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TOMOKA FARMS ROAD LANDFILL
NORTH CELL
LEAK DETECTION AND LEACHATE SYSTEM
DESIGN AND OPERATION

Prepared for

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IN-Leak Detect
& Leachate
System

8-8-1997

July 9, 1997
0995039.05

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29 Jul 97

SECTION 1

INTRODUCTION

On April 19, 1994, the Florida Department of Environmental Protection (FDEP) issued a permit to construct Volusia County's Tomoka Farms Road Landfill North Cell Expansion. At Volusia County's request, SCS Engineers (SCS) has made several significant changes to the proposed plans, including the following:

- Addition of a Geosynthetic Clay Liner: The original design had a single 60 mil high density polyethylene (HDPE) geomembrane for the primary liner, overlaying a geonet leak detection layer and a HDPE/geosynthetic clay liner (GCL) composite secondary liner. The County is now proposing to place an additional GCL such as Claymax or Bentomat directly under the primary geomembrane liner. This will greatly reduce the volume of leakage from the landfill.

- Modification to Leak Detection System (LDS) Laterals: Addition of the primary GCL greatly reduces the amount of flow expected in the leak detection system. Replacing the originally proposed 4-inch leak detection laterals with a layer of triplanar geonet such as Tenax will reduce the possibility of damage caused by the transportation of rock backfill over the exposed secondary liner during construction of the leak detection laterals.

- Elimination of Liner Penetrations: The original design included a penetration of the secondary liner at the lowest point of each of the three leachate sumps and both of the leachate basin sumps. Sumps are the most critical part of the liner system as they experience the maximum head of leachate. Sump penetrations are difficult to construct and difficult to test for leakage. Post construction settlement may overstress the penetration welds and create problems that would be very difficult to correct. The proposed LDS side slope riser design eliminates the sump penetrations.

- LDS Pump Modifications: The original design called for three, 100 gallons per minute (gpm) pumps to convey leakage from the LDS to the leachate collection sumps. With the secondary GCL, the leakage into the LDS is estimated to be less than 2 gallons per day the entire landfill. As a result, the three 100 gpm pumps have been replaced with three 15 gpm pumps located in side slope risers.

- HDPE Pipe Modifications: The original design used solid wall HDPE pipe for leachate collection pipe, leak detection pipe, and side slope risers. ADS N-12 pipe is HDPE pipe constructed with a smooth interior and corrugated exterior. The structural shape provides stiffness and compressive load bearing strength more efficiently than heavy, thick walled pipe. The County proposes to use the ADS N-12 pipe in lieu of the solid wall pipe previously proposed. Structural calculations for this pipe are enclosed.

- Phased Construction: The County intends to construct the entire North Cell as indicated on the Drawings. However, the project has been divided into five cells. Once the first cell is complete, including the first leachate basin, pumps, and piping, the County will request permission to place solid waste into the first cell while the remaining cells are being constructed. This phasing will allow the County to begin placing solid waste into the lined cell at an earlier date than they would if they had to complete the entire 33 acres. It will also allow the County to carefully place selective waste for the first level of the new cell while the old landfill is still in use for bulky waste or waste the County may not wish to place in the first lift over the new liner.

- Revised Landfill Bottom Elevations: A subsurface geotechnical investigation was conducted in support of the proposed design. Ten borings were installed in the 33-acre expansion area. Laboratory tests were conducted to assess structural characteristics of the formation soils. Stability and settlement calculations were performed for the proposed design configuration. Based on the recommendations of the geotechnical report, the slopes for the leachate collection lateral and header pipes and for the leak detection swales and header pipes were increased to accommodate the anticipated subgrade settlement.

The purpose of this report is to provide information for permit review on the design and performance of the modified Leak Detection System (LDS), and the structural design of the piping materials proposed for use in the LDS and the Leachate Collection System. A separate geotechnical engineering report has also been prepared describing the results of the subsurface investigation, discussing the anticipated settlement and slope stability, and presenting recommendations for dewatering during construction.

SECTION 2

PROPOSED LEAK DETECTION SYSTEM

2.1 PROPOSED LINER SYSTEM

The proposed liner system for the Tomoka Farms Road Landfill Expansion, North Cell, from top to bottom, consists of the following:

- 2-foot thick protective soil cover.
- Leachate Collection System (LCS) geocomposite drainage layer, consisting of a 0.2-inch thick geonet with 7-ounce non-woven geotextile heat bonded to both sides of the geonet (Fluid Systems, or approved equal).
- composite primary liner composed of a 60-mil thick HDPE geomembrane (GSE, National Seal Co., Poly America, or approved equal) placed on top of a low-permeability bentonite geosynthetic (Claymax, Bentomat, or approved equal).
- Leak Detection System (LDS) geocomposite drainage layer, consisting of a 0.25-inch thick tri-planar geonet with 7-ounce non-woven geotextile heat bonded to both sides of the geonet (Tenax, or approved equal).
- composite secondary liner composed of a 60-mil thick HDPE geomembrane (GSE, National Seal Co., Poly America, or approved equal) placed on top of a low-permeability bentonite geosynthetic (Claymax, Bentomat, or approved equal).
- prepared subgrade

2.2 PROPOSED LDS COLLECTION SYSTEM

The LDS geonet drainage layer rests directly on the secondary liner geomembrane, which is sloped at 2 percent toward the collector swales. The collector swales are located approximately 160 feet apart. Each collector swale is approximately 285 feet long, sloping at 1 percent toward the LDS header. In the collector swale, a 6.7-foot wide second layer of tri-planar geonet transports the LDS flow to the LDS headers. The three LDS headers are 6-inch diameter perforated ADS N-12 Landfill Grade corrugated HDPE pipe. The pipe slopes at 0.35 percent toward the LDS sumps. A pump is installed in each sump to remove flow that accumulates.

It is anticipated that the landfill subgrade will settle as the landfill is built up, with the maximum settlement of 0.74 feet (ft) occurring at the center of the landfill where the depth of solid waste will be greatest. The settlement expected to take place at the edge of the landfill is estimated to be 0.32 ft. This differential settlement will reduce the slope in the collector swales to as low as 0.50 percent, and in the leachate header to 0.22 percent.

SECTION 3

ESTIMATED LEAKAGE

3.1 BACKGROUND

Quality control procedures during construction include testing for leaks and repair of all leaks detected at the time of installation. The primary liner will be protected by the geocomposite and the two feet of sand drainage layer. The initial layer of solid waste will be screened and placed to avoid damage to the liner.

The primary and secondary liners are composite liners, which consist of an HDPE geomembrane placed on top of a bentonite geosynthetic clay liner (GCL). Leakage through composite liners is primarily due to leakage through defects (e.g., holes) in the geomembrane. Leakage due to permeation through geomembranes can be considered negligible.

Leakage rates through composite liners are a function of many parameters, including hydraulic head, size of the considered geomembrane hole, thickness, and hydraulic conductivity of the clay underlying the geomembrane, and quality of contact between the geomembrane and the underlying soil. The *good contact condition* corresponds to a geomembrane installed, with as few wrinkles as possible, on top of a GCL that has been placed on a soil layer which 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, on top of a GCL that has been placed on a soil that has not been well compacted and does not appear smooth.

The Florida Department of Environmental Protection (FDEP) recommends the use of an average contact condition to calculate leakage through composite liner systems. The leakage rate equation, which uses an average of coefficients from equations for the good contact and poor contact conditions, is:

$$Q_{tb} = 0.6\, C\, h_{tb}^{0.9}\, a^{0.1}\, k_{st}^{0.74} \qquad \text{(Equation 1)}$$

where:
- Q_{tb} = rate of leakage through the composite primary liner due to holes in the geomembrane (gal/day)
- C = conversion constant (2.28×10^7 gal-sec/day/m^3)
- h_{tb} = hydraulic head on top of the liner (m)
- a = area of the geomembrane hole (m^2)
- k_{st} = hydraulic conductivity of the bentonite geosynthetic underlying the geomembrane (m/sec)

(from FDEP, Municipal Solid Waste Landfill Alternative Design Closure Guidance, February 10, 1995, p.10)

3.2 CONSTRUCTION QUALITY

It is assumed that the liner 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 size and frequency of defects considered in the calculations were as follows (FDEP, 1995):

- *Hole Size.* A hole size of 1 square centimeter is appropriate for geomembrane liners installed with proper construction workmanship and very good CQA.

- *Hole Frequency.* A frequency of 1 defect per acre has been selected for leakage calculations.

3.3 HEAD OF LEACHATE ON THE PRIMARY LINER

The maximum head of leachate on the primary liner has been taken as the thickness of the LCS drainage layer, i.e., 0.20 inches (in) (5 mm). This assumption has been verified by the Hydrologic Evaluation of Landfill Performance (HELP) model, which showed an average annual head over liner value of approximately 0.2 in.

3.4 LEAKAGE RATE CALCULATIONS

The leakage rate through the primary liner is determined from Equation 1 using the following parameters:

- leachate head above the geomembrane,

$$h_{tb} = 0.2 \text{ in } (5 \times 10^{-3} \text{ m})$$

- area of geomembrane hole,

$$a = 1 \text{ cm}^2 \ (10^{-4} \text{ m}^2)$$

- hydraulic conductivity of the bentonite geosynthetic,

$$K_{st} = 1 \times 10^{-9} \text{ cm/sec } (1 \times 10^{-11} \text{ m/sec})$$

From Equation 1, the calculated leakage rate through a hole in the primary geomembrane is calculated as follows:

$$Q_{tb} = 4.2 \times 10^{-4} \text{ gal/day.}$$

The groundwater leakage rate through the secondary liner, under the maximum head of 12.5 ft (3.8 m) is calculated to be 0.17 gal/day/acre. Therefore, the total leakage into the LDS, assuming a frequency of defects of one hole per acre, is approximately 0.17 gal/day/acre.

SECTION 4

FLOW CAPACITY

The LDS must be capable of conveying the maximum flow rate into the LDS without excessive build-up of hydraulic head above the secondary liner. As previously discussed, the flow into the leak detection system is estimated to be 0.17 gal/day/acre or approximately 5.61 gal/day assuming a frequency of defects of one hole per acre over 33 acres.

4.1 LDS DRAINAGE LAYER FLOW CAPACITY

In order for the LDS drainage layer to have sufficient flow capacity, the thickness of the flow must be less than the thickness of the LDS drainage layer. The thickness of flow in the LDS drainage layer may be determined from Darcy's equation:

$$D_{db\,min} = \frac{Q_{ts}}{B_f\, k_{db}\, i_b} \qquad \text{(Equation 2)}$$

where:
- $D_{db\,min}$ = thickness of flow in the LDS drainage layer due to leakage through geomembrane holes
- Q_{ts} = leakage rate through a hole of geomembrane liner
- B_f = width of leakage flow
- k_{db} = hydraulic conductivity of the LDS drainage material
- i_b = hydraulic gradient along the base

The design leakage rate through a geomembrane hole, Q_{ts}, is 0.17 gal/day (7.5 x 10^{-9} m^3/s). The hydraulic conductivity of the LDS drainage material is K_{dbs} = 15.6 cm/s (0.156 m/s), and the hydraulic gradient is i_b = 0.02. A conservative leakage flow width B_f =3.3 ft (1 m) was selected. Therefore, the flow thickness is, according to Equation 2:

$$D_{db\,min} = 2.4 \times 10^{-6}\ \text{in}$$

The thickness of the LDS drainage layer is greater than the thickness of leakage flow. Therefore, the proposed LDS drainage layer has adequate flow capacity.

4.2 LDS COLLECTOR SWALE FLOW CAPACITY

The flow capacity of the LDS collector swales must be greater than the rate of flow entering the collector swales. The rate of flow entering the LDS collector swale may be calculated by multiplying the leakage rate through the liner by the plan area of the LDS drained by the collector swale (approximately 1 acre):

$$Q_{ts} = Q_{ts} \times A_d \qquad \text{(Equation 3)}$$

$$Q_{sr} = (0.17\ \text{gal/day/acre})((160\ \text{ft} \times 285\ \text{ft})\ /43{,}560\ \text{ft}^2\text{/acre})$$

Q_{sr} = 0.18 gal/day

= 1.2×10^{-4} gpm

Since the liner defects are not likely to be distributed uniformly over the landfill, the swale should be designed to handle at least ten times the expected frequency, or 1.2×10^{-3} gpm.

The flow capacity of the LDS collector swales my be calculated from Darcy's equation:

$$Q_{sa} = K_{sc}\, i_{sc}\, B_{sc}\, D_{sc} \qquad \text{(Equation 4)}$$

where:
- Q_{sa} = LDS collector swale flow capacity;
- K_{sc} = hydraulic conductivity of the LDS collector swale material;
- i_{sc} = hydraulic gradient along the LDS collector swale;
- B_{sc} = width of the LDS collector swale; and
- D_{sc} = depth of the LDS collector swale;

For the LDS collector swale, the hydraulic conductivity is K_{sc} = 63.0 cm/s (0.63 m/s), and the width and depth are B_{sc} = 6.7 ft (2.0 m) and D_{sc} = 0.25 in (6.4×10^{-3} m), respectively. The hydraulic gradient of the LDS collector swale is i_{sc} = 0.0050. Therefore, Equation 4 yields:

Q_{sa} = 0.64 gpm (4.0×10^{-5} m^3/s).

The calculated flow capacity of the LDS collector swale is greater than the calculated rate of flow entering the collector swale, therefore, the calculated flow capacity of the proposed LDS collector swale is adequate.

4.3 LDS HEADER FLOW CAPACITY

The LDS headers must be capable of conveying the maximum flow rate delivered by the collector swales without excessive build up of hydraulic head above the secondary liner. The capacity of each collector swale, Q_{sa}, has been calculated above to be 0.64 gpm. The central header drains 14 swales, seven on the north side and seven on the south, for a total maximum flow capacity of 8.96 gpm. The header must be able to accept 8.96 gpm to prevent hydraulic backup into the LDS geonet during a failure event.

The flow capacity of the LDS header pipe is calculated from Manning's Equation:

$$Q_{ha} = A_{hf}\left(\frac{1.486}{n}\right) R^{.67} S^{.50} \qquad \text{(Equation 5)}$$

where:
- Q_{ha} = LDS header flow capacity (cubic feet per second (cfs))
- A_{hf} = cross sectional area of flow in the LDS header (sf)
- n = Manning's roughness factor (dimensions)
- R = hydraulic radius of the LDS header at a given depth of flow (ft)
- S = slope of the LDS header (ft/ft)

For the LDS header, the nominal diameter is 6 inches, the actual diameter is 6.05 inches, the Manning's "n" is 0.012, and the slope is 0.0022. Using Equation 5, the flow and velocity in the LDS header for various depths of flow are calculated to be as shown in Table 4-1

TABLE 4-1 FLOW DEPTH VERSUS FLOW

Depth of Flow, in	Flow in GPM	Velocity ft/sec
0.5	0.01	0.1
1.0	7.7	0.8
1.5	17.5	1.0
2.0	30.6	1.2
2.5	46.4	1.3
3.0	64.1	1.4
3.5	82.5	1.5
4.0	100.7	1.6
4.5	117.4	1.6
5.0	131.0	1.7
5.5	139.1	1.6
6.0	134.9	1.5

The maximum flow into the LDS header, 8.96 gpm, will produce a depth of flow of less than 2 inches. The calculated flow capacity of the LDS header is adequate.

SECTION 5

LEAK DETECTION TIME

Leak Detection Time is the time it takes a leak to travel from the leak location to the sump. A short detection time is important in order to:

- minimize the residence time of the leachate on the secondary liner, thereby minimizing leakage through the secondary liner; and

- permit rapid detection and evaluation at the LDS sump of leakage into the LDS.

5.1 FLOW PATH OF LEAKAGE IN THE LDS

The time to detect leakage after it has passed through the primary liner is the time it takes the leakage to travel from the leak location to the LDS sump. The leakage path, from the leak location to the sump, includes three parts.

Leakage flows within the LDS drainage layer from the leak location to the collector swale. Since leak location is unknown, the travel distance in the LDS drainage layer is taken as the maximum distance between a point in the LDS drainage layer and the collector swale.

After the leakage reaches the collector swale, the travel time is essentially the time it takes the collector swale to convey the leachate to the LDS header. The travel distance can be conservatively taken as the maximum length of the collector swales. The travel distance in the LDS header can be conservatively taken as the maximum length of the header.

5.2 METHOD OF ANALYSIS

The leak detection time is typically calculated assuming steady-state flow conditions. The leakage travel time is then calculated using Darcy's equation, modified to take into account drainage material porosity:

$$t_t = n\, l/(ki) \qquad \text{(Equation 6)}$$

where;
t_t = leakage travel time;
n = porosity of the drainage material;
l = length of drainage path, measured along the slope;
k = hydraulic conductivity of the drainage material: and
i = hydraulic gradient.

In the following sections, steady-state conditions are assumed for the calculations of leakage travel time within the LDS drainage layer and the LDS collector swales.

5.2.1 Leakage Travel Time in the LDS Drainage Layer

As discussed above, leakage reaches a collector swale by traveling within the LDS drainage layer. The travel time to be considered in the leak detection time calculations is this travel time.

The leakage travel time in the LDS drainage layer is given by:

$$t_{td} = n_{db} l_b / (K_{db} \, i_b) \qquad \text{(Equation 7)}$$

where:
- t_{td} = leakage travel time in the LDS drainage layer;
- n_{db} = porosity of the LDS drainage material;
- l_b = length of the drainage path;
- K_{db} = hydraulic conductivity of LDS drainage material; and
- i_b = hydraulic gradient.

5.2.2 Leakage Travel Time in the LDS Collector Swale.

The leakage travel time is given by:

$$t_{ts} = n_{sc} l_{sc} / (K_{sc} \, i_{sc}) \qquad \text{(Equation 8)}$$

where;
- t_{ts} = leakage travel time in the LDS collector swales;
- n_{sc} = porosity of the LDS collector swale material;
- l_{sc} = length of the drainage path for the LDS collector swale;
- K_{sc} = hydraulic conductivity of the LDS collector swale material; and
- i_{sc} = hydraulic gradient for the LDS collector swale.

5.2.3 Leakage Travel Time in the LDS Header

The leakage travel time is given by:

$$t_{th} = l_n / v_h$$

where: l_n = length of the LDS header and v_h is the velocity of the flow in the LDS header.

5.2.4 Leak Detection Time Determination

The leak detection time is given by:

$$t_d = t_{td} + t_{ts} + t_{th} \qquad \text{(Equation 9)}$$

5.3 CALCULATIONS

5.3.1 Pertinent Data

The following parameters are assumed, which are associated with the longest drainage path between a leak and a collector swale and the longest drainage path in the collector swale:

- $n_{db} = n_{sc}$ = geonet porosity = 1
  (The actual geonet porosity will be less than 1; it is conservative to assume a value of 1 for the detection time calculation).

- l_b = length of drainage path along the LDS drainage layer = 80 ft (24 m).

- K_{db} = hydraulic conductivity of the LDS drainage material = 63.0 cm/s (0..63 m/s).

- i_b = minimum gradient along the base = 0.02.

- K_{sc} = hydraulic conductivity of the LDS collector swale material = 63.0 cm/s (0.63 m/s).

- l_{sc} = length of the LDS collector swale = 280 ft (85 m).

- i_{sc} = gradient along the LDS collector swale = 0.0050.

5.3.2 Leak Detection Time

Based on Equation 7, the leakage travel time in the LDS drainage layer is:

$$t_{td} = 1{,}900 \text{ sec (32 min)}$$

Equation 8 gives the following travel time in the LDS collector swale:

$$t_{ts} = 27{,}000 \text{ sec (450 min)}$$

Travel time in the LDS header is dependent on the depth of flow in the header. With a depth of flow of 0.5 inches, the velocity in the LDS header is shown in Table 1 to be approximately 0.1 ft/sec. The distance from the sump to the farthest collector swale is 1000 ft, so the travel time is calculated to be 10,000 sec or approximately 167 minutes.

The total time required for a leak to flow to the leachate header is given by:

$$t_d = 32 \text{ min} + 450 \text{ min} + 167 \text{ min} = 649 \text{ min} = 10.8 \text{ hours}$$

SECTION 6

PUMP STATIONS

The three LDS pump stations must be designed to pump the maximum flow that the LDS headers can deliver. This flow is limited by the capacity of the collector swales, previously calculated to be 8.96 gpm. To allow for additional leakage during construction, the pumps have been designed with a capacity of 15 gpm. The pumps would draw from the LDS sump, with a low shut off water level of 10.0, and discharge into the leachate pump side slope riser at elevation 32.0, for a total static head of 22.0 ft. Allowing 4.0 ft. of friction head loss in the 1½-inch discharge line results in a total discharge head of 26.0 ft. The pumps would therefore be designed to pump 15 gpm at a head of 26.0 ft.

SECTION 7

LEACHATE COLLECTION PIPING MODIFICATIONS

The pipe to be used for the leachate collection system and leak collection system is a corrugated HDPE pipe with an integrally formed smooth interior, as manufactured by Advanced Drainage Systems (ADS). The pipe must be strong enough to maintain adequate capacity after the landfill has been constructed to the final permitted elevations without excessive deflection.

The Iowa Formula for deflection is:

$$\Delta X = DL\,(kWr^3) / [\,(D^3\,(PS) / 53.77) + 0.061\,E'r^3]$$

where:

ΔX	=	vertical deflection in inches
DL	=	deflection lag factor (usually 1.5)
k	=	bedding constant (usually 0.1)
W	=	load per unit weight of pipe in pounds per linear inch
r	=	pipe radius in inches
D	=	pipe diameter in inches
PS	=	specified pipe stiffness in pounds per inch of length per inch of deflection
E'	=	modulus of soil reaction in pounds per square inch

(From James Goddard, Structural Design of Plastic Pipes, March 1983, Columbus, Ohio, Advanced Drainage Systems, Inc., March, 1983, page 20)

The deflection for six, eight, twelve, and 24-inch diameter ADS N-12 Landfill Grade HDPE pipe is calculated as follows:

Assume 150 feet of refuse weighing 65 pounds per cubic foot and three feet of cover soil weighing 100 pounds per cubic foot.

W, the load per linear inch,

$$= [(150\text{ft} \times 65\text{ lb/cf} + 3\text{ft} \times 100\text{ lb/cf}) \times D] / 144\text{ si/sf}$$

$$= 69.792 \times D$$

D = 6 inch	W = 418.75 lb/in
D = 8 inch	W = 558.33 lb/in
D = 12 inch	W = 837.50 lb/in

PS, pipe stiffness,

6 inch pipe	50 psi
8 inch pipe	50 psi
12 inch pipe	45 psi
24 inch pipe	34 psi

E', modulus of soil reaction = 3000 psi for rock backfill

For six inch pipe:

ΔX = (1.5)(0.1)(418.75)(27) / [(216)(50)/53.77 + (0.061)(3000)(27)]

= 0.3298 inches

0.3298 / 6 = 5.5 % deflection

For eight inch pipe:

ΔX = (1.5)(0.1)(558.33)(64) / [(512)(50)/53.77 + (0.061)(3000)(64)]

= 0.4398 inches

0.4398 / 8 = 5.5 % deflection

For twelve inch pipe:

ΔX = (1.5)(0.1)(837.50)(216) / [(1728)(45)/53.77 + (0.061)(3000)(216)]

= 0.6622 inches

0.6622 / 12 = 5.5 % deflection

For Side Slope Risers

Depth of solid waste = 50 feet

W = [(50ft X 65 lb/cf + 3ft X 100 lb/cf) X D] / 144 si/sf

= 24.65 D

D = 12 inch	W = 295.83 lb/in
D = 24 inch	W = 591.67 lb/in

E' = 1000 psi for sand drainage layer

For twelve inch Side Slope Riser:

$$\Delta X = (1.5)(0.1)(295.83)(216) \;/\; [(1728)(45)/53.77 + (0.061)(1000)(216)]$$

$$= 0.6555 \text{ inches}$$

0.6555 / 12 = 5.5 % deflection

For twenty-four inch Side Slope Riser:

$$\Delta X = (1.5)(0.1)(591.67)(1728) \;/\; [(13{,}824)(34)/53.77 + (0.061)(1000)(1728)]$$

$$= 1.3435 \text{ inches}$$

1.3435 / 24 = 5.6 % deflection

The calclated deflectons are within the manufacturer's recommendations and are acceptable for long term performance in the proposed application.