Part III

Attachment III-D
Appendix III-D.7-1

LINER QUALITY CONTROL PLAN – POORLY PERMEABLE DEMONSTRATION

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3-2-2015

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The following is included to satisfy the requirements of 30 TAC 330.337(b)(3) and is *"providing evidence satisfactory to the executive director that the soil surrounding the landfill is so **poorly permeable** that groundwater cannot move sufficiently to exert force that would damage the liner;"*

1.0 Subsurface Water Occurrence, Flow Characteristics, and Potentiometric Heads

Based on all available information, particularly that included in Part III, Attachment III-E and the accompanying Appendices, the following is noted as relevant to the demonstration:

1. The subsurface geology is typified by low-energy, fluvial sediments. Clayey materials predominate (over 90 percent) the subsurface soils with some fine sand and silt partings scattered through the clays and a few isolated sand/silt units. Materials are rock-like due to previous overconsolidation and limited mineral cementation.

2. The subsurface is anisotropic, i.e., at any point, horizontal flow potential is at least an order of magnitude greater than vertical flow potential. Therefore this demonstration is based on horizontal flow.

3. The great majority (over 90 percent) of potential sidewall materials are "practically impervious" clays with <u>horizontal</u> permeability, or hydraulic conductivity, K, in the mid 10^{-7} cm/sec range or even lower – see Appendix III-E.3. Note that permeability results determined from testing reflects the extensive secondary structure and the sand/silt partings that exist in the subsurface. Without the secondary structure, etc., the permeability of the clayey material would be several orders of magnitude less. This is consistent with their moderate to very high plasticity characteristics. The effective porosity of the overconsolidated clays has been determined to be in the two percent (2 %) range.

4. A few isolated, thin sandy/silty zones or units were encountered at random horizontal and vertical locations over the site and do not appear to be continuous. Based on boring log information and borehole geophysical logs (see Appendix III-E.2), several units considered to be the have the most permeable characteristics were investigated by field testing of piezometers screened across the identified sandy/silty units. The sandy/silty

zones had a <u>maximum</u> <u>horizontal</u> permeability, or hydraulic conductivity, K, in the range of 1 x 10^{-5} cm/sec although most results were in the mid to low 10^{-6} cm/sec range and a few were actually in the 10^{-7} range. (See Appendix III-E.4). The maximum thickness of the sandy/silty units is approximately ten feet and typically less. The effective porosity of these units is in the five percent (5 %) range.

5. For purposes of these calculations, we have conservatively assumed the existing water table coincides with the ground surface. This is a conservative assumption in that the 2012 Test Pits and existing surface water features indicate a much lower water table.

6. Assuming that the piezometers are hydraulically interconnected, average on-site gradients are in the range of 0.002 with a maximum gradient of approximately 0.0033 – see Figures 16-19 in the SIR (Appendix III-E.2). Regional aquifer gradients are in the same range as the geologic dip, i.e., approximately 50 feet to the mile or 0.01 feet per foot.

2.0 General Excavation and Excavation Dewatering Considerations

Based on the results of the site-specific investigations, subsurface water is expected to pose little threat to successful excavation.

1. Little if any seepage is expected – (a) due to the effective porosity of the subsurface materials, there is a very limited volume of water available for transmission; and (b) the little water that is available moves very slowly due to the practically impervious conditions. In addition, it is common to see a slight elastic rebound of the subsurface materials after excavation which can slightly increase effective porosity and in effect suck the water away from the excavation.

2. A benched sidewall has been designed to accommodate a staged sidewall liner construction due to the planned excavation depths of approximately 100 feet. If nuisance seepage is encountered during excavation, the benches provide a convenient opportunity to intercept the seepage, route it to a sump on the bench, and then pump it out.

3. If more than nuisance seepage is encountered, it would most likely be associated with a "channel" sand unit that was missed during the subsurface investigations. A simple temporary dewatering well could be installed to eliminate the seepage.

4. Hydrostatic heave or uplift of the excavation sidewalls, and particularly the bottom, is not expected. Investigations to date have not indicated any potential for such. There is so little water quantity, and the prevailing permeability of the subsurface soils is so low, that there is no opportunity for damaging uplift.

3.0 Liner Tensile Strain Design for Uplift

The commonly-used design procedure promoted by U.S. EPA is based on a "bubble" analog where allowable tensile strain is defined as a function of the distortion or displacement, **Δ**, perpendicular to the original plane of the liner divided by the radius of the bubble, **L**. Figure 2-16 – Relationship between distortion and tensile strain, (U.S. EPA, 1991 – see attached) shows the non-linear relationship between **Δ/L** and **Tensile Strain (%)**. As can be seen on the attached figure, using the U.S. EPA-recommended allowable tensile strain of 0.1%, an allowable **Δ/L** of 0.05 results.

Although the bubble is three dimensional, the common solution is two-dimensional for liner, i.e. per foot of width – usually along a sidewall. Assuming that the two-dimensional bubble cross-section can be approximated as a circular segment, the limiting area of the circular segment can be approximated to adequate accuracy by determining the area of a circular sector. The equation is:

$$\mathbf{A} = (2/3)\,\mathbf{ch} + \mathbf{h}^3/2\mathbf{c}$$

Where:

$\mathbf{c}$ = chord length

$\mathbf{h}$ = height from chord to top of circular segment

For this demonstration, we will use the following version:

$$\mathbf{A} = (4 \cdot \mathbf{L} \cdot \mathbf{\Delta})/3 + \mathbf{\Delta}^3/(4 \cdot \mathbf{L})$$

Where:

$\mathbf{A}$ = the area of the circular segment

$\mathbf{L}$ = ½ of the circular segment chord length **c**,

$\mathbf{\Delta}$ = the height of the segment, or **h**, and

$\mathbf{\Delta/L} \leq 0.05$

Volume, **V**, per foot of width is simply **A** expressed as feet^3/foot of width.

4.0 Site-Specific Liner Uplift and Tensile Strain Calculations

Assume:

- a ten-foot-thick "more permeable" sand unit at the bottom of the excavation
- a 100-foot-deep excavation
- water table corresponding to the ground surface
- clay materials above sand unit have been effectively dewatered by excavation
- sidewall liner will be placed on a 3H:1V nominal sideslope (18.435°).
- critical area for liner uplift and tensile strain is across the permeable sand unit, i.e., liner spanning limited flow area results in minimum water volume needed for stressing liner
- two-dimensional analysis is representative due to the large excavation (length and width)
- liner weight and weight of any waste are not considered in the analysis
- U.S. EPA recommendation for tensile strain limit is valid

4.1 Volume of water required to distort liner to allowable tensile strain.

Based on a liner segment, on a 3H:1V (18.435°) slope, covering a 10-foot thick sand unit, the volume of water needed to exceed a liner distortion, **Δ/L** = 0.05, is:

- Length of liner cross-section 2**L** = 10' / sin 18.435° = 31.62 feet and **L** = 15.81feet
- maximum liner displacement, **Δ**, corresponding to **Δ/L** = 0.05, is 0.05 · L , and **Δ** = 0.05 · 15.81' = **0.79 feet**
- using the volume equation previously given above, the corresponding volume, **V** is 16.66 feet^3/foot of width.

4.2 Subsurface water flow available to distort liner

Assumptions:

- Excavation has been open long enough that steady-state flow conditions in the sand unit are present

- maximum permeability for the sand unit, K = 3.7 x 10^{-5} cm/sec (0.0992 feet/day)
- sand unit is saturated over its entire thickness
- sand unit flow occurs under 'confined conditions" due to the greatly reduced permeability of the clay confining units above and below the sand unit
- Darcy's law is valid

Calculations:

Darcy's Law for flow in a saturated medium is simply

$$\mathbf{Q = K \cdot i \cdot A}$$

Where:

Q = rate of flow of the fluid (units of volume or length^3 per time),

K = permeability, hydraulic conductivity or coefficient of permeability (units of length per time),

i = hydraulic gradient (dimensionless, i.e., no units as in foot per foot)

A = cross-sectional flow area perpendicular to the flow direction (units of length^2)

For the ten-foot thick sand unit and assuming a maximum on-site piezometer gradient of 0.0033 (steady-state conditions), the flow rate per foot of width is

$$\mathbf{Q} = (0.0992\ \text{feet/day})(0.0033)(10\ \text{feet}^2\text{/foot of width}) = 0.0033\ \text{foot}^3\text{/day/foot of width}$$

4.3 Time required to distort liner up to the allowable tensile strain.

If the volume of water required to distort the liner (up to the allowable tensile strain limit) is 16.66 feet^3 per foot of width, the time required for liner distortion is simply:

Time required = **V** / **Q** = 16.66 ft^3 / foot ÷ 0.0033 ft^3 / foot / day = 5,088 days (*13.9 years*)

Considering that the time frame for liner distortion was based on maximum permeability and maximum gradient, the calculations demonstrate that the subsurface is poorly permeable in accordance with 30 TAC §330.337 (b)(3).

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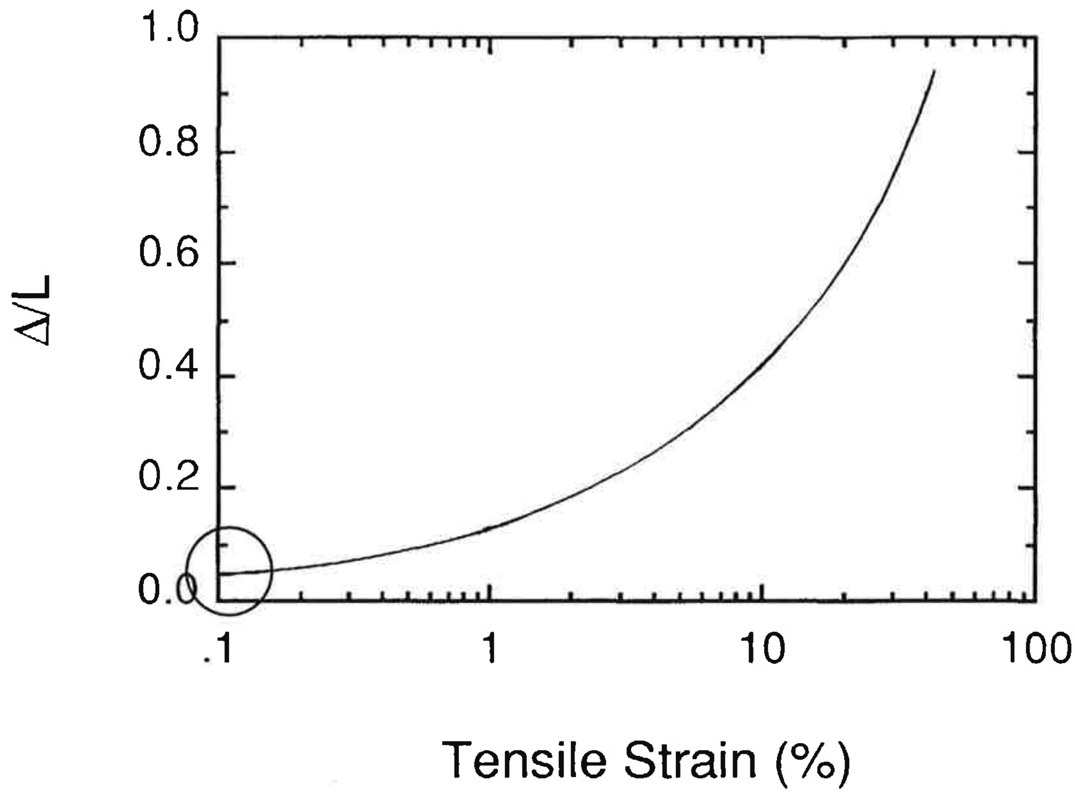

Figure 2-16. Relationship between distortion and tensile strain (9).

conditions when analyzing the stability of the cover system.

Methods of measuring interfacial friction between geosynthetic/geosynthetic or geosynthetic/soil interfaces are reviewed in detail by Takasumi et al. (11). No standard testing method exists, although one is under development by ASTM.

Seed and Boulanger (12) measured interfacial friction angles between a smooth high density polyethylene (HDPE) geomembrane and a compacted soil-bentonite mixture that contained 5 percent bentonite by dry weight. Interfacial friction angles were found to be very sensitive to compaction water content, dry unit weight, and the degree of wetting of the soil. For a given dry unit weight, increasing the molding water content or wetting the compacted soil reduced the interfacial friction angle. Increasing the density typically reduced the interfacial friction angle, as well. Unfortunately, the compaction conditions that would yield minimal hydraulic conductivity (i.e., compaction wet of optimum with a high energy of compaction) also yielded the lowest interfacial friction angles. Seed and Boulanger reported interfacial friction angles that were typically 5 to 10 degrees for the water content—unit weight combinations that would typically be employed to achieve minimal hydraulic conductivity.

The study of interfacial friction problems is an area of active research. At the present time, designers are cautioned to give careful consideration to the problem and to measure friction angles along all potential sliding surfaces using the proposed construction materials for testing. If adequate stability is not provided, the designer will need to consider alternative materials (e.g., rougher geomembranes with higher interfacial friction angles), flatter slopes, or reinforcement of the cover, e.g., with geogrids.

DRAINAGE LAYERS

Drainage layers are high-permeability materials used to drain fluids (such as infiltrating water) or gas produced from the waste. A drainage layer installed to drain infiltrating water is called a surface water collection and removal system. The hydraulic conductivity required for this layer depends upon the rate of infiltration, the slope of the layer, and the hydraulic conductivity of the underlying barrier layer. However, the efficiency of the drainage layer improves as the hydraulic conductivity of the drainage material increases. Thus, high hydraulic conductivity is a requirement for drainage layers.

The single most important factor controlling the hydraulic conductivity of sands and gravels is the amount of fine-grained material present. Geotechnical engineers define fine-grained materials as those materials that will pass through the openings of a No. 200 sieve (0.075 mm openings). A relatively small shift in the amount of fines