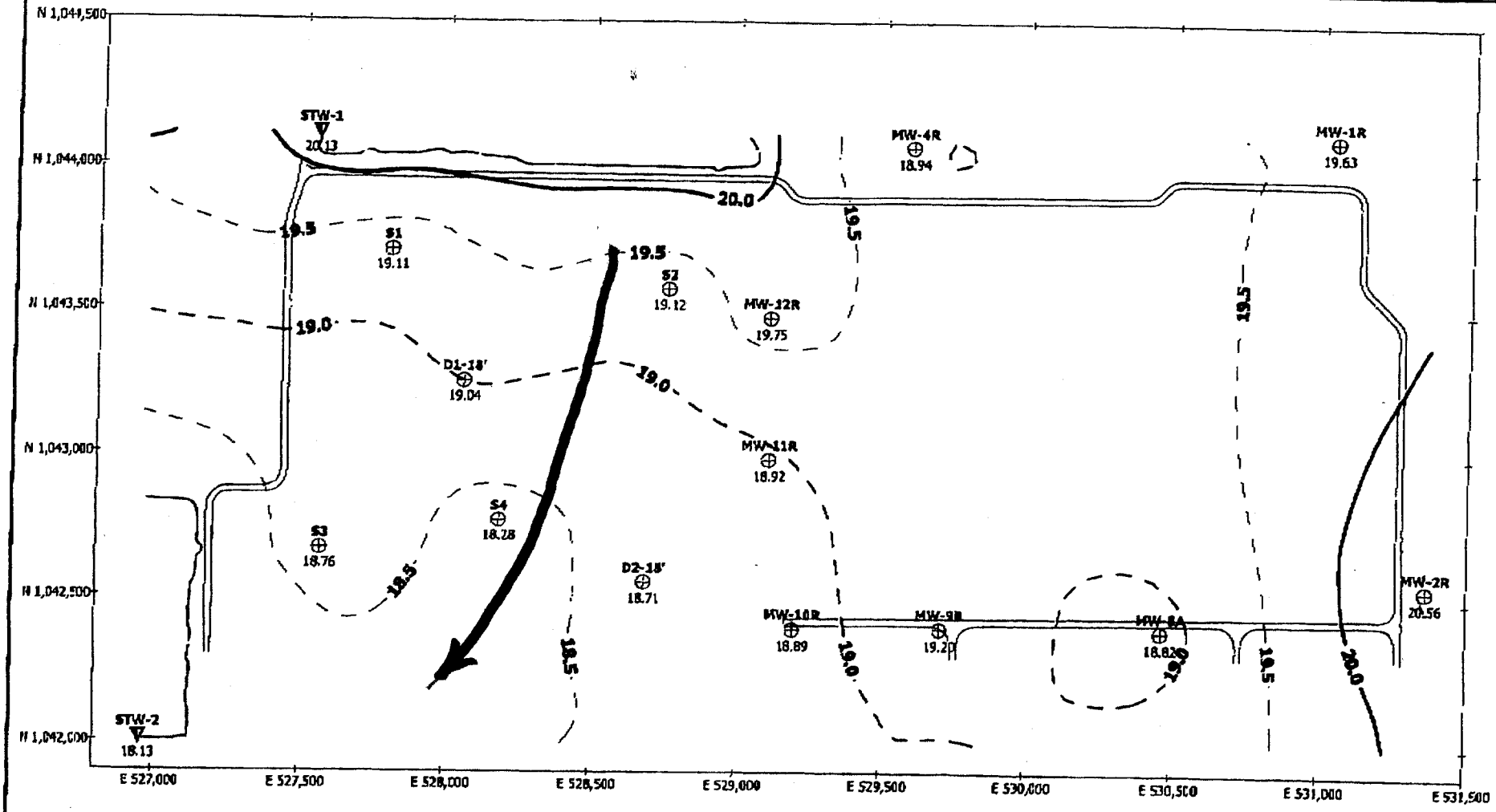


WATER TABLE MAP FOR FEBRUARY 2, 2007

Ardaman & Associates, Inc.
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL AND HYDROGEOLOGICAL EVALUATION FOR CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX PHASE II LANDFILL EXPANSION SARASOTA COUNTY, FLORIDA

DRAWN BY: PAX	CHECKED BY:	DATE: 03/06/2007
FILE NO. 05-035	APPROVED BY:	FIGURE: 11a

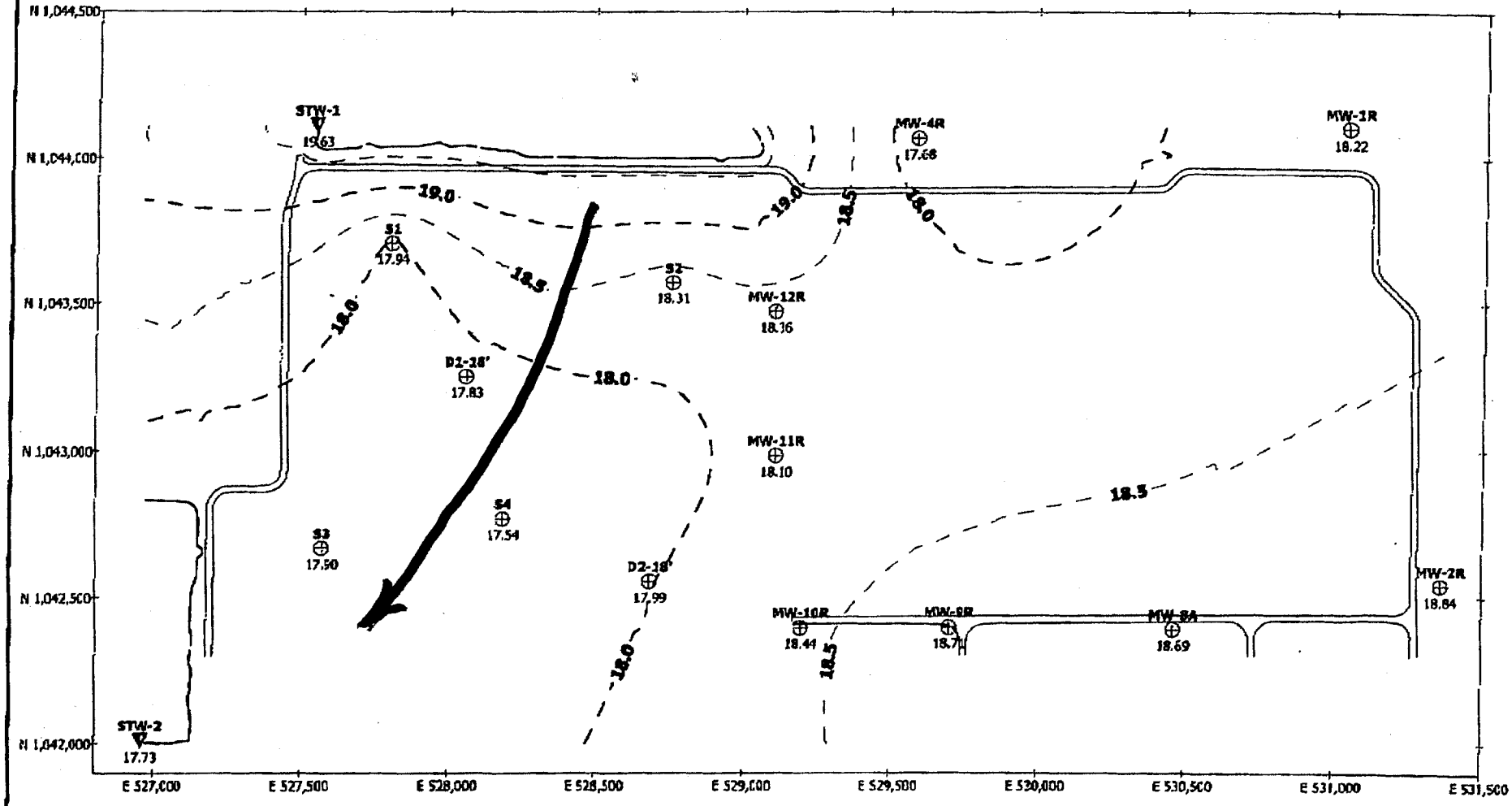


WATER TABLE MAP FOR JANUARY 10, 2007

Ardaman & Associates, Inc.
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL AND HYDROGEOLOGICAL EVALUATION FOR CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX PHASE II LANDFILL EXPANSION SARASOTA COUNTY, FLORIDA

DRAWN BY: PAK	CHECKED BY:	DATE: 01/15/2007
FILE NO. 06-035	APPROVED BY:	FIGURE 11

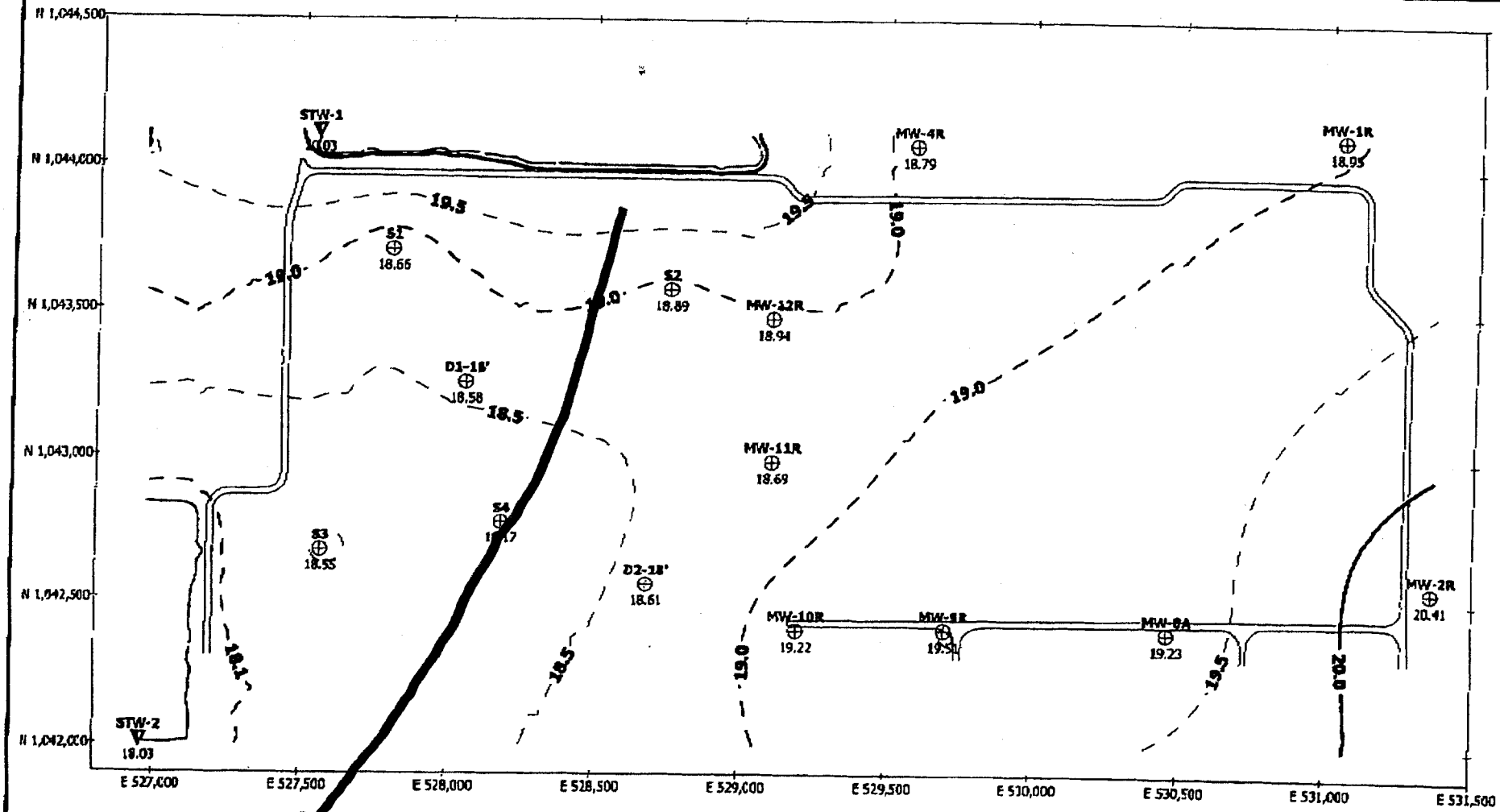


WATER TABLE MAP FOR DECEMBER 8, 2006

Ardaman & Associates, Inc.
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL AND HYDROGEOLOGICAL EVALUATION FOR CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX PHASE II LANDFILL EXPANSION SARASOTA COUNTY, FLORIDA

DRAWN BY: FAK	CHECKED BY:	DATE: 12/20/2006
FILE NO. 06-035	APPROVED BY:	FIGURE: 10

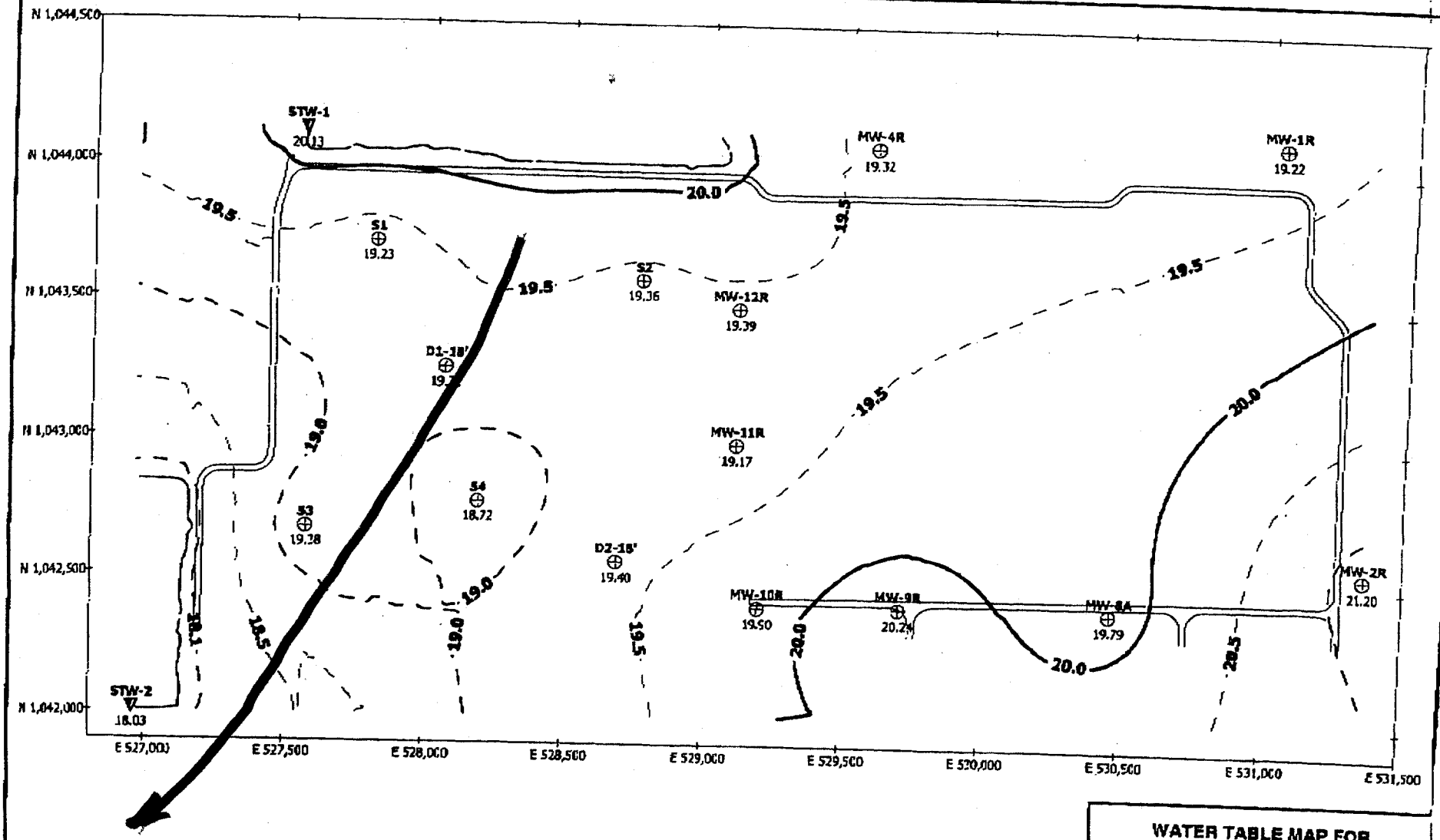


WATER TABLE MAP FOR NOVEMBER 9, 2006

Ardaman & Associates, Inc.
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL AND HYDROGEOLOGICAL EVALUATION FOR CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX PHASE II LANDFILL EXPANSION SARASOTA COUNTY, FLORIDA

DRAWN BY: PAK	CHECKED BY:	DATE: 12/20/2006
FILE NO. 06-035	APPROVED BY:	FIGURE 9



WATER TABLE MAP FOR OCTOBER 10, 2006

Ardaman & Associates, Inc.
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL AND HYDROGEOLOGICAL EVALUATION FOR CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX PHASE II LANDFILL EXPANSION SARASOTA COUNTY, FLORIDA

DRAWN BY: PAK	CHECKED BY:	DATE: 12/20/2006
FILE NO. 06-035	APPROVED BY:	FIGURE 8

ATTACHMENT 23

AFFIDAVIT OF PUBLICATION

SARASOTA HERALD-TRIBUNE
PUBLISHED DAILY
SARASOTA, SARASOTA COUNTY, FLORIDA

STATE OF FLORIDA
COUNTY OF SARASOTA

BEFORE THE UNDERSIGNED AUTHORITY PERSONALLY APPEARED SHARI BRICKLEY, WHO ON OATH SAYD SHE IS ADVERTISING MANAGER OF THE SARASOTA HERALD-TRIBUNE, A DAILY NEWSPAPER PUBLISHED AT SARASOTA, IN SARASOTA COUNTY FLORIDA; AND CIRCULATED IN SARASOTA COUNTY DAILY; THAT THE ATTACHED COPY OF ADVERTISEMENT BEING A NOTICE IN THE MATTER OF:

State of Florida Department of Environmental Protection Notice of Application The Department announces receipt of an application for permit to construct a flexible leachate storage container FLSC system, subject to Department rules, at the solid waste ma

IN THE COURT WAS PUBLISHED IN THE SARASOTA EDITION OF SAID NEWSPAPER IN THE ISSUES OF:

3/20 1x

AFFIANT FURTHER SAYS THAT THE SAID SARASOTA HERALD-TRIBUNE IS A NEWSPAPER PUBLISHED AT SARASOTA, IN SAID SARASOTA COUNTY, FLORIDA, AND THAT THE SAID NEWSPAPER HAS THERETOFORE BEEN CONTINUOUSLY PUBLISHED IN SAID SARASOTA COUNTY, FLORIDA, EACH DAY, AND HAS BEEN ENTERED AS SECOND CLASS MAIL MATTER AT THE POST OFFICE IN SARASOTA, IN SAID SARASOTA COUNTY, FLORIDA, FOR A PERIOD OF ONE YEAR NEXT PRECEDING THE FIRST PUBLICATION OF THE ATTACHED COPY OF ADVERTISEMENT; AND AFFIANT FURTHER SAYS THAT SHE HAS NEITHER PAID NOR PROMISED ANY PERSON, FIRM OR CORPORATION ANY DISCOUNT, REBATE, COMMISSION OR REFUND FOR THE PURPOSE OF SECURING THIS ADVERTISEMENT FOR PUBLICATION IN THE SAID NEWSPAPER.

SIGNED Shari Brickley

SWORN OR AFFIRMED TO, AND SUBSCRIBED BEFORE ME THIS 20th DAY OF MARCH, A.D., 2007 BY SHARI BRICKLEY WHO IS PERSONALLY KNOWN TO ME.

Bobbie J Clark
Notary Public

NOTARY PUBLIC STATE OF FLORIDA
Bobbie J Clark
My Commission DD347713
Expires October 11, 2008

My commission expires 11th day of Oct., 2008.

State of Florida
Department of Environmental Protection
Notice of Application

The Department announces receipt of an application for permit to construct a flexible leachate storage container (FLSC) system, subject to Department rules, at the solid waste management facility referred to as the Sarasota County Central Solid Waste Disposal Complex, located at 4000 Knights Trail Road, Nokomis, Sarasota County, Florida.

This application is being processed and is available for public inspection during normal business hours, 8:00 a.m. to 5:00 p.m., Monday through Friday, except legal holidays at the Department of Environmental Protection, Southwest District office, 13051 North Telecom Parkway, Temple Terrace, Florida 33637-0926.

Published: March 20, 2007