R E P O R T

CELL 2 LANDFILL LINER SYSTEM CONSTRUCTION PERMIT APPLICATION DOCUMENTS

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1.0 INTRODUCTION

International Paper (IP) operates an Industrial Solid Waste Landfill adjacent to its Kraft pulp and paper mill in Riegelwood, North Carolina. Process residuals, including dewatered wastewater treatment solids, boiler ash, and other solids are managed in the industrial process landfill (NCDENR Facility Permit No.:24-02). The landfill facility is located adjacent to the mill in Columbus County (34°21'06"N x 78°12'21"W).

1.1 Background

IP-Riegelwood submitted a Lateral Expansion Permit Application to NCDENR in February 2002, proposing to construct their North Bay landfill site in 2 similarly sized cells of approximately 13.5 acres each. Cell 1 was 13.6 acres in size, had a design capacity of 711,700 cubic yards (cyds), and was expected to last 4.1 years. Cell 1 construction was completed in September 2003, and active use of the cell began in early 2005. Figure 1-1 shows the North Bay landfill cells. The February 2002 Lateral Expansion Permit Application showed future cell development and facility closure information for conceptual purposes, and was not approved for construction in the NCDENR December 2002 Permit to Construct.

As IP-Riegelwood began to schedule Cell 2 construction they learned that Holtrachem Corporation (now owned by Honeywell Corporation) had discharged Polychlorinated biphenyls (PCB's) near the northwest corner of Cell 2 during their use of the mill's wastewater treatment system. While PCB-impacted sediment removal work was being planned, permitted and executed, IP-Riegelwood extended their landfill via an expansion to the south side of Cell 1. This extension was completed in 2008 and 2009 to extend Cell's 1 fillable life. Honeywell completed removal of the PCB-impacted sediments in January 2009, allowing IP-Riegelwood to resume their Cell 2 landfill expansion planning work.

1.2 Purpose

URS was retained by IP to update the Cell 2 development plans following Honeywell's sediment removal activities. Because of the extensive excavation work completed by Honeywell, IP met with NCDENR in September 2008 to explain the impact of Honeywell's work on their property and schedule for Cell 2 construction. IP submitted a subgrade restoration plan which NCDENR Division of Waste Management approved January 13, 2009. IP-Riegelwood has yet to begin that work, and is herein submitting complete Construction Permit Application plans for the remaining Cell 2 landfill features. Updated Construction Specifications and a Quality Assurance Manual are included as part of this package.

The various NCDENR approvals for the activities that have taken place for Cell 1 and 2 are provided in Appendix A. Cell 2 is designed using the same principles as were used for the Cell 1 design. The intent of this submittal is not to repeat already approved design concepts and supporting calculations, but rather to present modifications to the Cell 1 concepts that will be used to contruct Cell 2, and to provide the supporting calculations. Modifications are presented in the following sections.

Figure 1-1. Riegelwood North Bay Landfill Cells – 2004 Aerial Photograph

2.0 DESIGN MODIFICATIONS

Modifications to the February 2002 Lateral Landfill Expansion Permit application plans for Cell 1 and 2 are detailed in the following report sections. Modifications were made to account for IP infrastructure and to simplify Cell 2 construction. These modifications are reflected on the attached plans and are described below.

2.1 Cell 2 Footprint Configuration

The configuration of Cell 2 was adjusted slightly from the 2002 layout due to several design modifications. The waste footprint was reduced and the northwest perimeter berm top elevation was lowered. These modifications are described below.

2.1.1 Waste Footprint

The waste footprint for Cell 2 was reduced as shown on attached Existing Conditions, Sheet 3. The northeast corner of the cell was moved southward to avoid the oxygen tank and underground piping in that area. The footprint was changed slightly on the west side as a result of lowering the perimeter berm surface elevation.

2.1.2 Perimeter Berm Lowering

The 2002 Permit Application drawings had the north perimeter berm transitioning from elevation +47 to +37 on the northwest side of Cell 2. As part of the operational evaluation, URS shifted this perimeter berm surface transition to the southwest to allow for addition of an access ramp for trucks. The revised perimeter berm and access ramp are shown on the attached Subgrade Grading Plan, Sheet 5. This modification leads to a reduction in the originally permitted air space capacity. Table 2-1 summarizes the landfill acreage and capacity as originally permitted, and after incorporating design modifications presented herein.

Table 2-1

Summary of Originally Permitted Landfill Acreage and Capacity

	(1)		(1)	(2)	
	2002 PTC	2002 Lined	2009 Permit	Cell 2 PTC	Cell 2 PTC
Description	**Gross Vol. (cyds)**	**Acreage**	**Gross Vol. (cyds)**	**Cap (cyds)**	**Acreage**
Cell 1	749,000	13.80	749,000	872,460	13.89
Cell 1 Extension	0	NA	147,000	147,000	2.25
Cell 2	1,870,000	13.70	1,870,000	1,072,112	12.65
Cumulative	**2,619,000**	**27.50**	**2,766,000**	**2,091,572**	**28.79**

Notes: (1) Gross Volume includes final cover + residual volume.
(2) Cell 2 PTC Capacity is Residual Volume only.

2.1.3 Cell Floor

Cell 2 floor elevations have been modified as a result of revisions to the waste footprint discussed above. Cell floor slopes (2% parallel to leachate flow direction and 0.5% beneath leachate collection and removal pipes) have been maintained and replicate those already approved in landfill Cell 1. The Feb. 2002 permit plans showed the cell floor elevation ranging between elevation 15 and 26. The attached Underdrain Layout and Piping, Sheet 4, shows the cell floor elevation ranging between elevation 16 and 23.

2.2 Underdrain System

Cell 2's underdrain system was designed to replicate the Cell 1 underdrain system since it's already operating effectively. In addition to adjusting the underdrain pipe layout to account for adjustments to the cell footprint, URS has increased the separation distance between the underdrain pipe invert and liner from 4.0 feet minimum in the approved 2002 Permit Drawings, to between 4.4-6.0 feet for Cell 2. The Cell 1 underdrain manhole will be decommissioned, with the piping extended into the Cell 2 system. A new manhole will be constructed outside the northeast corner of Cell 2 that will receive the combined Cell 1 and 2 underdrain system water. This new manhole will contain a duplex pumping system to increase reliability and facilitate maintenance. This system was designed to maintain the ability to gravity drain the entire underdrain system to the river should water quality permit and the need arise.

2.3 Liner System Modifications

IP-Riegelwood is hereby requesting modification of the liner system for Cell 2 as follows.

2.3.1 Removal of Secondary Liner System.

IP-Riegelwood's Cell 1 Operating Permit includes an Action Leakage Rate (ALR) of 500 gal/acre/day for the primary liner system. This ALR has never been triggered. IP Riegelwood's electronic data acquisition system shows 16,876.5 gallons of water were pumped from the secondary system between March 2, 2005 and May 22, 2009. Using 13.89 acres as Cell 1's lined area, and time interval of 1544 days, the calculated average flow rate to the secondary collection system is 0.79 gal/acre/day. The flow rate into the secondary collection system is therefore very small.

NCDENR already approved removal of the secondary liner, the secondary collection system, and the geosynthetic clay liner portion of the primary liner in the Cell 1 Extension Construction Permit. The minor modification approval granting removal of the secondary liner system by NCDENR for the Cell 1 Extension is provided in Appendix A.

The underdrain system is designed to create an inward groundwater flow gradient from all sides of the Cell 2 landfill and will effectively provide for removal of water that penetrates the primary liner, returning it to the wastewater treatment system.

2.3.2 Primary Liner Upgrade

URS recommends increasing the primary geomembrane liner thickness from 40 to 60 mil textured HDPE. The geosynthetic clay liner component of the Primary liner system will be maintained as approved in the 2002 Construction Permit.

2.4 Leachate Collection System Enhancements

The filter media configuration in the leachate sump area was modified. The new design prescribes a berm of #57 stone, 15 feet in height, over the leachate sump area, as shown on attached Leachate Management Details, Sheet 14. This modification will increase the drainage surface area for the sump, increases storage capacity for large storm events, and should serve to enhance filtering effectiveness.

Appendix B provides calculations demonstrating filtering compatibility of Gooseneck borrow material, used as the Protective Cover in Cell 1 with the North Carolina Department of Transportation Table 1005-1 No. 78 and No. 57 aggregates, along with ½-inch diameter hole size proposed for the leachate collection and removal pipes. The protective cover soil will be extended over all No. 78 filtering aggregate over the leachate collection pipes for Cell 2 in order to improve the filtering efficiency of the collection system.

3.0 PUMP SELECTION

Both the leachate collection and removal system and the underdrain pump systems are designed as duplex pumping systems. The Cell 1 and Cell 2 leachate and groundwater pumping and conveyance systems were modeled using the SewerGEMS Version 8 computer program to confirm pump selection. These systems share the existing forcemain which currently conveys leachate and groundwater from the Cell 1 systems to the clarifier. Model output is provided in Appendix C. The system was modeled for six different operating scenarios:

- All pumps at all three locations on simultaneously;
- One pump at each of the 3 locations on simultaneously;
- One proposed leachate and underdrain pump on simultaneously;
- One proposed leachate pump on only;
- One proposed underdrain pump on only; and
- One existing leachate pump on only.

Performance curves for the pumps selected using these modeling results, along with the plotted range in discharge rates, are provided in Appendix C. The modeling effort was completed after flow rates were field-measured at the forcemain discharge for the existing Cell 1 primary leachate pumps and the underdrain pump.

3.1 Underdrain System

In order to size the Cell 2 underdrain pumps URS modeled the groundwater collection system. We calibrated the model using field-measured discharge rates from Cell 1, combined with IP-Riegelwood's electronic data acquisition records for the Cell 1 underdrain pump run time. Appendix D includes calculations that estimate the increase in groundwater flow from Cell 2 is approximately 6 to 7 gal/min. The modeled Cell 1 collection system flow rate was approximately 19 gal/min, compared with a field measured average flow rate of 12 gal/min URS field measured the discharge rate for the Cell 1 underdrain system pump at 60 gpm. Therefore, if a pump capable of producing 60 gpm is installed in Cell 2 it would be sufficient to handle the expected long-term flow rate. The pump calculations are provided in Appendix C. Stormwater generated within Cell 2 prior to liner installation was not considered in sizing the underdrain system pumps.

3.2 Leachate System

The Cell 2 leachate collection and removal system was designed to replicate the Cell 1 leachate collection and removal system since Cell 1's system has been operating efficiently. The existing pumps in Cell 1 are rated at 75 gpm. Field measurements confirmed discharge rates of 85 gpm for Primary leachate pump P-1, and 50 gpm for P-2. P-2's lower pumping rate was reasonable based on an observed 90° bend in the discharge hose. Based on existing Cell 1 system performance, and the nearly identical size of Cells 1 and 2, the pumps proposed for Cell 2 are each rated for 85 gpm. The pump calculations are provided in Appendix C.

4.0 STABILITY ANALYSIS

While preparing the Cell 2 Liner System Permit drawings we modified the final contour drawings to account for the adjusted Cell 2 footprint, and included the Cell 1 Extension. The resulting net residual capacity and lined landfill areas were provided in Table 2-1. Final landfill slopes (4H:1V) and the peak elevation are similar to those shown on the 2002 Permit Drawings. The Final Cover Grading Plan (Sheet 8) shows the proposed final cover contours. Stormwater control features are also provided, and supported by calculations in Appendix F. The Operational Plan (Sheet 9) shows Cell 1 final contours just prior to transitioning into Cell 2. The operating plan contemplates a 3H:1V active filling face as the steepest safe operating slope.

The following stability calculations were performed to confirm the adequacy of these design slopes for Cell 2:

- Final Cover Veneer Stability; and
- Active Operating Slope Stability.

The outcome of these calculations is described below.

4.1 Final Cover Stability

URS has proposed a final cover cross-section that includes a geomembrane overlain by a geocomposite drainage layer. Veneer Stability calculations were performed to determine the minimum interface friction angle to maintain stability using these final cover components. In order to achieve a static stability safety factor of 1.5 or greater, an interface friction angle of at least 21 degrees calculates required (see Appendix E). To achieve a seismic stability safety factor greater than 1.0, the minimum interface friction angle for the cover components must be greater than 21 degrees.

4.2 Operational Slope Stability

Since intermediate operating landfill slopes are specified at 3H:1V, these slopes represent the critical condition from a slope stability perspective. These analyses considered both static and seismic conditions. Piezometric (water table) conditions considered the underdrain system performance, effectiveness of the leachate collection system at the landfill base, and the prospect for perched leachate levels within the landfilled residuals.

4.2.1 Static Analysis

Table 4-1 below summarizes results of the various stability analysis trials. Undrained analyses (total stress strength parameters) were used for the stability evaluation. Soil, landfilled residuals, and geosynthetic interface strength parameters used for the analysis are provided in Appendix E.

Table 4-1

Intermediate Slope Stability Analysis Result Summary
IP-Riegelwood Landfill Cell 2

Trial	Description	Safety Factor
Trial 1	Int. Slope 3:1 – high water - static	1.5
Trial 2	Int. Slope 3:1 – low water - static	1.7
Trial 3	Int. Slope 3:1 – high water- seismic	1.2
Trial 4	Int. Slope 3:1 – low water - seismic	1.4

Computer models were prepared for analysis using the selected cross-section. The stability analyses were conducted using the computer program PCSTABL6. This program calculates safety factors for assigned material properties, sliding surface geometry, and groundwater conditions. For these analyses randomly shaped (Modified Janbu method) sliding surfaces were evaluated to determine the most critical condition (lowest safety factor). The landfill slope was modeled using a two-dimensional analysis. The outputs for the different trials are provided in Appendix E. A safety factor of 1.5 for static conditions and 1.0 for seismic analyses of the intermediate landfill slope was considered acceptable.

4.2.2 Seismic Analysis

The IP-Riegelwood landfill lies inside a seismic impact zone ($a_{max} \geq 0.10$ g according to USGS (Frankel, et. al.)). The peak bedrock acceleration for the project site was obtained from the 2008 USGS' Earthquake Hazards Program custom mapping and analysis tools - Latitude Longitude interpolation mapping feature. This USGS interpolation program reports that a Peak Ground Acceleration (PGA) of 0.125g would have a 2% probability of being exceeded in a 50 year time span. This PGA also has a 90% probability of not being exceeded in 250 years, which corresponds to an earthquake return period of once in 2400 years.

Normal practice interprets the PGA as the horizontal acceleration at a hypothetical bedrock outcrop at a project site (EPA/600/R-95/051). The PGA is normally attenuated as the shear wave propagates through soil overlying the bedrock according to a relationship presented by Seed and Idriss (1982) and Idriss (1990). The amount of amplification/attenuation was estimated using empirical relationships suggested by Idriss (1990), described in EPA/600/R-95/051. This procedure relates the level of motion in bedrock to motion at the surface of a soil deposit based on the soil type through which the seismic motion propagates.

The Idriss (1990) relationship suggests the 0.125 g rock accelerations would attenuate/amplify to 0.125g by the time it reached the natural ground surface beneath the landfill.

Procedures recommended by Bray et al. (1995) suggest motion would then attenuate as it passes from the landfill bottom into, and through, the landfilled residuals. The amount of attenuation is estimated at 50%, resulting in an estimated ground motion of 0.07g within the landfill mass.

Results of the seismic analyses, summarized in Table 4-1, suggest the operational slopes should remain stable under both static and seismic conditions.

5.0 FINAL COVER AND STORMWATER MANAGEMENT

The new design for the final cover shows the high point at elevation 132 (see attached Final Cover Grading Plan, Sheet 8). The Feb. 2002 permit plans show a high point of 136. Final cover slopes have been adjusted to be compatible with the slight change in the waste footprint and lowering of the perimeter berm surface elevation.

The Final Cover Grading Plan presents a conceptual design and we expect a more detailed and specific closure plan will be submitted as the site nears final capacity. This will allow for IP to take advantage of current technology and methods that are being practiced at that time.

5.1 Drainage Channels and Swales

Along with modifications to the final cover, URS has included stormwater management details for the final cover, including perimeter drainage channels, runoff diversion swales, and downdrain pipe systems. The calculations were based on a 25 year 24 hour storm event. The calculations are provided in Appendix F. The drainage features were designed at a depth of 2 feet or more, and should therefore provide more then 0.5 feet of freeboard. The maximum allowable velocity for the type of vegetation that will line the drainage features is 5.0 ft/sec. The maximum calculated velocity is 4.69 ft/sec, therefore grass vegetation would be adequate.

There will be three down pipes strategically located on the final cover. The drawing in Appendix F outlines the drainage areas and shows the discharge flow for each area. The required down pipe size is shown on the Final Cover Grading Plan, Sheet 8. The down pipe calculations are provided in Appendix F. The down pipe calculations indicated a discharge velocity of greater then 5 ft/sec, therefore an energy dissipator surrounded by Class B rip rap will be installed at the end of the down pipes to minimize the impact to the surrounding area.