A-1

FIELD CHANGE ORDERS

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 001

Project Number: 742577	Date: 4/11/03
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Extraction Trench

a. Trench Alignment: To allow two-way traffic on RHR and create more space for construction equipment along the trench, the alignment will be moved east approximately 20 feet within South Pond. Refer to Drawing C-9 for the original alignment and the attached Extraction Trench Work Plan for the proposed alignment.

b. Backfill: Replace the NYSDOT #1 Stone trench backfill with a NYSDOT #1A Stone. A small gradation is required to support the trench and filter silts from entering the sumps. The material must have a minimum permeability of 1 X 10-2 cm/s. Refer to the attached sieve and permeability testing results.

c. Work Platform Top Width: Increase from 20 feet to 40 feet. This width increase will allow a safer platform for heavy equipment (e.g., crane, excavator, dump trucks) and personnel during the extraction trench installation.

d. Work Platform Side Slope: Increase the slope from 1V:3H to 1V:2H to minimize encroachment into South Pond. Refer to Shaw's attached slope stability analysis which demonstrates an acceptable factor of safety for the proposed slope.

e. Trench Collection Pipe: Eliminate the perforated 8 inch collection pipe and associated cleanouts. The collection pipe is redundant given the extraction trench backfill (NYSDOT #1A) permeability. Elimination of the collection pipe, cleanouts, and connections to the sumps will improve constructability of the extraction trench. Refer to Drawings C-9 and C-102. To address USEPA concerns that the 3 sumps is not adequate to pump the required flow and that additional piezometers beyond the designed 4 piezometers should be placed within the trench for monitoring purposes, (2) 8 inch stainless steel screens will be installed within the trench. The screens will serve as piezometers, and if additional pumping capacity is needed, the screens could serve as sumps for submersible pumps. The pumps and associated piping could easily be installed if it is deemed necessary.

f. Collection Sumps: Reduce the diameter of the collection sump from 3 feet to 2 feet. The reduced sump is still adequate for the sump pumps, instruments, and supports but will be much more easily constructed within the 3 foot wide trench. Refer to Drawing C-102. Also, eliminate the filter sock around the sumps. A filter sock in this application has a high risk of blinding from silt. If the filter sock is blinded, the removal of filter sock would require removing the entire sump. The sumps will be maintained such that any silt or backfill stone entering the sumps through the 3/8 inch holes will be vacuumed out.

g. Delivery Pipe: Replace Alloy 20 pipe within the extraction sumps with HDPE or PVC Schedule 80 pipe which is used throughout the GWTP. Refer to Drawing C-102.

h. Sand Testing: Requirements for sand testing and desanding within the biopolymer slurry [Section 02250.3.04C4] will be eliminated. There is no sand within the slurry or trench backfill. Sand testing is typical for other slurry applications.

i. Vertical Panel Alignment: The Electronic Verification Device (EVD) seal test for the vertical panels [Section 02406.3.05B] will be replaced with standard industry visual testing. When adjoining panels are installed, their corners should match up. A simple visual check is more reliable than the "light bulb" seam test.

j. Low Point Manhole: Eliminate the low point manhole [Design Drawing C-101 and C-104] along RHRL midway between the GWTP and Sump 1. The current configuration doesn't allow for isolation and cleanout. The groundwater force main is continuous from the GWTP to Sump 1. Also, the manhole could be a potential source of contamination if a leak occurs that overflows the manhole.

k. Trench Cap: Modify the trench cap to consist of separation geotextile and 2 feet of soil cap (common fill). The trench cap will be installed after the trench is backfilled to the top of the work platform. Refer to Drawing C-9 Section A for the original design and the Extraction Trench Work Plan for the proposed trench cap.

Note: Shaw and Parsons have prepared backup information supporting this field change order: (1) a technical memo supporting the collection pipe elimination; and (2) a response letter regarding the extraction trench.

APPROVALS:

Parsons Representative

Name: Matt Millias	
Signature:	Matthew D. Millias
Date:	5/8/03

Shaw Representative

Name: Scott Sutton/John Waechter	Scott Sutton
Signature:	
Date:	5/8/03

USEPA

Name: Young Chang	
Signature:	[signature]
Date:	5/14/03

cc: Joe Bianchi - Amphenol
John Mojka - Honeywell

parsons

From: <y.s.chang@rcn.com>
To: <parsonsrhrl@frontiernet.net>
Cc: <chang.young@epa.gov>
Sent: Wednesday, April 23, 2003 8:08 AM
Subject: field change form #1

Matt:

For the most part EPA has no objections to the Field Change Form #1; we do however request further information regarding item "e"- proposed elimination of the horizontal collection pipe. It is our understanding that these horizontal perforated drainage pipes were to provide a conduit for the flow of contaminated groundwater in the trench to the sumps, without which there is no assurance that the sumps will have hydraulic influence on the groundwater between the sumps. Please provide calculations/further information that demonstrates that the trench will perform as originally designed with the proposed change.

With this change, EPA would also request that piezometers be installed at every 100 feet of the trench to determine if the sumps are impacting the groundwater throughout the entire trench. Prior to the start of the pumps (and post biopolymer removal), a round of GW levels will need to be made to determine the static groundwater level. Theoretically, once the pumps are on, drawdown should occur at all of the piezometers. If any of the piezometers is not showing drawdown, then additional sumps may be necessary.

As for the #1A Stone that been selected for use, note that it did not pass the strict definition of the NYSDOT #1A Stone however it meets the minimum permeability needed and therefore EPA has no objections of its use.

Young

MEMORANDUM

May 7, 2003

To: Matt Millias

From: Steve Rossello

Subject: Honeywell Richardson Hill Road: Trench Pipe

Matt,

I used the RHRL groundwater model to evaluate how the trench would operate without the horizontal drain running between the three sumps. The drain nodes that represented the trench were replaced with three well nodes. The trench backfill was simulated by specifying a hydraulic conductivity of 60 ft/day (2 x 10^{-2} cm/sec) for the nodes that represented the trench. This hydraulic conductivity value is based on the lab results for the 1A stone that is to be used as trench backfill.

The model calculated a drawdown of about 6 feet at the sumps and 2 feet halfway between the sumps, (see Figures 1 and 2). I redid the particle-tracking analysis, which also indicated full capture by the trench. I also ran a simulation with 5 sumps (see Figure 3), which calculated a more "even" distribution of drawdown along the trench, similar to what would be observed with a horizontal drain.

Based on these simulations, the three-sump trench would provide adequate capture, but the 5-sump trench would offer more flexibility in trench operation and a higher safety factor, especially during periods of increased recharge.

I understand that we have surplus 8-inch diameter screen on site. If the two trench piezometers that were to be located half-way between the sumps were to be constructed with the 8-inch diameter screen and riser, they could still be used to measure water levels within the trench. However, if at some time in the future post-construction monitoring shows that additional sumps are needed, the 8-inch diameter piezometers could be converted to sumps by installing pumps and piping. Conservatively assuming that these "contingency sumps" would be used in the future, it would be prudent to also install additional piezometer pairs so that we could monitor water levels half-way between all of the active and contingency sumps.

cc: William Long

Figure 1. MODEL RESULTS FOR 3-SUMP EVALUATION (cross-section across trench)

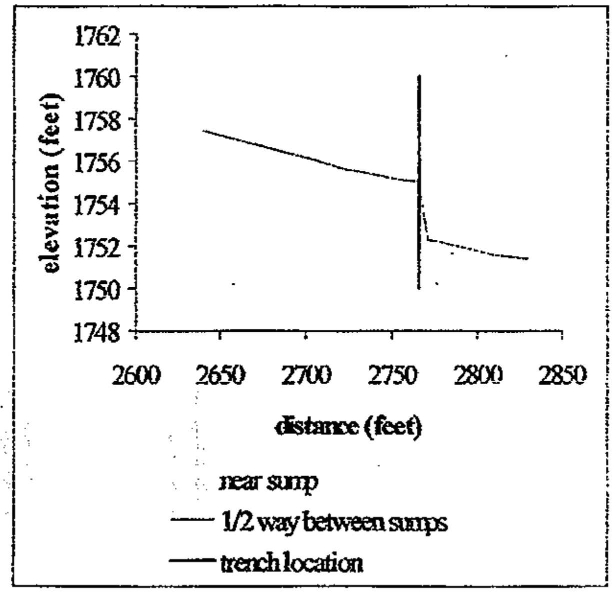


Figure 2. MODEL RESULTS FOR 3-SUMP EVALUATION (profile along trench)

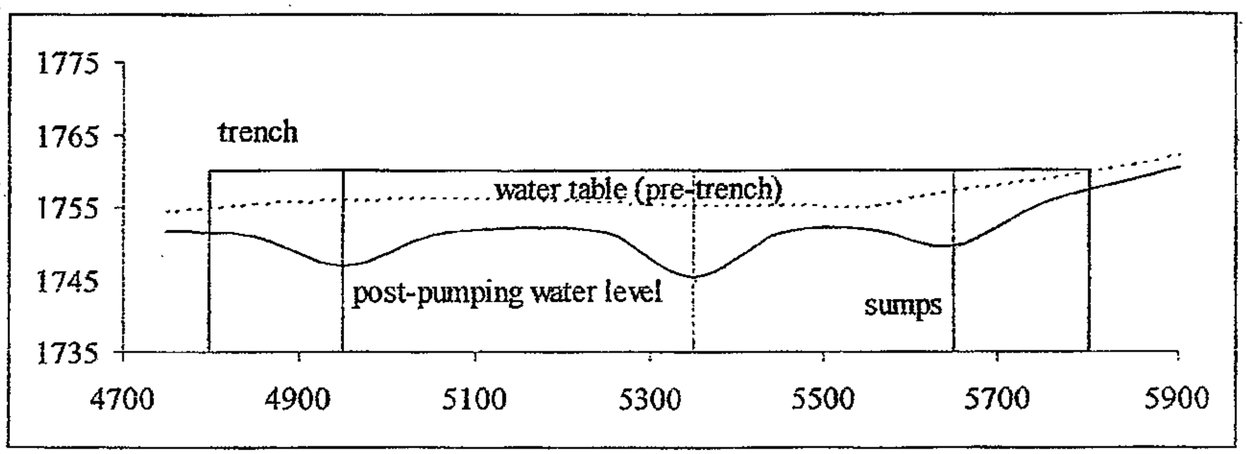
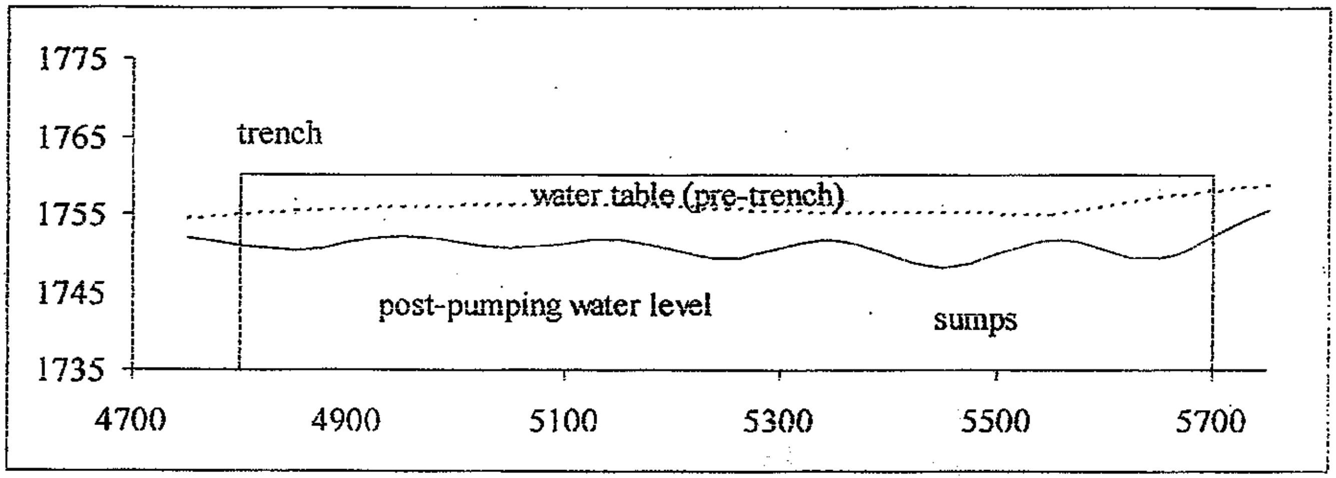


Figure 3. MODEL RESULTS FOR 5-SUMP EVALUATION (profile along trench)



EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 002

Project Number: 742577	Date: 5/08/03
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Well Abandonment

- Per USEPA/EarthTech direction, MW3S, MW3D, MW5D, MW7S, and MW7D are not to be abandoned as part of the well abandonment activities.
- Many of the wells to be abandoned are not constructed as depicted in historical well diagrams. In some cases, well materials, backfill materials, well diameters are incorrect. For example, MW3DD was not grouted between the 12 inch casing and 8 inch casing or between the 8 inch casing and 2 inch casing. When attempting to overdrill the 2 inch casing and grout via tricone methods, sand, plastic bags, wood chips, and other materials were discovered. It was agreed that the driller would alter the method and try a thick slurry mix to force this material out so that the bottom of well (140 ft bgs) could be found and tremied. This example as well as others have caused Parsons, Shaw, EarthTech/USEPA and the driller [Parrott Wolff] to make modifications to Specification Section 02085 in order to complete the work. Also, several wells identified in the design have not been found. In contrast, several wells not located on the drawings have been found. Parsons, Shaw and Parrott Wolff will abandon all wells that will impede the Earthwork with exceptions as noted above.

Landfill

- Drawing C-10 incorrectly identifies CV-4 as CV-1 (there is no CV-1 as part of this design). This change will be made in the as-built drawings. Also, C-6 is not clear where the leachate collection trench is located along the landfill toe. The corresponding C-10 detail, however, clarifies the distances from the road. It should be noted that the southern length follows the landfill access road toward RHR and ends at CV-2. The northern length starts adjacent to RHR and the existing storage shed and continues to CV-4.
- The geomembrane and associated cap along RHR at the southern end should extend to the leachate collection trench based on the distance in the C-10 detail.
- Friction testing between layers was intended for the top cap on steep slopes. Bottom TSCA Cell liners are located on a low slope surface, and therefore, this requirement will be waved for the bottom TSCA Cell liners.
- Soil cuts within the original landfill grading plan (southwest side in area of surface water berm) will be used for TSCA cell berms.
- Soil cuts from the work platform area (former parking area) will be used within the South Pond for creating segregation berms. The remaining soil cuts will then be used for work platform fills and/or landfill fills.
- The TSCA Cell contains 2 sumps: leachate collection sump and leak protection sump. To clarify these sumps, one solid pipe penetrating through the lower berm will connect from the leachate collection perforated pipe into a precast concrete manhole. Leachate will be periodically removed with a submersible pump, drummed and treated at the GWTP. For the leak detection sump, a solid pipe connected to a perforated pipe between the two bottom liners will penetrate the lower berm and enter an outer 24 inch diameter HDPE sump and inner 6 inch diameter HDPE sump. The 6 inch sump will allow easy monitoring for leaks in the TSCA liner.

Forcemain/Delivery System

- Eliminate the cleanouts on Drawing C-9 associated with acid tubing, electrical conduit, and instrumentation conduit. These cleanouts are not appropriate. Cleanouts associated with the groundwater forcemain will be installed as designed at Sump 1, 2, and 3.
- Pullboxes for acid lines will be water-tight precast concrete rather than a poured slab under a separate vault. The acid pullboxes will be water sealed (epoxy coating) in the field.
- The groundwater forcemain will run outside the acid pullboxes. The acid pull boxes are to provide a location for pulling acid tubing through the 4 inch HDPE carrier pipe.

Sediments

- Sediments within South Pond excavated/relocated within the South Pond or Landfill must be covered daily.

E&S

- The piping originally located on the west side of South Pond to control surface water is being moved to the eastern ditch of RHR.

APPROVALS:

Parsons Representative

Name: Matt Millias	
Signature: Matthew D. Millias	
Date: 5/14/03	

Shaw Representative

Name: Scott Sutton/John Waechter	
Signature: Scott Sutton	
Date: 5/14/03	

USEPA

Name: Young Chang	
Signature: [signature]	
Date: 5/14/03	

cc: Joe Bianchi - Amphenol
John Mojka - Honeywell

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 003

Project Number: 742577	Date: 5/21/03
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Use of Cleared Trees and Stumps:
As part of the clearing and grubbing activities for the RHRL site, trees were cut at or above ground surface and staged in the field north of the landfill as well as within the landfill itself. Stumps were excavated and placed in separate piles from the trees on the landfill.

According to the approved final design, it was acceptable to sell the trees to the public for a fair market value. Shaw contacted several potential buyers, but there was no interest due to the relatively low volume of hard woods. At this time, Shaw is interested in chipping the trees and utilizing the wood chips for erosion control measures as follows:

- Erosion control on the landfill slopes;
- Erosion control for disturbed areas; and
- Erosion control for the access roads and parking areas.

Above bulleted items do not include wood chips from the stumps.

APPROVALS:

Parsons Representative

Name: Matt Millias	Date: 5/28/03
Signature: Matthew D. Millias	

Shaw Representative

Name: Scott Sutton/John Waechter	Date: 5/28/03
Signature: [signature]	

USEPA

Name: Young Chang	Date: 5/28/03.
Signature: [signature]	

cc: Joe Bianchi - Amphenol
John Mojka - Honeywell

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 004

Project Number: 742577	Date: 8/14/03
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Extraction Trench Sump Vaults and Forcemain Piping:

- To prevent freezing, water and acid piping must be installed a minimum of 4 ft below ground surface. Currently on Design Drawing C-102, the pipe penetration into sump vault 1, 2, and 3 are shown at 2.5 ft below ground surface. The concrete vaults should be sized to allow pipe penetration of 4 ft bgs or deeper.
- Insulation board must be installed within the sump vaults as well as pullboxes to prevent freeze damage to piping.
- The acid carrier pipe penetration and acid tubing will be installed along the vault floor (near the west wall) to minimize the potential acid leak hazard should a maintenance be conducted within the vault.
- For cleaning purposes, the groundwater force main piping will be 3 inch diameter rather than 2 inch diameter.
- Flush mounted double wye cleanouts (one for installing a water jet hose and one for vacuuming solids/water from the cleaning process) will be installed as part of the 3 inch diameter HDPE groundwater forcemain at Pullbox 2, Pullbox 6, and end of header near Sump 3. Since these wyes will be under pressure and filled with water to ground surface, they will be heat traced to prevent freezing. An additional 1 inch diameter conduit will be installed to run the heat tracing wire from Sump 1 to the four wyes at Pullbox 2 and Pullbox 6; an additional 1 inch diameter conduit will be installed to run the heat tracing wire from Sump 3 to the two wyes at the header end.
- Pullboxes 2-6 will be constructed flush with the road. Clean fill will be used to build the embankment necessary to install the piping and pullboxes. The live optical fiber phone will be avoided.

Landfill Compaction Requirements:

- Within the Technical Specification Section 02228 – Compaction (Minimum Compaction Requirements), the landfill subgrade compaction requirement is 95% minimum compaction (ASTM D1557). Since no subgrade soil layer is specified in this landfill design (there is municipal waste mixed with soil), the compaction requirement will be three passes with vibratory compaction equipment.
- Within the Technical Specification Section 02228 – Compaction (Minimum Compaction Requirements), the relocated soil/sediment materials from South Pond and HHC compaction requirement is 95% minimum compaction (ASTM D1557 - Modified). (This was changed to 85% modified proctor by Parsons' June 6, 2003 response to USEPA earthwork comments.) Based on the discovery of a new site condition, an organic peat material (approximately 50% organic content; approximately 45lb/cf unit weight) within

South Pond between the sediments and clay layers, there is no practical method to meet this compaction requirement. Parsons' Technical Memorandum dated August 7, 2003 details the material properties, compaction requirements, and alternate testing methods including the recommended stabilization/compaction procedures/measurements. The recommended method includes stabilizing the sediment/organic peat with Portland Cement, placing the material in 12 inch lifts with a dozer, allowing the Portland Cement to set up, rolling the material with a smooth drum roller (minimum of three passes) and measuring the deflection between rolled rows. Based on field tests conducted at the site, approximately 30 to 45 percent cement is required for the South Pond peat. The actual amount of cement required for the peat or other excavated sediment may be adjusted depending on actual field conditions. The amount of cement added to each batch of sediment/organic peat will be measured and recorded. **A passing lift is based on a deflection of 3 inches or less.** Shaw conducted a pilot test to evaluate the Portland Cement mixture and set up time. Given the variability of the material, these parameters may be modified as appropriate. However, the deflection measurement will determine whether the lift passes or fails. If the lift fails, Shaw will allow more time for set up and retest. Note that the material will cure over time.

- For pipe bedding material below, adjacent to, and above pipes, the compaction requirements are not applicable. To avoid crushing the pipe, pipe bedding material used to backfill portions of the trench will not be compacted to the specified requirements. Rather, tamping with a jumping jack or small vibratory compactor will be used for compaction.

APPROVALS:

Parsons Representative

Name: Matt Millias	Date: 8/18/03
Signature: Matthew D. Millias	

Shaw Representative

Name: Scott Sutton/John Waechter	Date: 8/19/03
Signature: Scott Sutton	

USEPA

Name: Young Chang	Date:
Signature:	

cc: Joe Bianchi - Amphenol
John Mojka - Honeywell

South Pond between the sediments and clay layers, there is no practical method to meet this compaction requirement. Parsons' Technical Memorandum dated August 7, 2003 details the material properties, compaction requirements, and alternate testing methods including the recommended stabilization/compaction procedures/measurements. The recommended method includes stabilizing the sediment/organic peat with Portland Cement, placing the material in 12 inch lifts with a dozer, allowing the Portland Cement to set up, rolling the material with a smooth drum roller (minimum of three passes) and measuring the deflection between rolled rows. Based on field tests conducted at the site, approximately 30 to 45 percent cement is required for the South Pond peat. The actual amount of cement required for the peat or other excavated sediment may be adjusted depending on actual field conditions. The amount of cement added to each batch of sediment/organic peat will be measured and recorded. **A passing lift is based on a deflection of 3 inches or less.** Shaw conducted a pilot test to evaluate the Portland Cement mixture and set up time. Given the variability of the material, these parameters may be modified as appropriate. However, the deflection measurement will determine whether the lift passes or fails. If the lift fails, Shaw will allow more time for set up and retest. Note that the material will cure over time.

- For pipe bedding material below, adjacent to, and above pipes, the compaction requirements are not applicable. To avoid crushing the pipe, pipe bedding material used to backfill portions of the trench will not be compacted to the specified requirements. Rather, tamping with a jumping jack or small vibratory compactor will be used for compaction.

APPROVALS:

Parsons Representative

Name: Matt Millias	Date: 8/19/03
Signature: Matthew D. Millias	

Shaw Representative

Name: Scott Sutton/John Waechter	Date: 8/19/03
Signature: Scott Sutton	

USEPA

Name: Young Chang	Date: 8/19/03
Signature: [signature]	

cc: Joe Bianchi - Amphenol
John Mojka - Honeywell

POSTED TO PLANS & SPECS
5/11/04

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 005

Project Number: 742577	Date: 8/19/03
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Extraction Trench

Based on the Parsons' August 7, 2003 Technical Memorandum, there is a site condition that would likely impact the extraction trench construction. The site condition includes municipal waste located beneath RHR (approximately 350 ft long; approximately 10-15 ft wide; approximately 7-10 ft deep; stations 2+00 to 5+50). This site condition will require design and construction adjustments as follows:

- Due to structural stability concerns of the extraction trench and road in this area, the municipal waste will be removed and replaced with clean backfill. Excavation of municipal waste will be conducted based on visual confirmation. Municipal waste, soil, and asphalt pavement excavated from the northern extent through the known oily section below the Waste Oil Pit will be placed within the TSCA Cell. Note that the previous test pit in this area had a PCB detection of 64 ppm which is between the applicable PCB range of 50-500ppm. At a point where oily stained soil is not visually observed, a soil sample will be taken to verify that PCB concentrations are less than 50 ppm. If that confirmation is made, the remaining municipal waste, soil, and asphalt will be placed within the general landfill.
- Clean backfill previously approved in Phase 2 Submittal #66 will be used to replace the excavated material. One foot of crushed stone will be placed above the clean backfill as subbase. These materials selected are based on conversations with the highway departments of Masonville and Sidney. During RHR reconstruction, it may be necessary to alter the road surface contours to match the work platform. Following extraction trench construction, RHR will be restored to a condition suitable for public use.
- The Town of Masonville and Sidney highway departments are amenable to closing RHR for 30-60 days to remove the municipal waste and complete the extraction trench. The local schools and emergency officials have been contacted.

APPROVALS:

Parsons Representative

Name: Matt Millias	Date: 8/27/03
Signature: Matthew D. Millias	

Shaw Representative	
Name: Scott Sutton/John Waechter	Date: 8/27/03
Signature: [signature]	

USEPA	
Name: Young Chang	Date: 8/27/03
Signature: [signature]	

cc: Joe Bianchi - Amphenol
John Mojka - Honeywell

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 006A

Project Number: 742577	Date: 7/7/04
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Extraction Trench Changes:

1. The following items from Field Change Form #001 have been revised (revisions **for FCO #006A** shown in bold text):

   a. ~~Trench Alignment: The alignment has been moved approximately 5 feet east (i.e. toward~~ the South Pond from the original design alignment – See Parsons Memo dated 2/13/04 **(Rev#2 -7/7/04)**). (Note that a distance of 20 feet was used in Field Change Form #001.)

   e. Trench Collection Pipe: Eliminate the perforated 8-inch collection pipe and associated cleanouts. The collection pipe is redundant given the extraction trench backfill (NYSDOT #1A) permeability. Elimination of the collection pipe, cleanouts, and connections to the sumps will improve constructability of the extraction trench. Refer to Drawings C-9 and C-102. To address USEPA concerns that the 3 sumps are not adequate to pump the required flow and that additional piezometers beyond the designed 4 piezometers should be placed within the trench for monitoring purposes, four (4) 8-inch stainless steel screens will be installed within the trench. The screens will serve as piezometers, and if additional pumping capacity is needed, the screens could serve as sumps for submersible pumps. The pumps and associated piping could easily be installed if it is deemed necessary. (Note that only two (2) 8-inch stainless steel screens were included in Field Change Form #001.)

2. The following items from Field Change Form #001 remain unchanged:

   b. Backfill
   c. Work Platform Top Width
   d. Work Platform Side Slope
   f. Collection Sumps
   g. Delivery Pipe
   f. Sand Testing
   g. Vertical Panel Alignment
   h. Low Point Manhole
   i. Trench Cap

3. The following item from Extra Work Order #004 remains unchanged:

- Provide an I-beam and vibratory hammer to dislodge/break boulders encountered during trench excavation.

4. The following additional changes are made:

   a. **Revised Drawing C-9, Rev 2** is issued incorporating all of the changes included in this Field Change Form. **(Rev 2 clarifies the depth of the trench and wells.)**
   b. Parsons technical evaluation of the revised trench alignment and panels lengths (**RHRL Alignment Eval 3 Rev2A.doc**, dated 2/13/04, **(Rev#2 -7/7/04)** is included as part of this Field Change Form.
   c. Move culvert CV-2 to the south to eliminate crossing the extraction trench.
   d. Delete Piezometer Pair Schedule on Drawing C-103. (Now shown on **C-9, Rev2.**)

APPROVALS:

Parsons Representative

Name: William J. Long	Date: 7/7/04
Signature: [signature] FOR W. LONG	

Shaw Representative

Name: John Waechter	Date: 7/7/04
Signature: [signature]	

USEPA

Name: Young Chang	Date: 7/7/04.
Signature: [signature]	

cc: Joe Bianchi - Amphenol
Frank Leming - Honeywell

PARSONS

Interoffice Correspondence

To: Bill Long

Date: 2/13/2004 **(Rev #2 - 7/7/04)**

From: Steve Rossello

Subject: RHRL Trench Evaluation

An evaluation was conducted to identify the effects, if any, of installing **the trench and barrier to the depth of** the HDPE barrier wall panels as fabricated and of relocating the RHRL trench and barrier five feet to the east (i.e. towards the South Pond from the original design alignment) on the trench discharge rate. The evaluation indicates that neither installing **the trench and barrier to the depth of** the HDPE barrier wall panels as fabricated nor moving the alignment five feet east would significantly affect the trench discharge rate. In order to ensure capture of the PCB-contaminated groundwater, **both** the trench and barrier need to extend vertically, at a minimum, to the top of the dense till. (Borings downgradient of the waste oil pit (PSB-3 and PSB-4) had oil contamination in the soils extending vertically downward to approximately the top of the till.) Based on the existing borings and the site topography, five feet was determined to be the distance the trench could be moved and still ensure the panels extend to the top of the dense till. Additional borings to determine the depth of the dense till are required if movement greater than five feet is proposed by the trench contractor; however, additional movement of the trench may not be allowable. Details of the evaluation follow.

As documented in the Pre-Design Investigation Technical Memorandum for the Richardson Hill Road Landfill, March 2000, (Revised February 2002), a MODFLOW model was used to calculate an approximate 30 gpm average and 80 gpm maximum groundwater flow rate into the trench. **Approximately 2 gpm would flow from bedrock to the bottom of the trench.** The model also indicated that the difference in flow of groundwater into the trench between the assumption of no open area and a 10% open area at the bottom of the barrier was less than 1 gpm. The conservative 10% value was based on the assumption that the downgradient hydraulic barrier would not be sealed to the bedrock and there would be an open area between the bottom of the **trench/barrier wall panels** and the top of bedrock. That open area would allow the leakage of groundwater under the barrier from the downgradient side of the trench. With no barrier wall, inflow from the trench was calculated to increase by 10 gpm (30 to 40 gpm average and 80 to 90 gpm maximum). The results are consistent with the low permeability of the overburden and a strong hydraulic gradient toward the South Pond.

During subsurface investigations conducted after development of the original model, a dense till layer was found to comprise the lower portion of the overburden. Furthermore, based on boring information in borings PSB-6 and PSB-24, the till and bedrock were found to slope to the east at an approximate 1:3 vertical to horizontal slope. While the hydraulic conductivity of the dense till was not measured, based on the greater density it is expected that the dense till would be at least one to two orders of magnitude lower than the overburden.

The top of dense till, projected to the trench alignment, was plotted on a profile of the trench *(GROUNDWATER EXTRACTION TRENCH PLAN/PROFILE, DRAWING C-09)*. The top of dense till was identified by blow counts (standard penetration test) values of approximately 50/foot or greater. The HDPE barrier wall panels, as fabricated, were also plotted on the profile. Based on the plot, the **trench bottom**/panels would be keyed a minimum of 4 feet and a maximum of 11 feet into the dense till.

If the panels were keyed into the dense till, there would be almost no open area at the bottom of the **trench**/panels. Because the panels would not be sealed against the dense till, there might be some slight flow of groundwater into the trench underneath the panels from the downgradient side of the trench. However, because the area of flow would be much smaller than the 10% open area assumed in the original evaluations, the flow would be insignificant.

If the trench alignment were moved 5 feet east, the depth of the "key" would be reduced by approximately 2 feet due to the slope of the till and bedrock, leaving a 2 to 9 foot key. Flow paths around the bottom of the **trench**/panels would be slightly shorter than in the original trench location, but the open area between the bottom of the **trench**/panels and the till would still be much less than 10%. Therefore, additional groundwater flow into the trench would not be significant.

To achieve the extraction of groundwater from the shallow bedrock as required by the Record of Decision, several options were evaluated:

1. **Excavate the trench to bedrock, backfill the over-excavated portion of the trench up to the bottom of the HDPE barrier wall panels, and install the panels as fabricated. This option was eliminated due to constructability concerns. Specifically, the backfill would slough into the area of the adjacent panel preventing installation of the next panel or disturbance of the previously placed panel. Additionally, too long a portion of trench would need to be open at a given time resulting in potential trench stability problems and equipment access.**

2. **Excavate portions of the trench to bedrock, backfill the over-excavated portions of the trench up to the bottom of the HDPE barrier wall panels, and install the HDPE barrier wall panels as fabricated. The portions of the trench excavated to bedrock would be limited to three adjacent panels of approximately the same length, which occurs in several locations along the trench. This option limits the potential for backfill sloughing into the area of the adjacent panel; however, too long a portion of trench would likely need to be open at a given time resulting in potential trench stability problems and equipment access. Additionally, the quantity of water from**

each deeper trench section was estimated to be approximately equal to the quantity of water from a single well extended into bedrock (See Option 3 and Footnote 4). Therefore, this option was eliminated.

3. **Extend the six proposed trench monitoring wells located within the trench so that they are screened in the upper 15 feet of bedrock, case the wells through the till and install a second screen in the trench backfill. The wells would be installed after the trench is completed. Calculations using the Theis equation indicate that each well would extract approximately 0.2 to 0.4 gpm of groundwater from bedrock for a total of 1.2 to 2.4 gpm, approximately equal to the rate originally calculated for the trench. Because the wells would be hydraulically connected to the trench, pumps would not be required in the wells and the wells could still be used for their original purpose of monitoring water levels within the trench. The advantage of wells over extending the trench depth is the greater yield from bedrock than would be obtained with Option 2. Furthermore, the potential for short-circuiting under the trench through any sand layers located within the till would be eliminated.**

The proposed trench changes will still meet the remedial intent of the Record of Decision.

Footnotes:

(1) The HDPE panels are too short to reach bedrock as a result of design changes proposed by the PRPs during the 2003 construction season. Specifically, Field Change Form #001 reduced the platform elevation by 5 feet and moved the trench 20 feet east from the original design alignment. The HDPE panels were manufactured based on Field Change Form #001. Subsequent design changes documented in Field Change Form #006 returned the platform to its original elevation and the trench alignment to 5 feet east of the original design alignment. Because of these changes, the **trench and** HDPE panels will not reach bedrock as originally designed.

(2) A seismic refraction study to determine the top of bedrock, as suggested by TAMS/EarthTech, is not required as the **trench and** HDPE panels will not reach bedrock based on the borings.

(3) The minor additional groundwater flow into the extraction trench as a result of the **trench and** HDPE panels not reaching bedrock will not affect operation of the groundwater treatment plant (GWTP). The GWTP has a total treatment capacity of 100 gpm. The expected average flow is 40 gpm (30 from the trench and 10 from the North Area wells) and the expected peak flow is 90 gpm (80 from the trench and 10 from the North Area wells).

(4) Using the assumptions that 40 to 50 feet of trench would be in contact with bedrock, and that one foot of silty sand would remain at the bottom of the trench, it was calculated that each section would extract approximately 0.2 gpm from bedrock.

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 007

Project Number: 742577	Date: 8/30/04
Construction Manager: Parsons	
Remedial Action Contractor: Shaw	

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Landfill Grading Plan:

a. Adjust the subgrade elevations to provide additional waste volume without increasing the cap footprint. The final subgrade elevations will depend on the actual quantity of waste excavated and will be adjusted in the field as waste placement progresses. Additional grade adjustments, if needed, will be made at the top of the landfill above the 1820 contour.

b. **Revised Drawing C-6, Rev 2A** is issued incorporating the changes included in this Field Change Form.

APPROVALS:

Parsons Representative

Name: William J. Long	Date: 9/1/04
Signature: [signature]	

Shaw Representative

Name: John Waechter	Date: 9-1-04
Signature: Jeffrey S Mage for John Waechter	

USEPA

Name: Young Chang	Date: 9/1/04.
Signature: [signature]	

cc: Joe Bianchi - Amphenol
Frank Leming - Honeywell

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 008

Project Number: 742577		Date: November 18, 2004
Construction Manager: Parsons		
Remedial Action Contractor: Shaw Environmental		

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Extraction Trench Repair of Liner Panel 16:

Liner panel # 16 was damaged during construction of the extraction trench on August 7, 2004. It was intended to repair and/or replace this panel once the extraction trench dewatering system became operational. On Monday November 8th an exploratory excavation to assess the damage to panel 16 was conducted. The extensive damage to the panel and differing site conditions prohibited repairs as originally planned. Conditions encountered included unstable sidewalls, loss of trench fill media, and rock outcroppings in the lower portion of the trench. These conditions restricted increasing the size of the excavation to install a shoring system and repair the liner in a safe environment.

Mr. Kristin Alzheimer PE representing Shaw Environmental submitted an alternate liner repair procedure on November9, 2004 (Attached) describing three repair options. Shaw's engineer recommended option three as the most appropriate method to suit the field conditions encountered.

Portions of the procedure called for applying filter fabric over the face of the trench media gravel pack and coating the surface with 2,500 psi concrete followed by a six hour cure and placement of flowable fill in the excavation. This task could not be executed as described. Specifically:

- Concrete is not a coating system that can be applied on the vertical face of a fabric.
- Application of the concrete to the fabric would have required entry into an unsafe excavation.
- The excavation sidewalls were unstable. Time was of the essence to fill the void before additional damage occurred.

Parsons as the Engineer of Record approved sealing off the liner discontinuity with a low strength (2,500 psi) high slump (8-10 inch) concrete mix. As specified in the repair procedure. The density of the concrete mix would not compromise the fill media. Moreover placement of the mix to fill the void in lieu of the flowable fill expedited the repair while achieving the desired impermeable medium.

A summary description of the repair follows:

1. The affected area was excavated to a depth of approximately 19-feet.
2. A six inch diameter HDPE pipe embedded in the trench fill material between panels # 15 and 17 was installed close as possible to the bottom of the excavation for trench continuity.
3. The vertical edges of panels 15 and 17 were straightened to envelope the exposed panel edges within the concrete fill media.
4. A layer of filter fabric was installed from ground surface to the bottom of the trench covering the exposed face of the trench media.
5. A lift of high slump concrete was slowly deposited enveloping the pipe to prevent dislodgement.
6. The excavation was then filled with concrete to within two feet of finish grade.
7. The exposed tops of liner panels 15 and 17 were trimmed.
8. The balance of the excavation was backfilled with compacted clean fill material to finish grade.

APPROVALS:

Parsons Representative

Name: William J. Long	
Signature: [signature] For W. Long	Date: 11/24/04

Shaw Representative

Name: John Waechter	
Signature: [signature]	Date: 11/24/04

USEPA

Name: Young Chang	
Signature: [signature]	Date: 11/30/04

cc: Joe Bianchi - Amphenol
Frank Leming - Honeywell

November 9, 2004

Richardson Hill Road Landfill Superfund Site
211 Richardson Hill Road
Sidney Center, New York 13839

RE: Repair of Vertical HDPE Curtin Wall

Background

Panel 17 has disconnected from Panel 16 and Panel 18 at the preformed joint locations. This disconnection has caused a gap in the barrier.

Options for Repair

Option 1
An open excavation can expose the damaged panel so as to enable a replacement panel to be welded into position.

To expose the in-place HDPE vertical barrier to 24-feet deep will require sloping the sides of the excavation to approximately 1 horizontal to 1 vertical. This will require the top of the excavation to be approximately 52- feet wide. There are obstructions on the down slope side of the excavation within 8-feet (electric power conduits and pressure pipelines). To the upslope side of the excavation is a public road (approximately 15-feet away). These physical constraints disallow the option for an open excavation.

Option 2
Utilize a trench box to maintain the excavation walls in the vertical position enabling access to the damaged vertical HDPE. Since trench boxes require interior bracing, straddling the HDPE wall will require cutting of the existing panels and welding numerous small panels in the damaged area to avoid the trench box bracing. A second alternative is to use a trench box on either side of the panel and minimize the required bracing between the boxes, therefore, minimizing the number of small patches required to repair the damage.

Preliminary excavation at the damaged liner has reveled there is significant rock at approximately 12 feet below grade. Breaking this rock in order to advance the trench boxes posses a danger to the integrity of the remaining in-place vertical HDPE and the in-place power conduits and pressure piping adjacent to the existing liner.

Option 3

Expose the ends of Panel 16 and Panel 18 to the depth of Panel 17's damage. Place a double layer of filter fabric over the existing in-place pea gravel and coat the fabric with a rich concrete mix to clog the porous ability of the fabric. Once the fabric coating has adequately solidified, continue filling the excavation with a flowable fill product that binds the edge of Panel 16 to Panel 18 with a 6-foot wide Portland cement product.

Selected Repair Option
Option 3 provides the best barrier repair considering the physical constraints to the work. Below is a more detailed outline of Option 3 repair methods.

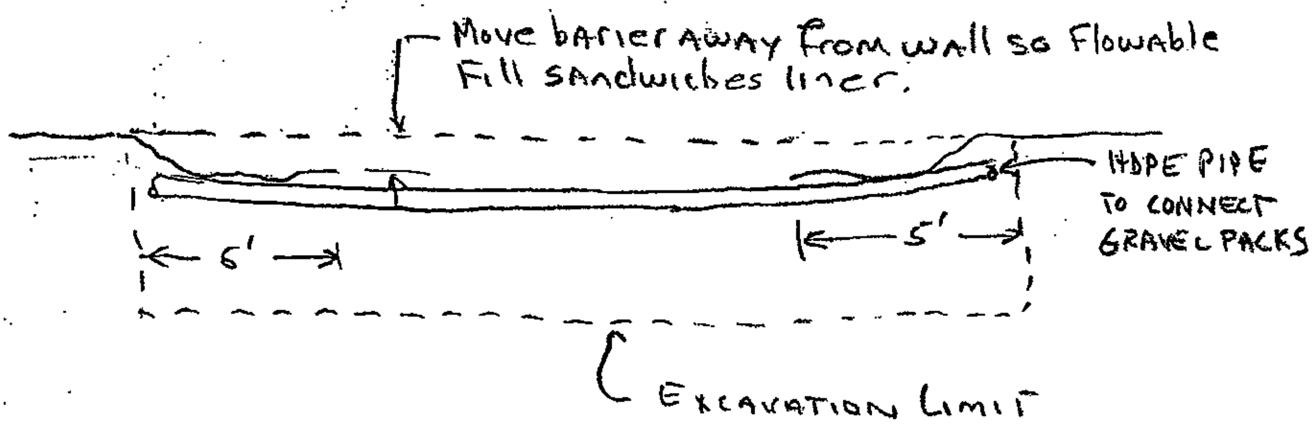
1) From preliminary excavations, it has been reveled that the excavation walls will stand at approximately vertical for a period long enough to perform the Option 3 repair. Excavate to expose approximately 5-feet of Panel 16 and 18 to the depth of panel 17's disconnection.
2) Once Panel 17 is exposed, an HDPE pipe will be laid from Panel 16 gravel pack to Panel 18 Gravel pack at the base of the excavation.
3) Additional pea gravel will be placed over the ends of the pipe terminations within the limits of Panels 16 and 18.
4) A double layer of filter fabric will be draped over the pipe and any exposed gravel pack, extending from ground surface to the depth of excavation and on the base of the excavation between Panel 16 and Panel 18.
5) A 2500 psi concrete will be used to coat the filter fabric in order to eliminate the porous ability of the fabric. This coating will be allowed to cure to a stiff consistency (approximately 6 hours). Attached is Quality Inspection Services, Inc. report of concrete mix design showing laboratory data on the proposed mix design from previous projects. This double layer of filter fabric with a stiff grout coating will minimize the potential of the Flowable Fill from entering the existing gravel pack upslope of Panels 16 and 18.
6) Upon adequate curing of the filter fabric coating, the excavation will be filled with Flowable Fill to within 2 or 3 feet of final ground surface. The Flowable Fill will sandwich the ends of Panel 16 and 18 to assure a solid bond of concrete between the two panels (see attached hand sketch, Figure 1). This NYSDOT Flowable Fill material has strength between 30 and 300 psi at 28 days (see attached Report of Concrete Mix Design and Keystone Material Testing, Inc. laboratory data showing strength testing of Flowable Fill in excess of 200 psi).

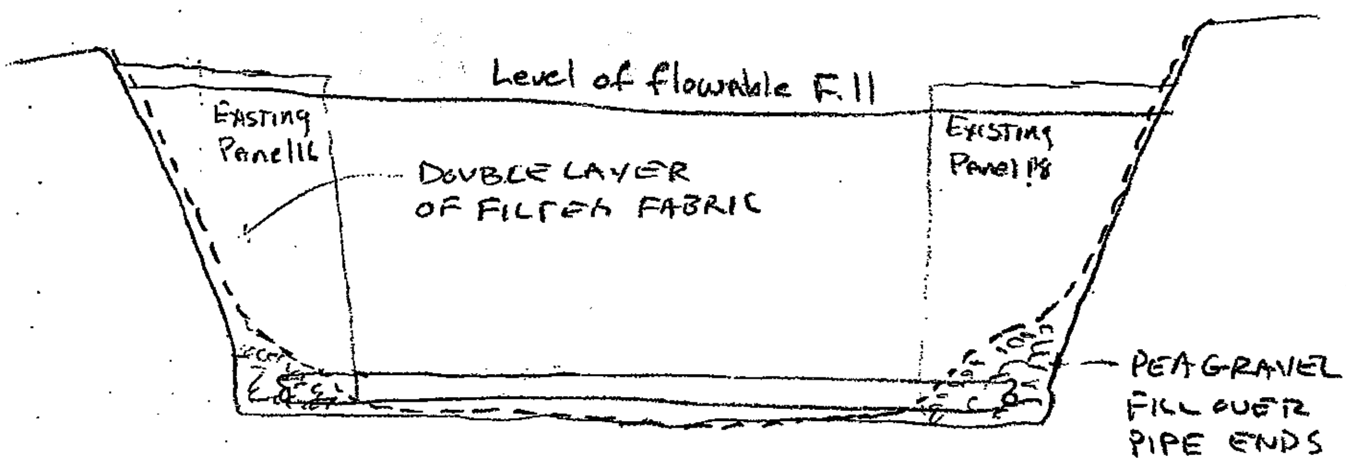
When the thin set concrete (Flowable Fill) cures to 28 days, a solid Portland cement, concrete barrier of at least 5-feet wide will replace the disconnected Panel 17 and form a barrier against flow from the landfill and direct the flow to the sump pumps on either end of the repair.

If you have any questions concerning this repair, or wish to discuss the procedures in detail, please do not hesitate to call me at (609) 584-6873.

Sincerely

Kristin A. Alzheimer, PE
NY PE# 72125
Shaw Environmental, Inc.





Richardson Hill

FIGURE 1

Quality Inspection Services, Inc.

Corporate Headquarters
Cathedral Park Tower
37 Franklin Street • Suite 400 • Buffalo, New York 14202
(716) 853-2611 • Fax (716) 853-2619
Visit Us At: www.qisi.com *E-Mail:* buffalo@qisi.com

REPORT OF CONCRETE MIX DESIGN

For: B & B Ready Mix - Greene, NY
Project: Various
Concrete Supplier: B & B Ready Mix - Greene, NY
Architect/ Engineer: –
General Contractor: –

Date: 7/30/02
Project No: SC-452
W.O. No: 35-1876
Mix I.D: QIS 420

SPECIFICATION REQUIREMENTS

Strength: 2,500 psi min.	@ 28 days	Slump: 8" - 10" inch max.	Entr. Air –%	To –%
Cement Type: I/II		Amount: 5.5 bags/cu. yd. Min.		W/C: –
Aggregate (Kind & Size Range) Coarse: Gravel #1's		Fine: ASTM C-33		
Admixture: Mid Range Plasticizer				

SOURCE OF MATERIALS

Cement: Essroc Cement Co. - Rochester, NY	Admixture: The Master Builders Company
Fine Aggregate: B & B Ready Mix - Greene, NY	Coarse Aggregate: Chenango Asphalt - N. Norwich, NY

TRIAL BATCH QUANTITIES per CUBIC YARD

These batch weights will require adjustments to compensate for moisture condition of aggregates.

Cement Type: I/II	Lbs.: 517	Bags: 5.5	W/C: .64 by wgt.	Total Water: 333 lbs.	40.00 Gals.
Fine Aggregate (S.S.D.)		2,078 Lbs.	Admixture: Polyheed 997-8 oz/ 100 lbs. cement		
Coarse Aggregate (S.S.D.) #1's		1,000 Lbs.	Admixture:		
Coarse Aggregate (S.S.D.)		Lbs.	Slump: 8" in.	Entr. Air: –%	
			Fresh Unit Wgt.: 145.48 lbs./ cu. ft.		

COMPRESSIVE STRENGTH DATA

(Cylinder Size: 6" x 12" Unless Otherwise Noted)

Laboratory Number	Date Tested	Age (Days)	Cross-Sectional Area (in²)	Maximum Load (lbs)	Compressive Strength (psi)
29899	8-6-02	7	28.27	75,200	2,660
29900	8-6-02	7	28.27	77,740	2,750
29901	8-6-02	7	28.27	73,780	2,610
29902	8-27-02	28	28.27	104,030	3,680
29903	8-27-02	28	28.27	100,920	3,570
29904	8-27-02	28	28.27	102,050	3,610

Quality Inspection Services, Inc.
Civil Testing Services

Copy: 1-QISI

Scientific Technologies
170 Fort Path Road
Madison, Connecticut 06443
(203) 245-7743 • Fax (203) 245-8017

P.O. Box 352
Garnerville, New York 10923-0352
(845) 429-2000

2004 Edison Avenue
Jacksonville, Florida 32204
(904) 359-0747 • Fax (904) 359-07
Toll Free (800) 927-3575

318 North Morrison Street
Warren, Pennsylvania 16365
(814) 726-1988 • Fax (814) 726-7

Sustaining Member

401 William Gaiter Parkway, Suite # 5
Buffalo, New York 14215
(716) 831-1404 • Fax (716) 831-1408

186 Warwick Avenue
Buffalo, New York 14215
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6730 Myers Road
East Syracuse, New York 13057
(315) 431-4291 • Fax (315) 431-4292

1576 Sweet Home Road
Amherst, New York 14228
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"For Job Satisfaction - Think Quality"

2376 STATE HIGHWAY 12
GREENE, NEW YORK 13778
Phone: 607-656-4145
Fax: 607-656-8919

READY MIXED CONCRETE
WASHED SAND & GRAVEL

REPORT OF CONCRETE MIX DESIGN

PROJECT NAME: RT. 8 DEPOSIT D259413 **DATE:** 11/07/03
GENERAL CONTRACTOR: KETCO
ARCHITECT/ENGINEER: N/A

SPECIFICATION REQUIREMENTS

STRENGTH 30-300 PSI MIN @ 28 DAYS SLUMP 6-8" ENTR. AIR N/A
CEMENT TYPE I/II MIN. AMT 185 LBS/C.Y. W/C RATIO N/A BY WGT.
COARSE AGGREGATE NONE FINE AGGREGATE: ASTM C-33 CONCRETE SAND
ADMIXTURES: AIR ENTRAINMENT

SOURCE OF MATERIALS

CEMENT ESSROC OSWEGO NY ADMIXTURES MASTER BUILDERS
FINE AGGREGATE B&B READY MIX, GREENE,NY SOURCE APPROVAL#02AF27
COARSE AGGREGATE ______ SOURCE APPROVAL#
OTHER N/A

DESIGN BATCH QUANTITIES PER CUBIC YARD

CEMENT TYPE I/II 185LBS 1.96BAGS /C.Y.
POZZOLAN 40
FINE AGGREGATE 3203LBS (SSD)
COARSE AGGREGATE 0LBS (SSD)
FIBER REINFORCEMENT N/A LBS (SSD)
W/C RATIO N/A BY WGT.1120 N/A LBS 50.GALS
ADMIXTURE MBVR 2.0/cwt
REMARKS: +.
CONTRACTOR IS ALSO RESPONSIBLE FOR MAXIMUM SLUMP.
CONTROLLED BACKFILL NYSDOT

NOTE: Batch weights may require adjustments in the field to compensate for moisture condition of the aggregates. Compressive strengths may be adversely affected by extreme temperatures, delays, and other field conditions when taking tests. ASTM curing procedures must be followed.

APPROVED BY: ARCHITECT/ENGINEER	RESPECTFULLY SUBMITTED FOR APPROVAL
SIGNED ____ DATE ____	
GENERAL CONTRACTOR	FOR B&B READY MIX
SIGNED ____ DATE ____	CC: ____

Keystone Material Testing, LLC

Information Data Sheet

Samples Received By: Bill Ellsworth Date: 07/6/99

Project Name and Location: Test Cylinders

Report Submitted To: B & B Ready Mix

Attn: Rich

Address:

Phone: Fax:

Copies To:

Any Additional Info.: Truck 51, Slump 6, Air N/A [illegible]

Notes:

Soil Samples

Source: ______ Sampled By: ______ Date of Sampling: ______

What kind of testing to be done: ___ Maximum Density/Optimum Moisture Content ___ Wet Sieve

___ Moisture Content ___ Other: ______

Type of Sample: ___ Jars ___ Buckets ___ Other: ______

Concrete Cylinder Testing

Location of Pour: Test

Date Cast: 06/8/99 Sampled By: REC Mix ID: ______ psi

\# of Cylinders: 2 Testing Schedule: 28

Test Results:

Lab Indent No.	Date Tested	Age (days)	Strength (psi)	Failure Type
8822	7/6/99	28	224	
8823	7/6/99	28	210	

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 009

Project Number: 742577	Date: 6/6/05 **(Final - Revised 02/27/06)**
Engineer: Parsons	
Remedial Action Contractor: DA Collins	

On behalf of Amphenol, you are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Landfill Cap:

a. Drainage Benches: Install additional drainage benches on the landfill cap. (See Figure 2, Drainage Benches and Subsurface Collection System for layout and Figure 3, Liner Flap Detail for details of construction.) The additional drainage benches will be created using sand and HDPE liner extrusion welded to the existing HDPE liner. The new benches are located at the uphill side of the north landfill half and around the TSCA cell. (Note that the missing bench at the west (uphill) end of the south landfill half will also be installed using this method.)

b. Lateral Drains: Install lateral drains in all drainage benches on the landfill cap. (See Figure 2, Drainage Benches and Subsurface Collection System for layout, Figure 4, Lateral Drain Detail for details of construction.)

c. Specification 02260, Soil Cover Layers, Table 02260-1: Increase the QC testing frequency for the hydraulic conductivity (ASTM D5084), moisture content (ASTM D2216), and particle size analysis (ASTM D422) from one test every 3,000 cubic yards to one test every 1,000 cubic yards of barrier protection material placed.

Revised Cap Design:

The original cap design was independently reviewed by Parsons and Professors Zimmie and De of Rensselaer Polytechnic Institute (RPI) since the erosion of the placed drainage sand in 2004. Both reviews concluded that the cap as designed is stable under final conditions, but that the stability, particularly during construction, can be improved by increasing the hydraulic conductivity of the drainage layer by using a geosynthetic drainage composite (GDC) in lieu of drainage sand. The following changes are being incorporated in addition to changes a, b and c above.

Specification 02260-2.02, Modified Barrier Protection Soil: Decrease the maximum hydraulic conductivity to 1.0E-4 cm/sec at 95% compaction based on ASTM D1557 to increase the factor of safety during construction by minimizing infiltration into the drainage layer prior to topsoil placement.

2a. Specification 02260-2.01, Modified Drainage Sand: (Superceded by 02260-2.02, Modified Barrier Protection Soil (Item 1 above))

2b. Install a geosynthetic drainage composite (GDC) and 24" of modified barrier protection soil in lieu of the 12" drainage sand and 12" of specified barrier protection soil. The GDC material shall meet the requirements specified below:

- A minimum transmissivity of 8.5E-4 m^2/sec at 350 psf, 0.33 gradient.

The geotextile affixed to the GDC that is in contact with the barrier protection soil shall achieve the following subsurface drainage criteria of a Class 2 geotextile as defined in ASSHTO M288-96. The drainage criteria in accordance with ASSHTO M288-96 is as follows:

Filter Criteria	Units	Percent of In-situ Soil Passing No. 200 (0.075 mm) Sieve		
		<15	15 to 50	>50
Minimum Permittivity, ASTM D-4491	sec^{-1}	0.5	0.2	0.1
Maximum AOS, ASTM D-4751	mm	0.43	0.25	0.22

Note: Provide manufacturer's certified test results as required by Specification 02621-1.03A, Geocomposite. The SKAPS TN-270-7/10 GDC should be installed parallel to (down) the slope with the 10-ounce geotextile placed against the textured geomembrane.

3. Specification 02990-2.01A.1, Topsoil and Seeding: Delete the topsoil filter criteria when the modified barrier protection soil is installed. The topsoil filter criteria was removed from the proposed modified design because if some clogging of the barrier protection soil occurs at the interface of the topsoil the resulting insitu permeability of the barrier protection material will decrease thereby increasing the factor of safety.

4. Drawing C-10, Detail C: Delete the fine riprap at the toe of the cap. (The geocomposite will drain directly to the riprap swale.)

Technical Backup:
Attachment B - Rev F, Drainage Evaluation
Attachment C, Veneer Stability (Option 2 – GDC Over Geomembrane)
Attachment D, Erosion Potential (Universal Soil Loss Equation)
Attachment E, GDC Test Results and Landfill Measurements

- Figure 1, Drainage Distances and Areas
- Figure 2, Drainage Benches and Subsurface Collection System
- Figure 3, Liner Flap Detail
- Figure 4, Lateral Drain Detail

Responses to USEPA Comments (Dated November 15, 2005)

APPROVALS:

Parsons Representative

Name: William J. Long	**Date:** 2/28/06
Signature: [signature]	

Name: James O'Loughlin	**Date:** 2/28/06
Signature: [signature]	

Name: Edward C. Glaza, P.E.	**Date:** 2/28/06
Signature: [signature]	

DA Collins Representative

Name: Dave MacDougall	**Date:** 3/1/06
Signature: [signature]	

USEPA

Name: Young Chang	**Date:**
Signature:	

cc: Joe Bianchi - Amphenol
Frank Leming – JTM Associates
Jeff Hall - EarthTech

APPROVALS:

Parsons Representative

Name: William J. Long	Date: 2/28/06
Signature: [signature]	

Name: James O'Loughlin	Date: 2/28/06
Signature: [signature]	

Name: Edward C. Glaza, P.E.	Date: 2/28/06
Signature: [signature]	

STATE OF NEW YORK EDWARD CADWELL GLAZA 070909 LICENSED PROFESSIONAL ENGINEER

DA Collins Representative

Name: Dave MacDougall	Date: 3/1/06
Signature: [signature]	

USEPA

Name: Young Chang	Date: 3/9/06
Signature: [signature]	

cc: Joe Bianchi - Amphenol
Frank Leming – JTM Associates
Jeff Hall - EarthTech

Rev	Date	By	Ck
A	4/20/2005	JPW	CMG
B	6/6/2005	JPW	CMG
C	8/19/2005	CMG	JPW
D	9/8/2005	CMG	JPW
E	10/28/2005	CMG	JPW
F	11/15/2005	CMG	JPW

Title: Honeywell
Richardson Hill Landfill
Sand Drainage Capacity Calculations Using Laboratory Test Data
Giroud's Method

Objective:

Based on laboratory hydraulic conductivity test data, determine if the drainage sand that has been placed on the cover of the Richardson Hill Landfill (RHLF) by Shaw has sufficient drainage capacity and confirm that the original design parameters produce an acceptable factor of safety. Additionally, the drainage capacity will be evaluated for the as-built landfill configurations as of September 2005.

Assumptions:

1. Confirmatory grain size and hydraulic conductivity test results from both Parsons/CME and Shaw/GeoTesting Express/Emcon data will be used for the existing conditions analysis.
2. Submitted values for grain size and hydraulic conductivity from Shaw Submittals #140 (9/17/04), #146 (11/04/04), #147 (12/01/04), #148 (12/01/04 - 12/11/04), and #150 (12/01/04).
3. Transmissivity values for SKAPS TN270-10/7GDC obtained from transmissivity testing (ASTM D4716) performed by SKAPS Industries on rolls 1711111, 1711121, and 1711129 on 9/7/05 and 9/8/05. The transmissivity testing was performed using a normal load of 350 psf which represents the potential loads the GDC will be subjected to within the cap.
4. For the sand drainage layer calculations, it was assumed that the topsoil used will have a hydraulic conductivity of 1.0E-5 cm/sec or less. It was assumed that the topsoil layer is allowing water to percolate at this rate. This corresponds to a worst case scenario during spring snowmelt. Evapotranspiration is assumed to be zero at this time of year.
5. The factor of safety for the drainage layer is discussed in Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers (Ref 1). This design guide recommends an overall drainage system factor of safety of between 2 and 3 for landfill closures when using a granular drainage layer. The factors of safety used in this calculation are presented below in the Factor of Safety section. The factors of safety presented below assume good quality control practices during construction and repair of areas that do not meet the minimum hydraulic conductivity requirements estimated herein (for both the drainage layer and the overlying layers).
6. An effective lateral drainage collector will be installed at the base of each bench.

Approach:

Estimate the drainage capacity of the cover system using modified Giroud's equations (Ref 1) for the following cases:

Note that the drainage capacity is evaluated for varying slope lengths (Subcases A, B, and C).

For Cases 1 to 4 these slope lengths are represented by the design typical horizontal length between benches (75 feet), and design maximum horizontal length between benches (110 feet); the 2004 as-built typical horizontal length between benches (87 feet), the 2004 as-built maximum horizontal length between benches (110 feet), and the 2004 as-built maximum horizontal length above the existing upper bench (170 feet).

For Case 5 the slope lengths are represented by the as-built slope lengths between benches as of 9/6/05. Case A evaluates a typical case for the majority of the landfill where the slope lengths are 85 feet or less and the slope is 3 Horizontal to 1 Vertical (3H:1V). Case B evaluates the longest slope length on a 3H:1V slope. Case C evaluates a critical slope length of 90 feet on a slope of 5H:1V near the top of the landfill. Note that these slope lengths are not horizontal lengths but measured along the as-built slopes. These lengths are adjusted to horizontal lengths in the calculations (see Page 4).

Case	$k_{topsoil}$ (cm/sec)	k_{drain} (cm/sec)	k_{drain} Description	Subcase A L (ft)	Subcase B L (ft)	Subcase C L (ft)
1	1.00E-05	1.00E-02	design (spec) value	75	110	n/a
2	1.00E-05	1.20E-02	Shaw Submittal #140 material value	87	110	170
3	1.00E-05	1.53E-03	Geometric mean of Parsons data	87	110	170
4	1.00E-05	5.28E-03	Geometric mean of Parsons and Shaw data	87	110	170
5	1.00E-05	(See Note)	SKAPS TN270 GDC Drainage Layer	85	118	90

Note: Case 5 evaluates the use of a GDC for the drainage layer with a transmissivity of 8.5E-4 m^2/sec.

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Title: Honeywell
Richardson Hill Landfill
Sand Drainage Capacity Calculations Using Laboratory Test Data
Giroud's Method

The modified Giroud's equations for granular drainage layers are presented below (Ref. 1):

q_h = liquid supply rate (assumed equal to $k_{topsoil}$)

k_{LTIS} = hydraulic conductivity of the drainage layer, (long term in soil flow capacity)

$k_{LTIS} = k_{measured} / (RF_{PC} \times RF_{CC} \times RF_{BC})$

λ = dimensionless characteristic parameter

$\lambda = q_h / k_{LTIS} \tan^2 \beta$

β = Slope angle

$$t_{max} = j \frac{\sqrt{1+4\lambda} - 1}{2\cos\beta / \tan\beta} L$$ (Giroud's modified equation)

$$j = 1 - 0.12 \exp\left[-\left[\log\left(\frac{8\lambda}{5}\right)^{5/8}\right]^2\right]$$ (Modification factor)

$FS_T = t_{allow} / (t_{max}(k_{LTIS}))$ (Factor of safety based on allowable head on the liner (t_{allow}))

Note that $t_{max}(k_{LTIS})$ in the equation above is not t_{max} multiplied by k_{LTIS}, but that it is t_{max} calculated as a function of k_{LTIS}.

Where:

$k_{measured}$ = hydraulic conductivity of a specimen of granular material representative of the granular material installed, measured in a hydraulic conductivity test performed with water during a short period of time such that clogging does not develop

RF_{PC} = reduction factor due to particulate clogging

RF_{CC} = reduction factor due to chemical clogging

RF_{BC} = reduction factor due to biological clogging

β = slope angle

L = horizontal length of the slope

When using GDC for the drainage layer, the equations are the same except that θ_{LTIS} has additional potential reduction factors:

$\theta_{LTIS} = \theta_{measured} / (RF_{IMCO} \times RF_{IMIN} \times RF_{CR} \times RF_{IN} \times RF_{CD} \times RF_{PC} \times RF_{CC} \times RF_{BC})$

Where: RF_{IMCO} = Reduction factor due to immediate compression of GDC core

RF_{IMIN} = Reduction factor due to immediate intrusion of geotextile into GDC

RF_{CR} = Reduction factor due to time-induced creep of the core under applied stress

RF_{IN} = Reduction factor due to delayed intrusion of geotextile into GDC

RF_{CD} = Reduction factor due to chemical degradation of GDC

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E	10/28/2005	CMG	JPW				
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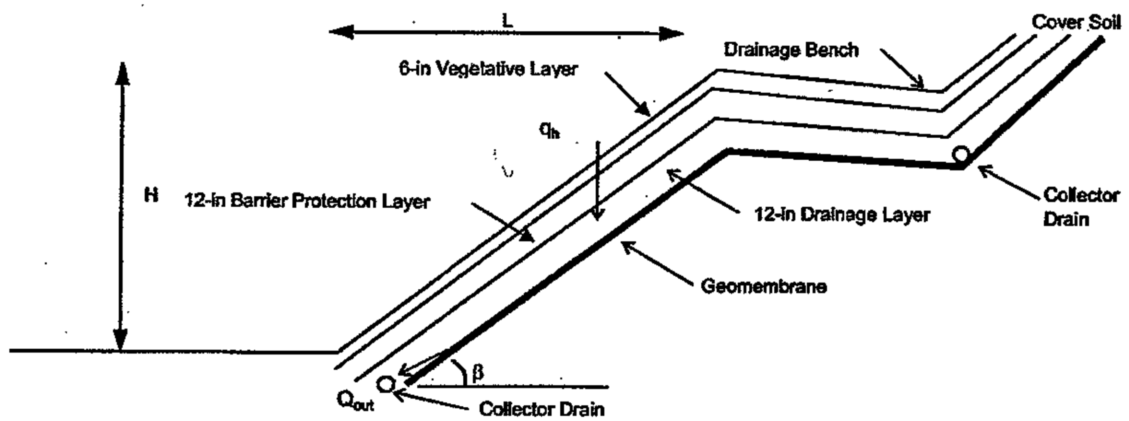

Figure 1 - Typical Cover Section Using Drainage Sand

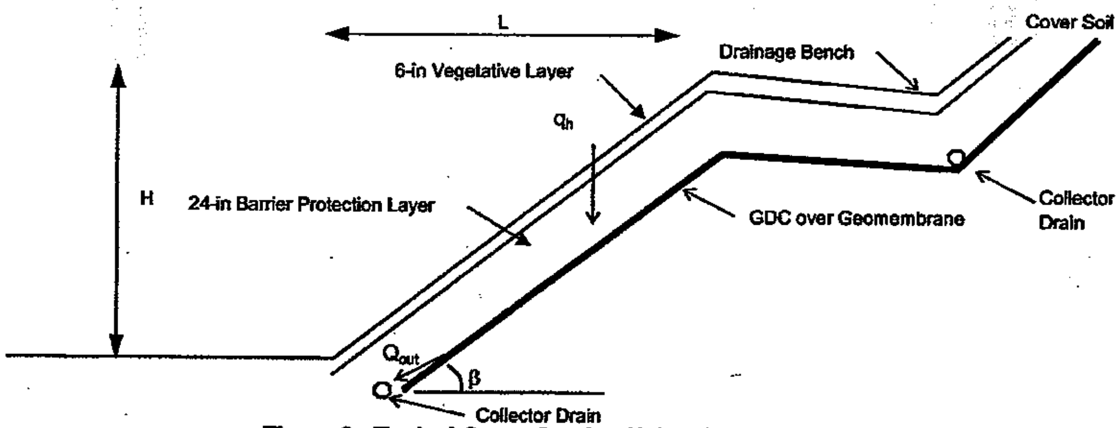

Figure 2 - Typical Cover Section Using GDC

Factor of Safety:

The factors of safety and the reduction factors used in this calculation are presented below. FS and RF values below are selected based on guidance provided in Ref. 1.

	Sand	GDC	
FS_d=	2.5	2.5	Design factor of safety
RF_{IMCO} =	N/A	1.0	N/A for sand, GDC test values are equal to or higher normal stress than will be present in cover
RF_{IMIN} =	N/A	1.0	N/A for sand, GDC test values are equal to or higher normal stress than will be present in cover
RF_{CR} =	N/A	1.4	N/A for sand, max value for GDC
RF_{IN} =	N/A	1.2	N/A for sand, max value for GDC
RF_{CD} =	1.0	1.0	Liquid is primarily water, chemical degradation not an issue
RF_{PC} =	1.0	1.0	Assumes filters are properly designed and particulate clogging is therefore negligible
RF_{CC}=	1.0	1.2	Assumes chemical clogging potential is negligible in sand; max value for GDC
RF_{BC} =	1.2	1.5	Small potential for biological clogging of the drainage sand; max value for GDC
Π(RF)	1.2	3.0	Overall reduction factors for computation of k_{LTIS} and θ_{LTIS}

Thus:

$k_{LTIS} = k_{measured} / 1.2$

$\theta_{LTIS} = \theta_{measured} / 3.0$

Calculations:

Geometry (Cover Slope Between Benches):

The following slope geometry is used to represent the final cover system:
Length of slope between benches, L (ft) = as shown above for various cases

Slope between benches =	3.0 Horizontal		1.0 Vertical
Slope Angle between benches	β = 0.3217506 radians =		18.43 degrees
Case 5c Slope =	5.0 Horizontal		1.0 Vertical
Case 5c Slope Angle =	β = 0.1973956 radians =		11.31 degrees
Maximum allowable head on liner =		0.30 m	

Results:

Case	q_h (m/sec)	$k_{measured}$ (= k_{drain}) (cm/sec)	k_{LTIS} (m/sec)	λ	j	Subcase A t_{max} (m)	A FS_T	Subcase B t_{max} (m)	B FS_T	Subcase C t_{max} (m)	C FS_T
1	1.00E-07	1.00E-02	8.33E-05	0.0108	0.9643	0.083	3.6	0.121	2.5	n/a	n/a
2	1.00E-07	1.20E-02	1.00E-04	0.01	0.9681	0.080	3.7	0.102	2.9	0.157	1.9
3	1.00E-07	1.53E-03	1.28E-05	0.07054	0.9155	0.564	0.53	0.714	0.42	1.103	0.27
4	1.00E-07	5.28E-03	4.40E-05	0.02044	0.9493	0.177	1.7	0.224	1.3	0.346	0.87

Assuming that the maximum drainage length is held to the maximum design length of 110-feet (horizontal) or less, Cases 1 and 2 demonstrate that the original design and the revised design based on the approved submitted values for the hydraulic conductivity of the drainage sand meet the recommended factor of safety of 2.5. Case 3 demonstrates that the sand placed on the landfill does not meet the recommended factor of safety of 2.5 using the geometric mean of Parsons' collected data. Case 4 demonstrates that the drainage sand placed on the landfill does not meet the recommended factor of safety of 2.5 using the combined geometric mean of Parsons' and Shaw's data.

Case 5: Geosynthetic Drainage Composite

$$FS_T = FS_H = \frac{\theta_{measured}}{\Pi(RF)} * \frac{\sin\beta}{q_h L}$$ (for rectangular area)

Where: $\theta_{measured}$ is the minimum value from SKAPS TN 270 7/10 test data submitted to Parsons by SKAPS on 9/07/200 5 and 9/8/05.

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Giroud's Method

Calculate $\theta_{measured}$ based on SKAPS test data at 350 psf normal load.

Sample	Load (psf)	Gradient	$\theta_{test\ value}$ (m^2/sec)
1	350	0.1	1.32E-03
2	350	0.1	1.40E-03
3	350	0.1	1.37E-03
		Mean	1.36E-03
		Std. Dev.	4.04E-05
		$\theta_{measured}$	1.28E-03
4	350	0.33	8.50E-04

Conversion of slope lengths to horizontal lengths.

Subcase	Slope Length (ft)	Horizontal Length L_H (ft)
A	85	80.6
B	118	111.9
C	90	88.3

Sub Case	Description	q_h (m/sec)	$\theta_{measured}$ (m^2/sec)	L_H (ft)	FS
A	Max drainage length of 85 feet	1.00E-06	8.50E-04	80.6	3.6
B	Max drainage length of 118 feet	1.00E-06	8.50E-04	111.9	2.6
C	Max drainage length of 90 feet (See Note)	8.30E-07	8.50E-04	88.3	2.5

Note: For Subcase C the as-built drainage areas are trapezoidal; however, to simplify the calculation they were assumed to be rectangular. The trapezoidal area reduces the contributing area of flow resulting in a factor of safety higher than the calculated value.

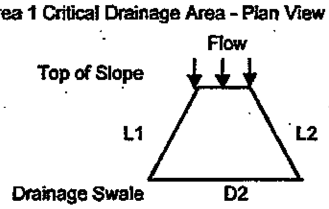
For a drainage length of 85 feet (slope length), the factor of safety against drainage layer failure for the Skaps TN 270-7/10 is 3.6; for a drainage length of 118 feet, the factor of safety is 2.6; and for a drainage length of 90 feet, the factor of safety is 2.5. All of these values exceed the minimum factor of safety of 2.5.

Check assumptions of Subcase C

The purpose of this calculation is to confirm that a hydraulic conductivity of 1.0E-6 m/sec is sufficient for Subcase C.

The trapezoidal geometry of the contributing drainage area is smaller than a contributing drainage area with equal lengths and widths (i.e., rectangular). This results in a smaller liquid inflow rate and increases the factor of safety.

Cap Area 1 Critical Drainage Area - Plan View (see Figure 1 for location)



L1 =	80 ft	24.4 m
L2 =	84 ft	25.6 m
D2 =	81.5 ft	24.8 m
Area (from ACAD) =	4924 ft^2	457.5 m^2

$$F.S. = \frac{Q\ out}{Qin} = \frac{\theta_{LTIS} * width * \sin\beta}{k_{bpm} A}$$

Horizontal Projected Area (A_h)

$A_h = \cos\beta * Area =$ 448.6 m^2

$k_{bpm} =$ 1.00E-04 cm/sec   1.00E-06 m/sec

FS = 3.1 ok

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Sand Drainage Capacity Calculations Using Laboratory Test Data
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SUMMARY OF RESULTS:

Case	Description	Factor of Safety for Subcase		
		A	B	C
1	k_{drain} = design (spec) value	3.6	2.5	n/a
2	k_{drain} = Shaw Submittal #140 material value	3.7	2.9	1.9
3	k_{drain} = Geometric mean of Parsons data	0.53	0.42	0.27
4	k_{drain} = Geometric mean of Parsons and Shaw data	1.7	1.3	0.87
5	SKAPS TN270-7/10 GDC Drainage Layer	3.6	2.6	2.5

Conclusions and Recommendations:

1. The results of the evaluation indicate that an acceptable factor of safety cannot be achieved using the drainage sand placed in 2004 that is currently on the landfill cover. The drainage sand does not have sufficient capacity to convey the estimated peak flow from the barrier protection layer using Parsons and Shaw combined test values. It is recommended that the drainage sand be removed in a manner that minimizes damage to the underlying liner and be replaced with materials consistent with the recommendations provided below.

2. If sand is used as the drainage layer, modify the original design specifications as follows:

- Decrease the maximum hydraulic conductivity of the 12-inch barrier protection layer to 5.0E-5 cm/sec to increase the factor of safety during construction, minimize infiltration into the drainage sand layer, and minimize the potential risks associated with damage to the topsoil layer from erosion, root damage, or animals.

- Modify the drainage sand requirements to include:

The drainage sand should be well graded and stable. For a soil to be classified as both well graded and stable, it must meet the following criteria (Ref 2):

$$C_U = \frac{D_{60}}{D_{10}} > 4 \quad \text{and} \quad 1 < C_C = \frac{D_{30}^{\,2}}{D_{10} D_{60}} < 3$$

Where:
C_U = Coefficient of uniformity
C_C = Coefficient of curvature
D_{60} = the diameter at which 60 percent of the soil is finer
D_{30} = the diameter at which 30 percent of the soil is finer
D_{10} = the diameter at which 10 percent of the soil is finer

The drainage sand must act as a filter to the overlying barrier protection soil. For the drainage sand to be an effective filter, it must meet the following criteria:

$D_{95,filter} > 3 * D_{15,barrier}$ and $\frac{D_{95,filter}}{D_{85,barrier}} < B$

B = 1 for $C_{U,barrier} > 8$
B = $8/C_{U,barrier}$ for $4 < C_{U,barrier} < 8$

Where:
$D_{95,filter}$ = the diameter at which 95 percent of the filter (drainage sand) is finer
$D_{85,barrier}$ = the diameter at which 85 percent of the barrier protection layer is finer
$D_{15,barrier}$ = the diameter at which 15 percent of the barrier protection layer is finer
$C_{U,barrier}$ = coefficient of uniformity of the barrier protection soil

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3. Consider an alternate drainage layer design which further increases the factor of safety and minimizes the risk of wash out during construction. The alternate drainage layer design substitutes a geosynthetic drainage composite (GDC) with a minimum transmissivity of 8.5E-4 m^2/sec at a gradient of 0.33, based on laboratory test data from SKAPS at 350 psf normal load. The barrier protection layer should have a hydraulic conductivity no greater than 1.0E-4 cm/sec if this alternative is selected.

If GDC is selected for the drainage layer, the following revised drainage system design should be utilized (top to bottom):

• 6-inch layer of topsoil meeting the design topsoil specifications
• 24-inch barrier protection layer with a maximum hydraulic conductivity of 1.0E-4 cm/sec to increase the factor of safety during construction and minimize the potential risks associated with damage to the topsoil layer from erosion, root damage, or animals.

• GDC material that meets the requirements specified below:
A transmissivity of 8.5E-4 m2/sec or greater when tested following ASTM D4716 for the cap loads and at a gradient of 0.33.
The geotextile affixed to the GDC that is in contact with the barrier protection soil shall achieve the following subsurface drainage criteria of a Class 2 geotextile as defined in AASHTO M288-05. The drainage criteria in accordance with AASHTO M288-05 is as follows:

Filter Criteria	Units	Percent of In-situ Soil Passing No. 200 (0.075 mm) Sieve		
		< 15	15 to 50	>50
Minimum Permittivity, ASTM D-4491	sec^{-1}	0.5	0.2	0.1
Maximum AOS, ASTM D-4751	mm	0.43	0.25	0.22

• 40-mil geomembrane, textured both sides (already in place).
• GDC material (already in place).
• Increase the QC testing frequency specified (Table 02260-1) for the hydraulic conductivity (ASTM D2434), moisture content (ASTM D2216), and particle size analysis (ASTM D422) for the drainage sand to one test every 500 cubic yards and the barrier protection layers to one test every 1000 cubic yards of material placed.

Note: Based on the parameters above, and a reported AOS (O_{95}) of the 7-oz geotextile side of the geocomposite equal to US sieve #70 (0.212 mm), the D_{15} of the barrier protection soil layer must be less than or equal to 0.071 mm.

4. Construct additional drainage benches and extend existing drainage benches as necessary to limit the maximum drainage length to 110 feet. Drainage benches can be constructed on the existing LDPE liner where necessary using a liner flap.

5. Install lateral collector drains along each slope bench to provide an outlet for the drainage layer and to prevent pore water pressure from building up at these locations. To effectively remove water from the cover system, the collector should be placed directly at the toe of the slope.

6. A Parsons engineer with landfill cap construction experience should conduct a site visit to verify the conditions assumed in these calculations. Additionally, it is recommended that a Parsons engineer experienced in landfill cap construction be on site periodically during the sand removal, liner removal and testing (as applicable), and a minimum of once a week during subsequent construction.

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Sand Drainage Capacity Calculations Using Laboratory Test Data
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References:

1. Giroud, J.P., Zonrberg, J.G., and Ahao, A., "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers," Geosynthetics International, Vol. 7, 2000, pp. 285-380.
2. Richardson, G. N., Giroud, J. P., Zhao, A., "Design of Lateral Drainage Systems for Landfills," 2000.
3. Thiel, R.S. and Stewart, M.G., "Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific Northwest," Proceedings, Geosynthetics 1993, Vancouver, IFAI, pp. 1131-1144.
4. AASHTO M288-05, Geotextile Specification for Highway Applications, 2005

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A	9/22/2004	JPW	CMG	Honeywell
B	4/18/2005	JPW	CMG	Richardson Hill Landfill
C	6/6/2005	JPW	CMG	Veneer Stability Calculations Using Laboratory Test Data
				Option 2 - GDC over Textured Geomembrane

Objective:

Based on recently obtained laboratory test data, analyze the drainage geocomposite interface with the geomemberane and determine if the factor of safety meets the design requirements for veneer slope stability failure of the Richardson Hill Landfill (RHLF) cover. Utilize the previously established veneer stability calculation worksheet that was based on assumed parameters.

Assumptions:

1. The laboratory test data of direct shear testing (ASTM D 5321) and interface friction testing by GeoTesting Express (Shaw Submittals #140 and #142) will be used for this analysis and is representative of the materials that will be used to construct the cover system. The material used for drainage sand and barrier protection soil was from Clark Stone Poultz pit.
2. The probability of occurrance of an earthquake during peak saturation conditions in the 30-year design life of the landfill is low. The pseudo-static condition is analyzed in the unsaturated condition (no hydraulic head).
3. The worst case scenario for drainage (static condition) is that the geosynthetic drainage composite (GDC) layer will become fully saturated at some point during the 30-year design life of the landfill.
4. The difference in hydraulic conductivities between the barrier protection layer and the GDC will create a capillary break between the two layers. Because of the capillary break, the case of saturation of the full thickness of the cover system is not evaluated.
5. In accordance with Subtitle D regulations, acceptable factors of safety are 1.5 (static) and 1.05 (pseudo-static).

Approach:

Estimate the factor of safety (FS) against sliding of the cover system for the following cases:

Option	Description
2-1	GDC sliding on the geomembrane under static, dry conditons.
2-2	GDC sliding on the geomembrane in the static condition with a hydraulic head acting on the geomembrane.
2-3	Cover in pseudo-static condition with no hydraulic head.

Geometry and Material Properties

The following slope geometry configurations define the landfill cap.

Property	Typical Slope, North	Typical Slope, Center	Typical Slope, South	Inter-bench Slope
Height of slope, H (ft) =	47	80	80	36.7
Length of slope, L (ft) =	160	271	287	110

The options are therefore divided into sub-options according to the following:

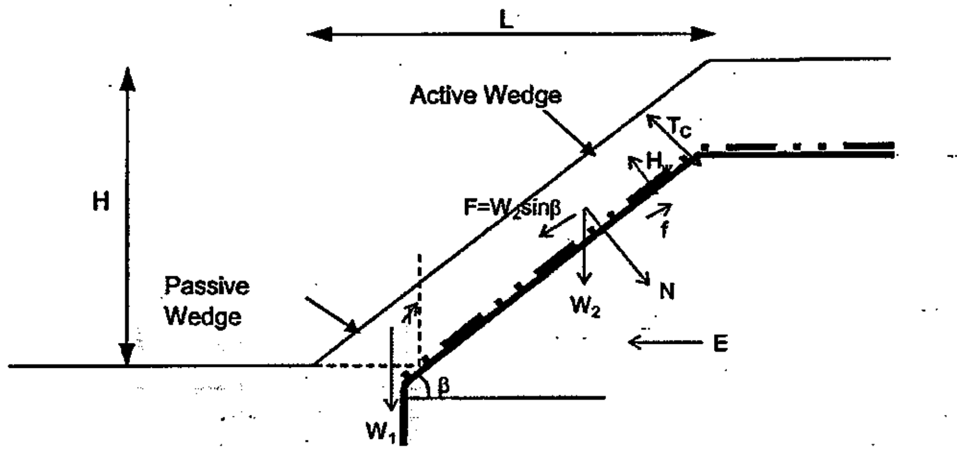
Sub-Option	Description
a	Full Slope, Center geometry
b	Inter-bench (as-constructed)

Note that the passive force can be relied upon only for **Sub-Option a** that extends to the base of the cover.

Calculations:

$$W_1 = \frac{\gamma_c T_c^2}{\sin 2\beta} \qquad W_2 = \frac{\gamma_c T_c^2}{\sin 2\beta}\left(\frac{2H\cos\beta}{T_c} - 1\right) \qquad P = W_1 \frac{Sin\phi_c}{\cos(\phi_c + \beta)}$$

$$F = W_2 Sin\beta \qquad f = \tan\delta\left[W_2\cos(\beta-\delta)\right] \qquad E = W_2 k_s$$



Geometry and material properties used:

Thickness of cover, Tc =	2.5 ft		
Unit weight of cover, γ_c =	123.5 lb/ft^3 (avg of 90% maximum dry density from Proctor)		
Unit weight of water, γ_w =	62.4 lb/ft^3		
Friction angle of cover soil, ϕ_c =	45.8 degrees	0.799 radians	(Submittal #140)
Interface friction angle, δ =	27 degrees	0.471 radians	(Submittal #140)
Ground acceleration, k_s =	0.075		

Sub Case	Slope Height H (ft)	Slope Length L (ft)	Slope Angle β (degrees)	Slope Angle β (radians)	W_1 (lbs/ft)	W_2 (lbs/ft)	P (lbs/ft)	f (lbs/ft)	F (lbs/ft)	E (lbs/ft)
a	80	271	16.4	0.287	1421.6	85836.9	2188.6	42996	24303	6437.8
b	36.7	110	18.4	0.322	1286.7	34520.1	0	17393	10916	2589.0

Option 2-1: Veneer Stability of Cap During Static (no earthquake loading) Conditions:

$$FS = \frac{\Sigma \text{ Resisting Forces}}{\Sigma \text{ Driving Forces}}$$

Option 2-1a: Full length cover section with passive resistance at the base.

$$FS = \frac{f + P}{F} = 1.86$$

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Option 2-1b: Inter-bench cover section with no passive resistance at the base.

FS = $\frac{f}{F}$ = **1.59**

Check using Thiel and Stewart, 1993:

$$FS = \frac{T_c \gamma \tan \delta + \frac{P}{L \cos \beta}}{T_c \gamma \tan \beta}$$

Option 2-1a FS = 1.82 (Checks OK)
Option 2-1b FS = 1.53 (Checks OK)

Case 2: Account for hydraulic head

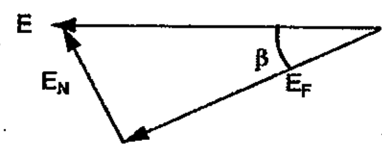
Height of water in cover Hw = 0.021 ft (full saturation of drainage layer)
Using Thiel and Stewart:

$$FS = \frac{[(T_c - h_w)\gamma + h_w \gamma_{sat} - h_w \gamma_w]\tan\delta + \frac{P}{L\cos\beta}}{[(T_c - h_w)\gamma + h_w \gamma_{sat}]\tan\beta}$$

Option 2-2a FS = 1.81
Option 2-2b FS = 1.52

Case 3: Veneer Stability of Cap During Psuedo-Static (with earthquake loading) Conditions:
Assuming no Hydraulic Head

$$E_N = E \sin \beta$$



$$E_F = E\cos\beta$$

		3a	3b
Vertical Component of Earthquake Loading (lb/ft) =	E_N =	1822.7	818.7
Horizontal Component of Earthquake Loading (lb/ft) =	E_F =	6174.4	2456.2

$$FS = \frac{[(T_c \gamma L) - E_N]\tan \delta + P}{(T_c \gamma L \tan \beta) + E_F}$$

Option 2-3a FS = 1.42
Option 2-3b FS = 1.23

Results:

The results of the veneer stability calculations for a GDC layer over a textured geomembrane are summarized in the table below.

Option	Factor of Safety	Acceptable?
2-1a	1.8	Yes
2-1b	1.5	Yes
2-2a	1.8	Yes
2-2b	1.5	Yes
2-3a	1.4	Yes
2-3b	1.2	Yes

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Conclusions and Recommendations:

The cover is stable with an acceptable factor of safety for all cases anlayzed.

References:

1. Giroud and Beech, "Stability of Soil Layers on Geosynthetic Lining Systems", Proceedings, Geosynthetics 1989.
2. Thiel, R.S. and Stewart, M.G., "Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific
3. GeoTesting Express, Interface Shear Test Series by ASTM D5321, Series #1; steel plate / SOIL / GEOMEMBRANE / textured
4. Algermissen et al., Probabilistic Earthquake Acceleration and Velocity Maps of the United States and Puerto Rico, USGS

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A	4/18/2005	JPW	MEP	Honeywell Richardson Hill Landfill Estimate of Erosion Potential of Final Cover using Universal Soil Loss Equation

Objective:

Based on Universal Soil Loss Equation (USLE), estimate the soil loss of the Landfill Final Cover System during a 30-year design life.

Approach:

Estimate the loss of soil per acre in tons/acre/year and the corresponding loss in inches per year.

The USLE Equation is:

A = R x K x LS x C x P

Where:
- A = Potential long term average annual soil loss [tons/acre/year]
- R = Rainfall and runoff factor based on geographic location
- K = Soil erodibility factor
- LS = Slope length-gradient factor
- C = Crop / vegetation and management factor
- P = Support practice factor

Calculations:

Geometry and Equation Factors

R = 110 (From Average annual rainfall factor map of US)

LS Factor: $LS = [0.065 + 0.0456(slope) + 0.006541(slope)^2] \times (slope\ length / constant)^{NN}$ Check eqn.

Where: slope = slope steepness (%)
= 33.33 %

slope length = length of slope in feet
= 110 feet

constant = 72.5 Imperial, 22.1 metric

NN = Slope factor
= 0.2 for slopes less than 1%
0.3 for slopes greater than or equal to 1% but less than 3%
0.4 for slopes greater than or equal to 3% but less than 5%
0.5 for slopes greater than 5% **(Used for calculation)**

C = Crop type factor * tillage method
0.02 Crop type factor (Hay and Pasture)
0.25 tillage method (No-till)

C = 0.005

P = 1 (Up and down slope)

Note: K factor values used below are for average organic matter content

Case	R	K	Soil Type	LS	C	P	A (tons/acre/yr)
SL-1	110	0.38	Silty Loam	10.90	0.005	1	2.3
SL-2	110	0.3	Clay Loam	10.90	0.005	1	1.8
SL-3	110	0.18	Fine Sandy Loam	10.90	0.005	1	1.1
SL-4	110	0.11	Loamy Fine Sand	10.90	0.005	1	0.66

PARSONS Calculation Sheet	Job Number: 742577	WBS Number: 05100	Page Number: 2	Sheet 2 of 2

Rev	Date	By	Ck	Title
A	4/18/2005	JPW	MEP	Honeywell Richardson Hill Landfill Estimate of Erosion Potential of Final Cover using Universal Soil Loss Equation

Estimate the soil loss in terms of inches per year.

Vegetative soil unit weight = 90 pcf (assumed, Ref. 2)

Loss Area = 43,560 ft^2

Case	A (lbs/acre/yr)	A ($lbs/ft^2/yr$)	Loss (ft/yr)	Loss (in/yr)	30-year Loss (in)
SL-1	4,557	0.10462	1.16E-03	1.39E-02	0.42
SL-2	3,598	0.08260	9.18E-04	1.10E-02	0.33
SL-3	2,159	0.04956	5.51E-04	6.61E-03	0.20
SL-4	1,319	0.03028	3.36E-04	4.04E-03	0.12

Conclusions and Recommendations:

All soil types evaluated produce acceptable average soil loss values.

References:

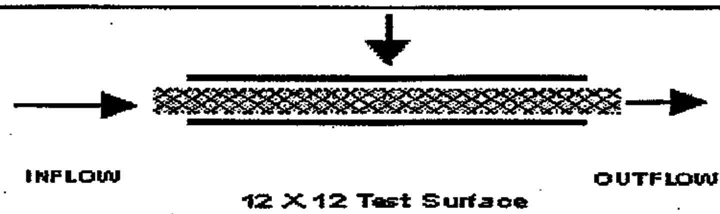
1. Ontario Ministry of Agriculture and Food website, http://www.gov.on.ca/OMAFRA/english/engineer/facts/00-001.htm.
2. R-factor map: www.abe.iastate.edu/AST324/Lesson5USLE.ppt
3. Topsoil unit weight, http://www.vitalearth.com/specbook/soils/enriched.htm.

SKAPS Industries

ASTM D 4716

Client:	Antana Linings, Inc.	**Job #**	1711
Project:	Richardson Hill Landfill, NY		
Product:	TN270-10/7		

Test Configuration:



Test Information:

Boundary Conditions: Steel Plate / Geocomposite / Steel Plate

Normal Load:	350 psf
Gradient:	0.33 ft
Seating Time:	15 minutes
Flow Direction:	MD

Test Results:

Roll No.	Pressure (psf)	Gradient, ft	Transmissivity, m^2/sec 15 minutes
1711111	350	0.33	8.52×10^{-4}

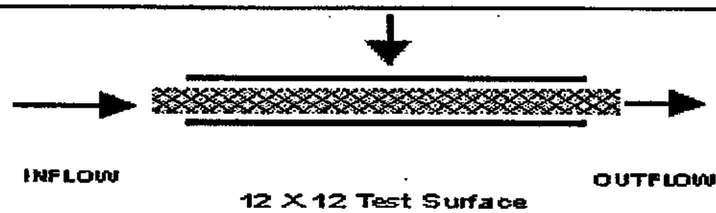
571 Industrial Parkway, Commerce, GA 30529 **Phone:** 706-336-7000 **Fax:** 706-336-7007 **Email:** skaps@skaps.com

SKAPS Industries

ASTM D 4716

Client:	Antana Linings, Inc.	**Job #**	1711
Project:	Richardson Hill Landfill, NY		
Product:	TN270-10/7		

Test Configuration:



Test Information:

Boundary Conditions:	Steel Plate Geocomposite Steel Plate	**Normal Load:** **Gradient:** **Seating Time:** **Flow Direction:**	350 psf 0.1 ft 15 minutes MD

Test Results:

Roll No.	Pressure (psf)	Gradient, ft	Transmissivity, m^2/sec 15 minutes
1711111	350	0.1	1.32×10^{-3}
1711121			1.40×10^{-3}
1711129			1.37×10^{-3}

571 Industrial Parkway, Commerce, GA 30529 **Phone:** 706-336-7000 **Fax:** 706-336-7007 **Email:** skaps@skaps.com

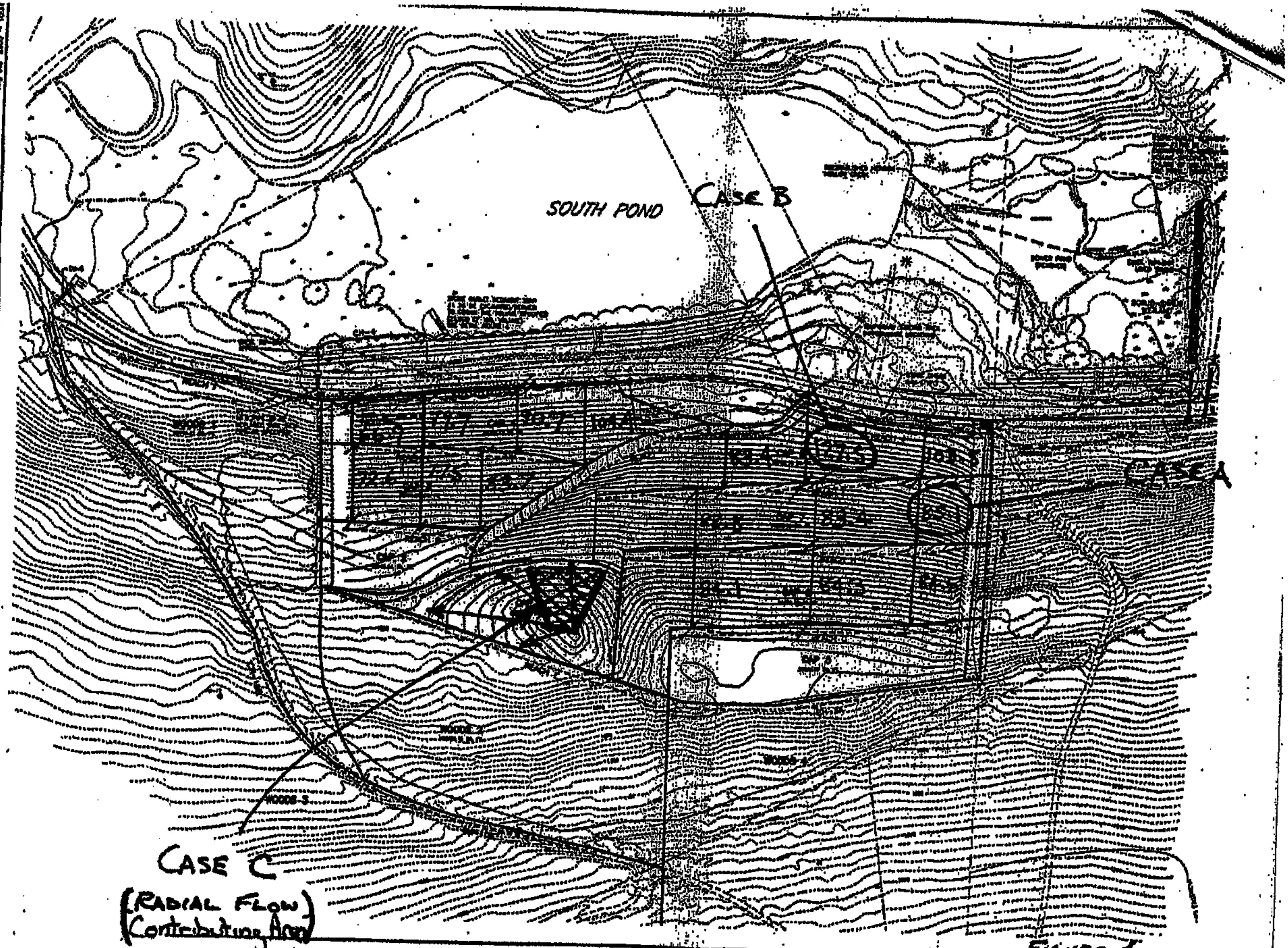
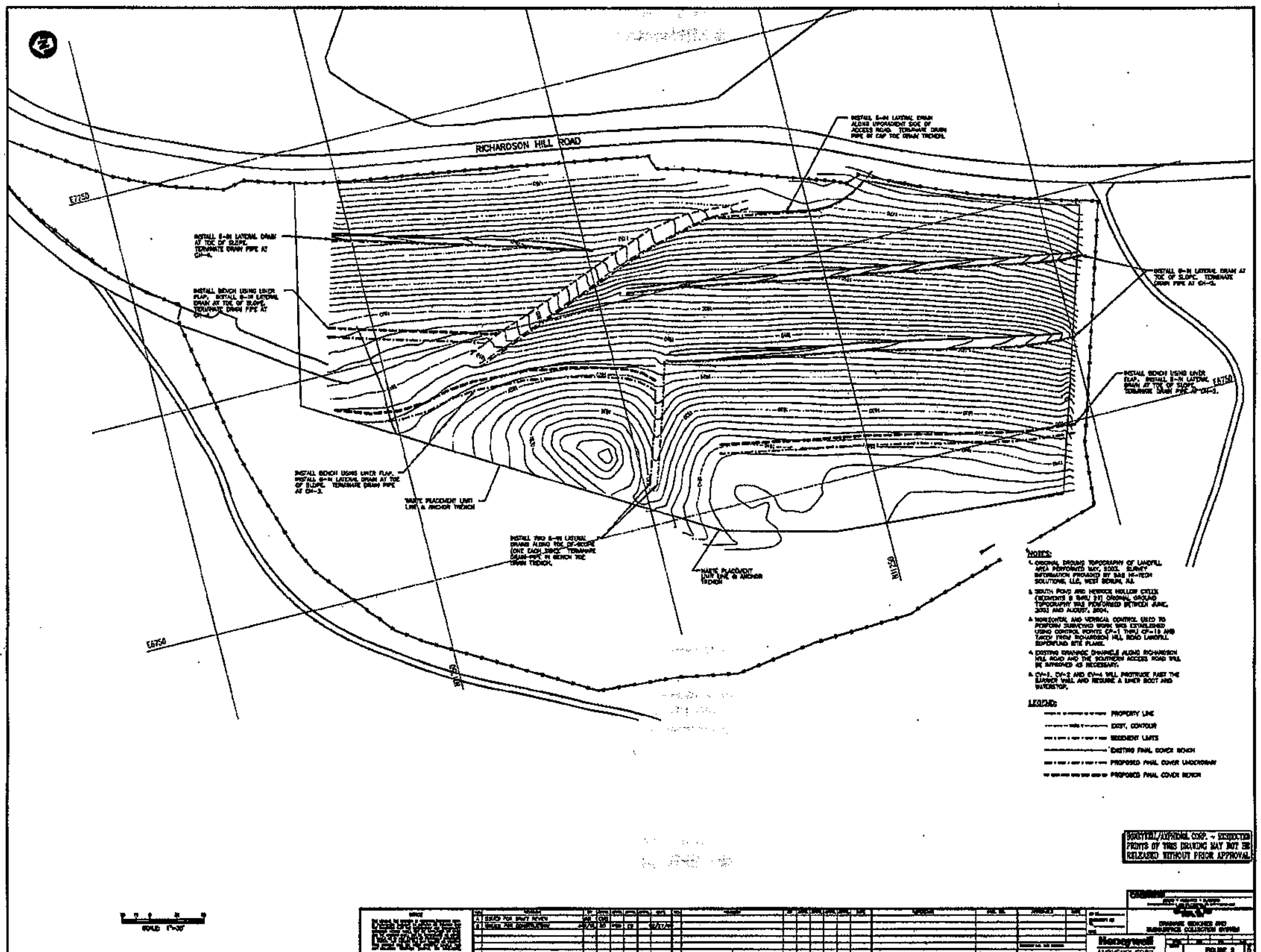
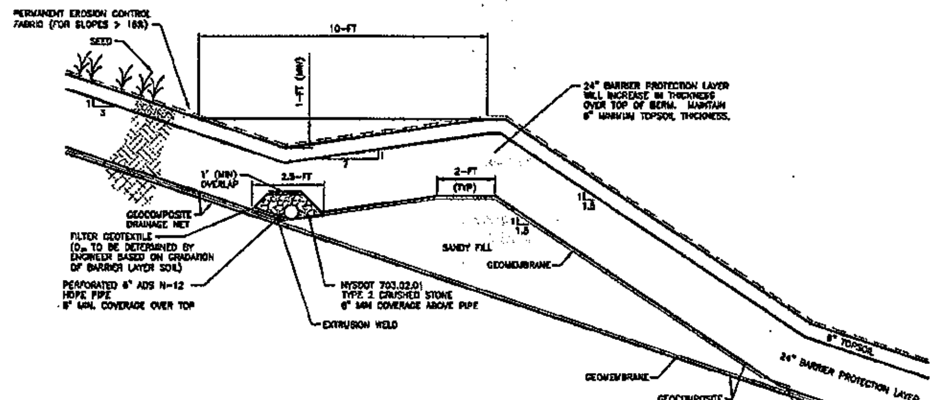

SOUTH POND
CASE B
CASE A
CASE C
(RADIAL FLOW)
(Contributing Area)

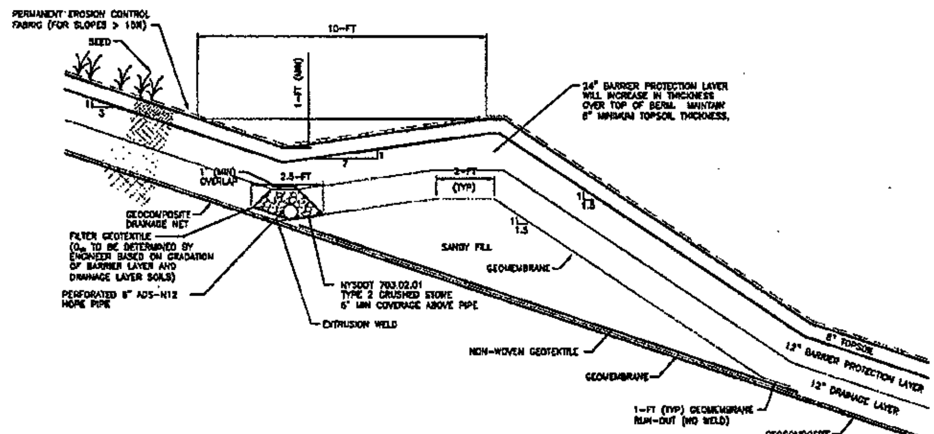
FIGURE 1


RICHARDSON HILL ROAD
E7250
E6750
N11250
INSTALL 8-IN LATERAL DRAIN ALONG UPGRADIENT SIDE OF ACCESS ROAD. TERMINATE DRAIN PIPE IN CAP TOE DRAIN TRENCH.
INSTALL 8-IN LATERAL DRAIN AT TOE OF SLOPE. TERMINATE DRAIN PIPE AT CH-2.
INSTALL BENCH USING LINER FLAP. INSTALL 8-IN LATERAL DRAIN AT TOE OF SLOPE. TERMINATE DRAIN PIPE AT CH-3.
INSTALL TWO 8-IN LATERAL DRAINS ALONG TOE OF SLOPE (ONE EACH SIDE). TERMINATE DRAIN PIPE IN BENCH TOE DRAIN TRENCH.
WASTE PLACEMENT LIMIT LINE & ANCHOR TRENCH
NOTES:
1. ORIGINAL GROUND TOPOGRAPHY OF LANDFILL AREA PERFORMED MAY, 2003. SURVEY INFORMATION PROVIDED BY SAI HI-TECH SOLUTIONS, LLC, WEST BERLIN, NJ.
2. SOUTH POND AND HEWRICK HOLLOW CREEK (SEGMENTS 8 THRU 31) ORIGINAL GROUND TOPOGRAPHY WAS PERFORMED BETWEEN JUNE, 2003 AND AUGUST, 2004.
3. HORIZONTAL AND VERTICAL CONTROL USED TO PERFORM SURVEYING WORK WAS ESTABLISHED USING CONTROL POINTS CP-1 THRU CP-10 AND TAKEN FROM RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE PLANS.
4. EXISTING DRAINAGE CHANNELS ALONG RICHARDSON HILL ROAD AND THE SOUTHERN ACCESS ROAD WILL BE IMPROVED AS NECESSARY.
5. CV-1, CV-2 AND CV-4 WILL PROTRUDE PAST THE BARRIER WALL AND REQUIRE A LINER BOOT AND WATERSTOP.
LEGEND:
PROPERTY LINE
EXIST. CONTOUR
SEGMENT LIMITS
EXISTING FINAL COVER BENCH
PROPOSED FINAL COVER UNDERDRAIN
PROPOSED FINAL COVER BENCH
PRINTS OF THIS DRAWING MAY NOT BE RELEASED WITHOUT PRIOR APPROVAL.
Honeywell
AMPHENOL CORP.
FIGURE 2



LINER FLAP WITH
GEOCOMPOSITE DRAINAGE LAYER

NOT USED



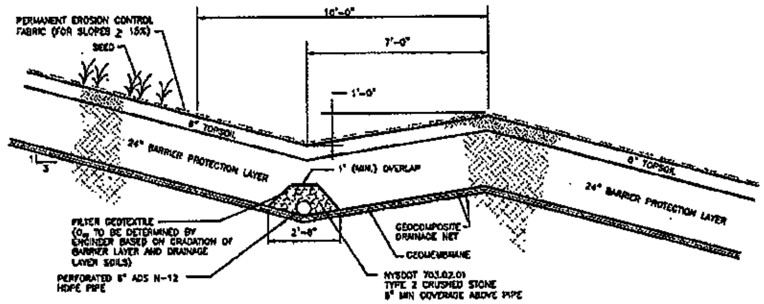
LINER FLAP WITH
SAND DRAINAGE LAYER

NOTES:
1. THE LINER FLAP DETAILS SHOULD BE USED AT NEW DRAINAGE BENCH LOCATIONS.

HONEYWELL/AMPHENOL CORP. - RESTRICTED PRINTS OF THIS DRAWING MAY NOT BE RELEASED WITHOUT PRIOR APPROVAL

NO.	REVISION						DATE
B	ISSUED FOR REVIEW						
0	ISSUED FOR CONSTRUCTION						6/?/27/06

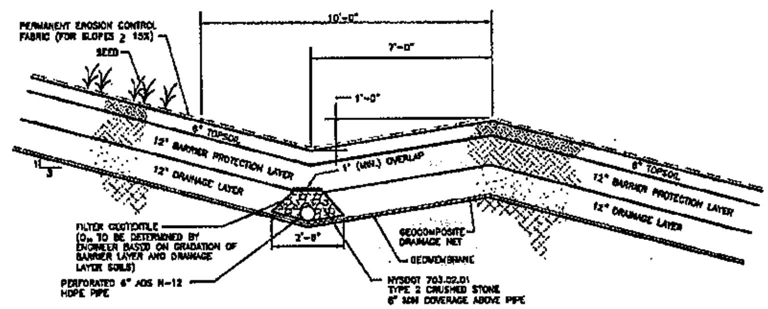
Honeywell
AMPHENOL CORP.
LINER FLAP DETAIL
FIGURE 3



LATERAL DRAIN WITH
GEOCOMPOSITE DRAINAGE LAYER

NOT USED

NOT USED



LATERAL DRAIN WITH
SAND DRAINAGE LAYER

HONEYWELL/AMPHENOL CORP. - RESTRICTED PRINTS OF THIS DRAWING MAY NOT BE RELEASED WITHOUT PRIOR APPROVAL

Honeywell AMPHENOL CORP.	LATERAL DRAIN DETAIL	FIGURE 4

Responses to Comments on
Field Change Order #009 Rev 5, Landfill Cap Changes for the
Richardson Hill Road Landfill (RHRL) Site

(Responses in Bold)

USEPA Comments (November 14, 2005):

General

Comment 1:
Page 2, Revised Cap Design, 2b, 2nd bullet: The specification for the geotextile affixed to the geosynthetic drainage composite has changed from a site-specific design to generic design (AASHTO M288-96). AASHTO M288-97, Section 7.2.3, states, "The property values in Table 2 (Subsurface Drainage Geotextile Requirements) represent default values which provide sufficient geotextile survivability under most construction conditions. Additionally footnote 4 for Table 2 states, "Site specific geotextile design should be performed especially if one or more of the following problematic soil environments are encountered: unstable or highly erodable soils such as non-cohesive silts; gap graded soils; alternating sand/silt laminated soils; dispersive clays; and/or rock flour." Please justify why the criteria was changed and state why this change will not increase risk relative to the previous site specific design.

Parsons Response:

Parsons does not consider AASHTO M288 a "generic design". AASHTO M288 criteria evaluate soil retention using site specific grain size analysis data and provide recommended geotextile properties. AASHTO M288 was developed in the 1980s and 1990s by a public-private consortium of federal and state transportation agencies, the geotextile manufacturers, academia, and consulting engineers to improve the performance of geotextiles in construction and to standardize the various types of geotextile into classes to allow for simpler and more accurate design and specification. . During this time, extensive and systematic field and laboratory testing was performed and documentation collected to support the AASHTO M288 requirements. The transportation industry is the primary user of geotextiles in the country and developed AASHTO M288 to provide a simple yet robust set of specifications that will function under many different construction and loading conditions in many different geologies. AASHTO M288 was first published in 1992 and was substantially revised in 1996. Changes since 1996, including the most recent revision in 2005, have been minor, indicating that this goal has been achieved.

More recently, the AASHTO M288 requirements have been used in design specifications of geotextiles in landfill cover applications. This approach is documented in several available references and design documents including:

- **Qian, Xuede, Gray, Donald, and Koerner M. Robert, "Geotechnical Aspects of Landfill Design and Construction", Prentice Hall, 2002.**

- **Koerner M, Robert, "Designing with Geosynthetics – 4th Edition", Prentice Hall, 1997.**
- **www.landfilldesign.com, Advanced Geotechnical Systems, 2005.**

The proposed barrier protection material is not considered an unstable or highly erodable soil as defined in AASHTO M288. Also the combination of slopes and protection of geomembrane systems generally results in lower equipment weights operating on the geotextiles than normally used in highway construction, so the survivability requirements in AASHTO M288 are typically conservative for landfill construction applications. Parsons believes the requirements set forth in AASHTO M288 are applicable to the RHRL landfill cap design.

Attachment B

Comment 1:
Although the calculations provided in Attachment B appear to demonstrate that a barrier protection soil (BPL) with a permeability of 1.0E-4 cm/sec works within the given design parameters, the effects of weather (frost action, soil loss due to erosion) and/or other physical factors (animal burrows, root growth) might increase the permeability of the soil. That is, the permeability of the BPL may be 1.0E-4 cm/sec right after placement and compaction (and this is theoretical, as the permeability of the material will be tested in a laboratory setting, and not in-situ), but the effects noted above may increase the permeability over time. Specifying a lower permeability BPL, e.g., less than 1.0E-5 cm/sec, would decrease long term risk.

Parsons Response:

Parsons believes that the approach used to design the barrier protection material provides an acceptable factor of safety against the long term risks of a drainage layer failure. Parsons acknowledges the concerns discussed above; however, Parsons does not believe that the overall permeability of the barrier protection material is going to change significantly. This is support by the following items:

- **The barrier protection layer has a thickness of 2 feet;**
- **The calculations do not incorporate the 6-inch topsoil layer, which typically has a lower permeability as noted in USEPA comment 3 for Attachment B, placed on top of the barrier protection layer.**
- **Increasing permeability due to environmental action is typically more of an issue with relatively low permeability soils (e.g. clays w/k<10^{-6} cm/sec) which rely on specific soil states to achieve these low permeabilities. Higher permeability soils tend to decrease permeability when roots and other elements intrude into the layers as often noted in the admonitions in USEPA landfill design and maintenance literature to prevent plant growth in drainage layers.**
- **Soil loss due to erosion is typically a short-term problem prior to the establishment of vegetation. Once vegetation is established, soil loss due to**

erosion should be minimal, especially in the barrier protection layer as discussed in the response in the next bullet,

- **Long-term soil loss was estimated and the results included in Attachment D of FCO #9 Rev 5. The calculations indicate that long term soil loss will not be a significant issue.**
- **A minimum factor of safety of 2.5 was used in the calculations which allows for some variability in the cap material over the long term.**
- **The O&M plan will recommend that the landfill cap be periodically inspected for soil loss, deep rooted vegetation (i.e. trees, shrubs, etc) and animal borrows. Repairs will be recommended to mitigate maintenance issues identified.**

Comment 2:

For Subcase 5C, the table in the middle of page 5 (preceding Summary of Results header) indicates a maximum q_h of 8.30E-7 m/sec (8.30E-5 cm/sec) is necessary to meet the factor of safety goal. A footnote to the table indicates that the radial condition reduces the contributing area of flow increasing the factor of safety. Please recalculate the factor of safety for Subcase 5C assuming radial flow and the specified permeability of 1.0E-4 cm/sec to verify that assumption.

Parsons Response:

The calculations to support the assumption in Subcase 5C have been incorporated into the drainage evaluation calculations. As stated in the original calculation, the radial condition results in a reduced contributing flow area, thereby increasing the factor of safety. The revised calculations are attached. (Attachment_B_Drainage_Eval_rev F.xls)

<u>**Specific Comments – Attachment B:**</u>

Comment 1:

Page 3, Factor of Safety table: Earth Tech disagrees with the selection of a reduction factor of 1.0 for RFimin. This factor should be 1.0 only if the test boundary conditions mimic field conditions, which is not the case here (steel plates were used in the lab; geotextile and soil will abut the drain in the field).

Parsons Response:

Parsons believes that the overall reduction factor (Π(RF)) calculated for the GDC of 3.0 is appropriate for the calculation and will result in an acceptable factor of safety. Π(RF) is a product sum of eight individual reduction factors. Reduction factors are typically selected using engineering judgment, site specific conditions, and guidance from published documents.

RFimin is defined by Giroud et al (Ref. 2) as: "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". Parsons believes that the laboratory load, which was greater than anticipated in the field, closely simulated how the geotextile would intrude into the geonet immediately following the application of the field load (i.e. barrier protection material and topsoil). It is acknowledged that an argument could be made to use a different value for RFimin or different values for any of the reduction factors based on various engineering opinions. Reduction factors at the upper end of published ranges (i.e. most conservative) were used for each remaining applicable factors and could arguable be reduced. Reducing any of these and increasing RFmin would produce a similar Π(RF) value resulting in an acceptable factor of safety. A summary of the reduction factors used in the calculation and published recommendations from Table 3.3 of Giroud et al (Ref 2) is presented below.

Application	Normal Stress	Liquid	RFin	RFcr	RFcc	RFbc
Landfill cover drainage layer recommend values	Low	Water	1.0 – 1.2	1.1 – 1.4	1.0 – 1.2	1.2 – 1.5
Parsons assumed values	Low	Water	1.2	1.4	1.2	1.5

Comment 2, Page 4, Case 5:
Please note the typographical error for SKAPS data submitted "09/07/2006" (assumed 2005 intended).

Parsons Response:

Acknowledged. The text has been edited.

Comment 3, Page 5, Case 5:

a. It appears that the permeability for topsoil has not been considered. It would seem prudent to include the effect of a low permeability soil at the surface to minimize infiltration.
b. Please note typographical error in text immediately preceding Summary of Results header, where the FOS for drainage length of 90 feet (assumed to be Case 5C) is "2.6" while table above text indicates 2.5. Please review final sentence of the paragraph in light of the previous edit.

Parsons Response

a. **Parsons acknowledges the comment and did not include the topsoil layer in this calculation realizing that the placement of a lower permeability topsoil would increase the overall factor of safety. Parsons believes the recommendations provided in Attachment B are appropriate and the calculation does not require modification.**
b. **Acknowledged. The text has been edited.**

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 010

Project Number: 742577 Date: 9/23/05

Construction Manager: Parsons
Remedial Action Contractor: DA Collins

You are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Change Description:

Leave the existing temporary construction access road bordering the west (uphill) perimeter of Richardson Hill Road Landfill from Northing N11000 on the south half to N11750 on the north half. The access road is composed of compacted drainage sand averaging 16 to 24 inches in thickness and approximately 20 to 24 feet in width. (Note that the drainage sand was placed on the landfill in 2004 and does not meet the specifications. The rest of the drainage sand was removed from the landfill in 2005.)

Reason for Change:

The purpose of this change is a value engineering decision to eliminate an unnecessary construction task and reduce additional damage to the underlying landfill liner through removal of the temporary access road.

Specific Requirements:

The existing grade on the south half drains in an easterly (downhill) and northeasterly direction without entrapment of water. The existing grade on the north half drains in a northeasterly direction away from the temporary access road without entrapment of water with exception of the west side of the TSCA cell from N11500 to N11620 which drains west toward the temporary access road.

Install geonet drainage fabric (on top of the liner) up to the east perimeter of the access road from N 1100 to N11750 in accordance with the design documents. To facilitate drainage, install 6-inch HDPE slotted drain piping from Northing N11250 to Reach E at approximately N11810. Wrap the drain piping with filter fabric to eliminate silting of the pipe. The temporary access road will remain and be integrated with the barrier protection material. Refer to attached field sketch FSK FCO-10 for details.

The combination of geonet drainage fabric and drainage piping will convey water that may percolate through the barrier protection layer from both the north and south ends of the landfill. This change will not compromise the design function of the landfill cap system.

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

Reference Drawings and Details:

1. Field Sketch FSK FCO-10 Rev 0
2. As-Built Subgrade Topography Landfill Area Dated 12/27/04 by B&B Hi-tech Solutions, Inc.
3. Drawing C-6 Rev 2B Subgrade Plan.
4. Drawing C-7 Rev 1 Final Grading Plan.
5. Drawing C-10, Rev 0, Detail G Cap Section West Side North Half.
6. Drawing C-10, Rev 0, Detail H Cap Section West Side South Half.

APPROVALS:

Parsons Representative

Name: William J. Long	Date:
Signature:	

DA Collins Representative

Name: Dave MacDougall	Date:
Signature:	

USEPA

Name: Young Chang	Date:
Signature:	

cc: Joe Bianchi - Amphenol
Frank Leming – JTM Associates
Jeff Hall - EarthTech

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

Reference Drawings and Details:

1. Field Sketch FSK FCO-10 Rev 0
2. As-Built Subgrade Topography Landfill Area Dated 12/27/04 by B&B Hi-tech Solutions, Inc.
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4. Drawing C-7 Rev 1 Final Grading Plan.
5. Drawing C-10, Rev 0, Detail G Cap Section West Side North Half.
6. Drawing C-10, Rev 0, Detail H Cap Section West Side South Half.

APPROVALS:

Parsons Representative

Name: William J. Long	Date: 1/30/06
Signature: William J. Long	

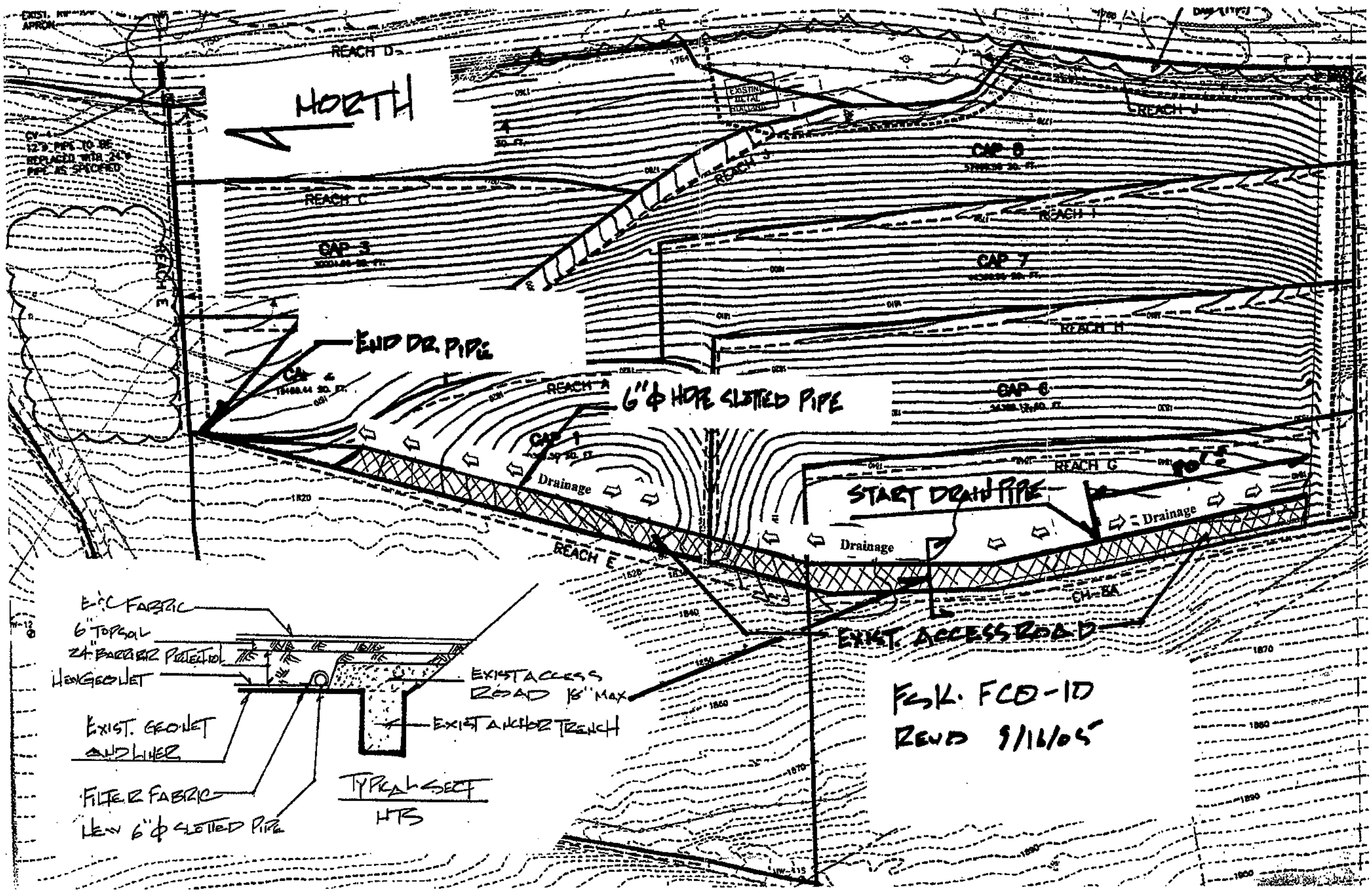
DA Collins Representative

Name: Dave MacDougall	Date: 1/16/06
Signature: [signature]	

USEPA

Name: Young Chang	Date: 2/23/06.
Signature: [signature]	

cc: Joe Bianchi - Amphenol
Frank Leming – JTM Associates
Jeff Hall - EarthTech


EXIST. RIP RAP APRON
REACH D
NORTH
CV-4 12"Ø PIPE TO BE REPLACED WITH 24"Ø PIPE AS SPECIFIED
EXISTING METAL BUILDING
REACH J
CAP 8
REACH C
REACH I
CAP 3
CAP 7
END DR. PIPE
REACH H
REACH A
6"Φ HDPE SLOTTED PIPE
CAP 6
CAP 1
Drainage
REACH G
START DRAIN PIPE
Drainage
Drainage
REACH E
CH-8A
EXIST. ACCESS ROAD
E/C FABRIC
6" TOPSOIL
24" BARRIER PROTECTION
NEW GEONET
EXIST ACCESS ROAD 18" MAX
EXIST. GEONET AND LINER
EXIST ANCHOR TRENCH
FILTER FABRIC
NEW 6"Φ SLOTTED PIPE
TYPICAL SECT NTS
FSK. FCD-1D
REVD 9/16/05

EARTHWORK REMEDIAL ACTION
RICHARDSON HILL ROAD LANDFILL SUPERFUND SITE
SIDNEY, NEW YORK

FIELD CHANGE FORM # 011

Project Number: 742577	Date: 7/31/06
Engineer: Parsons	
Remedial Action Contractor: DA Collins	

On behalf of Amphenol, you are hereby authorized and instructed to complete the following modifications to the approved Final Design:

Topsoil Specification (02990):

a. Revise Specification Section 02990 Part 2.01.A.2 and 2.01.A.3 as follows (Note: Modifications highlighted in **BOLD**):

2.01.A.2 – pH between 5.5 and 7.5;

2.01.A.3 – Shall contain not less than 3 percent organic matter nor more than 20 percent as determined by loss of ignition of moisture-free samples dried at 100° to 110° Celsius.

The source proposed for the landfill topsoil has organic contents ranging from 3.5 and 3.6 percent respectively based on recent laboratory results submitted by DAC (Submittal Pack-21, dated 7/26/06). Topsoil from this source, with organic contents greater than 3 percent, was successfully used at the Sidney Landfill nearby. For critical application, such as the landfill cover, recent experience has indicated that an organic content of 3.0 percent or greater and a pH of 5.5 are generally a minimum values that provide a good growing substrate. These values achieve the minimum recommended in standard NYSDOT Specifications (713-01)

APPROVALS:

Parsons Representative

Name: Raymond D'Hollander, P.E.	Date: 7/31/06
Signature: Raymond D'Hollander	

DA Collins Representative

Name: Dave MacDougall	Date: 8/1/06
Signature: [signature]	

USEPA

Name: Young Chang	Date: 8/2/06
Signature: D. A. Bott on behalf of EPA	

cc: Joe Bianchi - Amphenol

FCO011.doc
7/31/2006

A-2

FIELD MEMOS

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: John Waechter

Company: Shaw Environmental

From: Norm Sulock

Date: Wednesday, July 07, 2004

Reference: Richardson Hill Road Landfill Project

Subject: Field Change Request

This correspondence serves to document the approval of two change requests from Shaw Environmental for approval by Y. Chang of the USEPA's Region II office. They are:

1. Request approval to allow fall planting of wetland restoration in lieu of spring planting.
2. Amend the traffic plan to allow the transport of top soil from the south via Route 206 to Richardson Hill Road.

Item 1 was approved by Y. Chang on Tuesday July 6th.

Item 2 was approved by Y. Chang on Wednesday June 30th provided Shaw has obtained approval from local authorities.

A formal change order will not be issued for these amendments.

Cc
Young Chang USEPA Region II
Project File

Memo_03.doc

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: John Waechter

Company: Shaw Environmental

From: Norm Sulock

Date: Tuesday, July 13, 2004

Reference: Richardson Hill Road Landfill Project

Subject: Segment 16 Grids 16-4 and 16-5

Pursuant to my discussion with Jeff Gage on Monday July 12th , place the excavated spoil material from Segment 16 Grids 16-4 and 16-5 in the TSCA cell. The high analytical results of 47 and 150 ppm respectively for these grids necessitates disposal to this location.

Please advise should you have any questions.

Cc
J. Gage- Shaw
J. Hall Earth Tech
Project File

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: John Waechter

Company: Shaw Environmental

From: Norm Sulock

Date: Thursday, July 08, 2004

Reference: Richardson Hill Road Landfill Project

Subject: Erosion Control Fabric Waiver

Parsons received authorization from Young Chang this afternoon to install the erosion control fabric in 7.5-foot widths within the narrow Segments 17 through 14. Resume the 10-foot wide coverage in Segments 13 through 10.

Please advise should you have any questions.

Cc
Young Chang USEPA Region II
J. Hall Earth Tech
Project File

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: John Waechter

Company: Shaw Environmental

From: Norm Sulock

Date: Monday, July 19, 20044

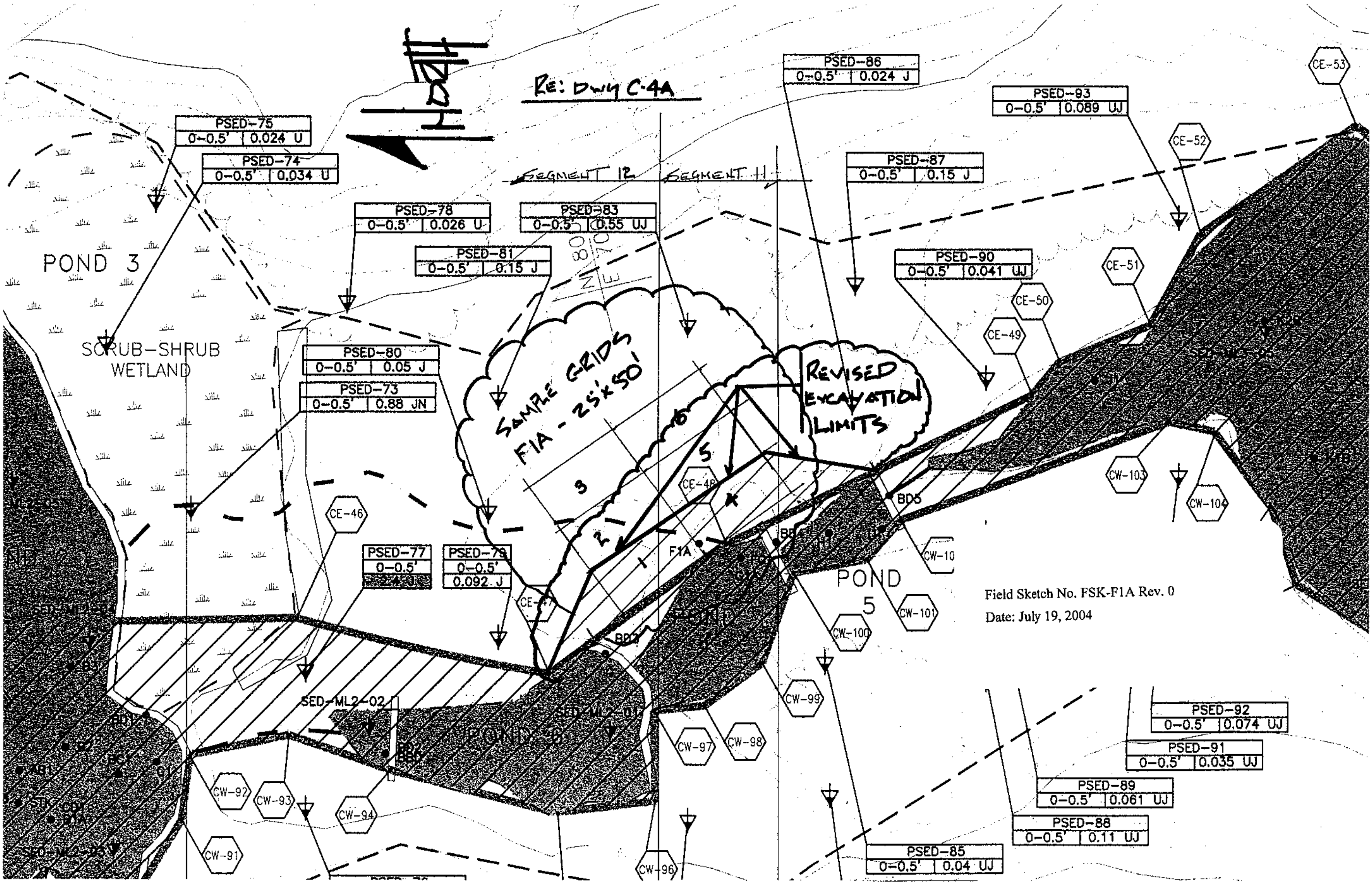
Reference: Richardson Hill Road Landfill Project

Subject: Project Change Notice

Soil sample results from grids 1 and 4 within the proximity of sediment sample location F1A indicated PCB contamination of 1.1 ppm respectively. Parsons has been instructed to extend a portion of the excavation on the east side of Segments 11 and 12 as indicated in the attached field sketch FSK-F1A Rev. 0. The additional excavation area totals approximately 407 square yards. Reimbursement for this work will be in accordance Section 1025 Measurement and Payment Item 11.

Please advise should you have any questions.

Cc w/ Attachments
Young Chang USEPA Region II
F. Leming Honeywell
J. Gage Shaw Environmental
J. Hall Earth Tech
Project File



Field Sketch No. FSK-F1A Rev. 0

Date: July 19, 2004

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: Bill Long

Company: Parsons

From: Norm Sulock

Date: August 4, 2004

Reference: RHRL Project

Subject: Zone 4 (Segments 9 through 13)

Please clarify the following items for the above subject matter:

1. Measurement and Payment Section 01025-9 Item 26A paragraph 3 specifies payment requirements for Segments 14 through 20. See attached file 01025-9.tif. What is the proposed method of payment for Zone 4 as shown on drawing C-8A Rev A? **Same as for Zones 1-3 – area of excavation unless sediment depth varies from anticipated. (Item 26A of the bid schedule should have stated the anticipated sediment depths, but it was included in Addendum #2.** Has an excavation depth been established? **Yes. See Addendum #2 – 24" in ponds and 12" in floodplain.**
   My experience to date with the upstream segments indicates a volumetric measurement would be appropriate however the pricing must be agreed upon. **A price adjustment may be warranted for quantities greater than that anticipated.**

2. There is no callout to back fill Segments 9 through 13 (Zone 4) as required in the upstream segments. Is it intended to backfill Zone 4 to pre-excavation grade or top soil and seed the excavated subgrade? **Topsoil and seed the excavated subgrade. As per the Parsons January 16, 2003 letter included in Addendum #2, 3rd paragraph, last sentence: "Similar to the upstream ponds, the downstream ponds will not be backfilled to original grade to allow for the establishment of open water habitat." If you look on C-4A, you will see that almost the entire excavation area is ponds. The short stretch of segment 13 upstream of the first pond (~120') should probably be backfilled with fill and topsoil as per Zone 3 if it is presently not part of a pond. (Keep in mind the size of these ponds has been changed since the design due to beaver activity.)**

These segments are approaching the excavation window Shaw needs guidance on how to proceed.

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: Young Chang

Company: USEPA Region II

From: Norm Sulock

Date: September 22, 2004

Reference: RHRL Landfill Project

Subject: Creek Bedding Stone

Recent storm events that occurred during the period of September 17th through September 18th eroded the creek bedding stone in restored Segments 13 through 16. Parsons recommends the creek bedding stone currently specified on drawing C-8 as median 1-inch cobble should be increased to the sizes used for the cross vanes. The cross vane cobble is natural water worn rock ranging from 2 to 12 inches in diameter. The increased size and weight of this material will be less susceptible to displacement.

The proposed application of this material will be used to repair the eroded channel in Segments 16 through 13, and restoration of the channel in remaining Segments 12 through 9. Please advise if this recommendation is acceptable to your office.

Cc

Project File

PARSONS

290 Elwood Davis Road Liverpool, NY 13088 Phone (315) 451-9560 Fax (315) 451-9570

MEMORANDUM

To: John Waechter

Company: Shaw Environmental

From: Norm Sulock

Date: October 20, 2004

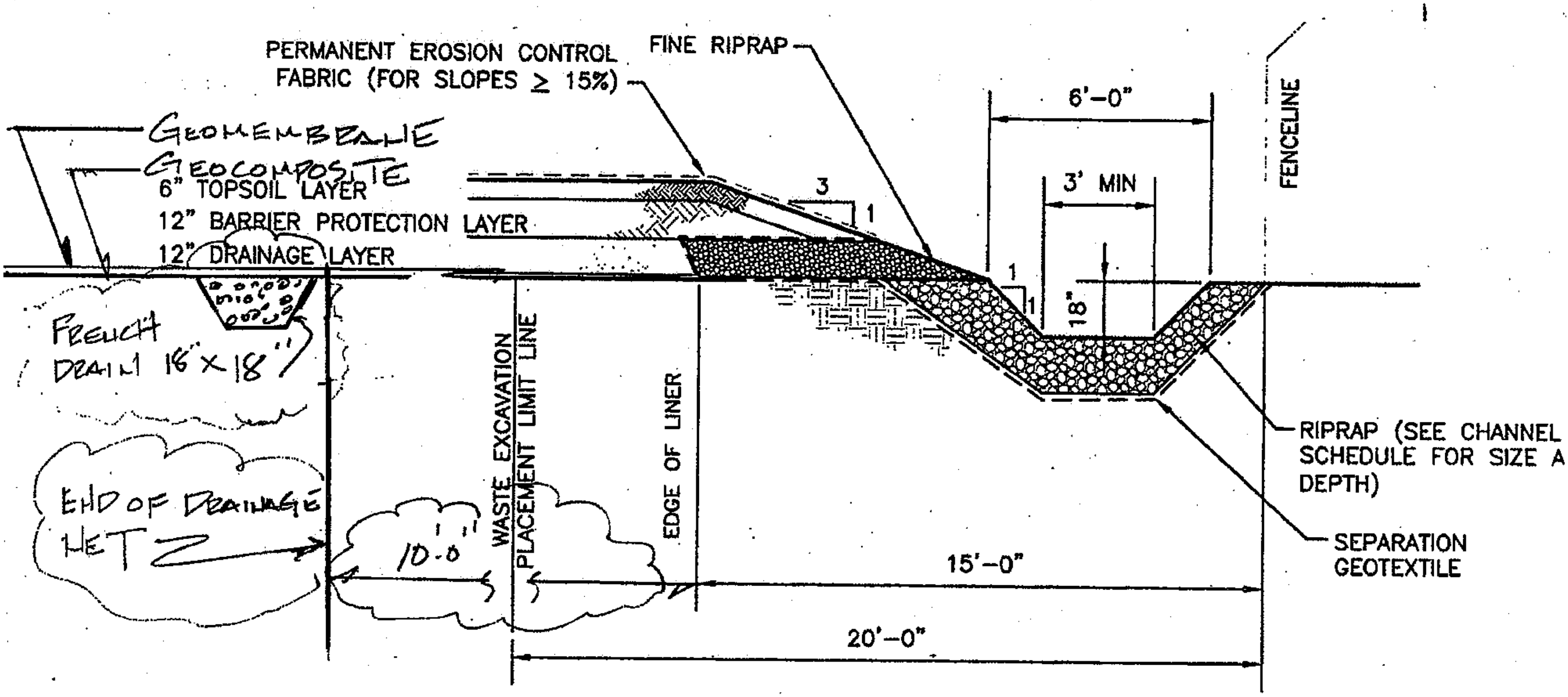
Reference: Richardson Hill Road Landfill Project

Subject: Geocomposite Drainage Net

Terminate the end of the Geocomposite drainage net ten feet from the end of the liner on the north and south ends of the landfill per the attached field sketch FSK-2 Rev 1. The purpose of this modification is to eliminate the possibility of leakage from under the liner at the ends of the horizontal drainage swales.

Please advise should you have any questions.

Cc w/Attachments
Y. Chang USEPA
J. Hall Earth Tech
Project File



C
C-6,7

TYPICAL CAP SECTION (NORTH AND SOUTH SIDES)

NOT TO SCALE

Notes

1. Terminate Geocomposite drainage net 10-Ft. from the end of the liner to eliminate possible end seeps.
2. Install an 18"x18" French Drain across each end and center of the five drainage swales to channel flow down the slope
3. Use existing onsite clean stone for the French Drain.

Field Change Notice
FSK-2 Rev 1 10/20/04

Long, William

From: O'Loughlin, James
Sent: Friday, June 23, 2006 1:33 PM
To: daniel.bennett@earthtech.com
Cc: 'Chang.Young@epamail.epa.gov'; Gerard Burke; Kaczor, James; jbianchi@amphenol-aao.com; Sam Waldo; Galloway, Rich; D'Hollander, Ray; Greene, Chris
Subject: FW: RHLF Toe Detail Clarification

Attachments: RHLF_Toe Detail.pdf

Dan,
Dan,

RHLF_Toe Detail.pdf

Clarification to the toe drain detail to address current as-built conditions. Please call Chris Greene if any questions.

Regards,

Jim

From: O'Loughlin, James
Sent: Friday, June 23, 2006 10:49 AM
To: jbianchi@amphenol-aao.com
Cc: Sam Waldo; Galloway, Rich; Greene, Chris; D'Hollander, Ray; 'David MacDougall'; Prohaska, Ronald
Subject: FW: RHLF Toe Detail Clarification

Joe,

Ron has reviewed this clarification w/ Dean and he is fine w/ it. We will transmit later today to Earthtech (cc EPA, DEC) for their review. Please let me know if you have any questions or comments.

Regards,

Jim

From: Greene, Chris
Sent: Friday, June 23, 2006 9:53 AM
To: O'Loughlin, James
Subject: FW: RHLF Toe Detail Clarification

Jim -

Attached is a sketch to clarify Detail D (Toe Detail) on Drawing C-10 based on the as-built configuration slopes at the toe landfill. I talked to Dan about it and he seems on board with it and doesn't see any problem with it. He would still like to take a quick look at the sketch. If you have any questions please let me know.

Chris

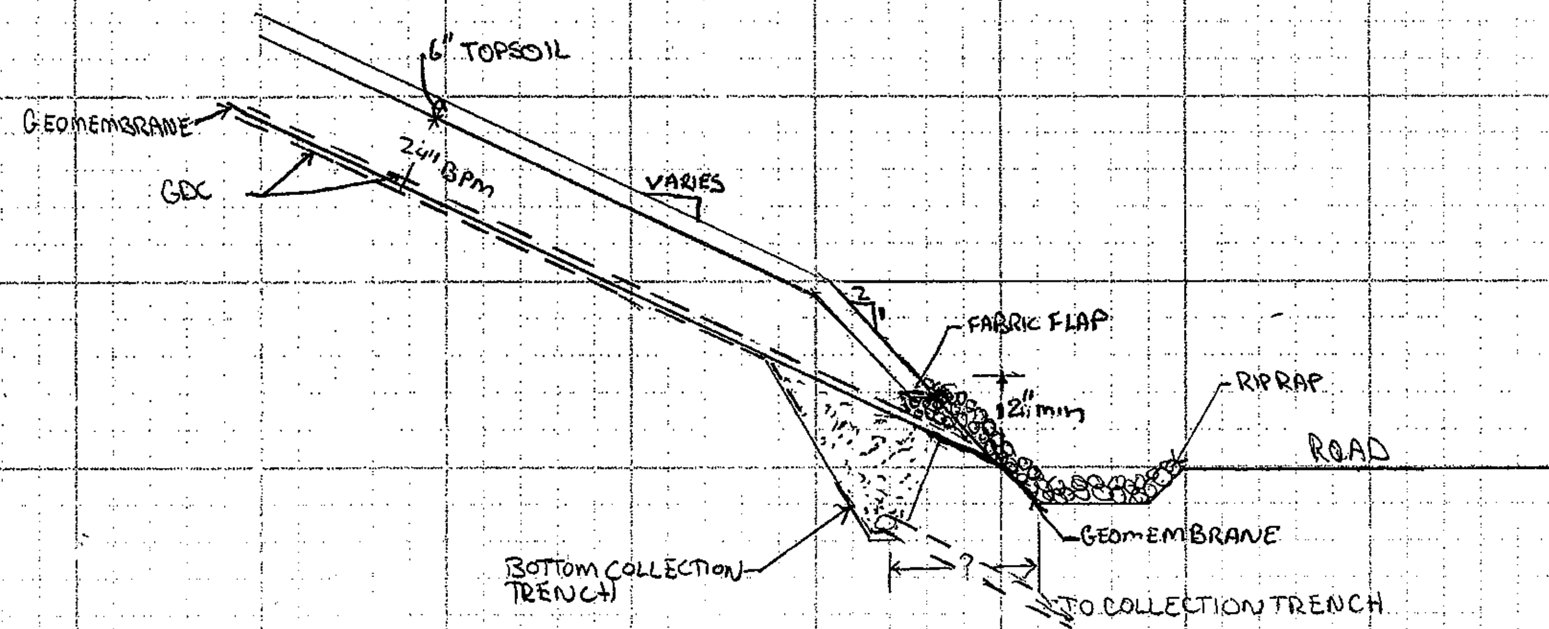
Chris Greene, PE
Environmental/Geotechnical Engineer

PARSONS
150 Federal Street
4th Floor
Boston, MA 02110
617-449-1573
617-946-9777 Fax
chris.greene@parsons.com

PARSONS

SUBJECT RHLF Toe Detail Clarification JOB NO. 742577
SHEET NO. ____ OF ____
MADE BY CMG DATE 6/22/06 CHECKED BY ROD DATE 6/23/06

TOE ANCHOR DETAIL (NTS)



NOTES:

1) ANCHOR GEOMEMBRANE UNDER RIP-RAP

2) SEPARATE 7oz FABRIC WHERE GDC TIES INTO RIP-RAP TO ALLOW FREE FLOW FROM GEONET TO SWALE

3) INSTALL PERMANENT EROSION CONTOL FABRIC FOR SLOPES ≥ 15%.

4) SWALE DIMENSIONS AND COLLECTION TRENCH DIMENSIONS AS SHOWN ON DETAIL D ON C-10.

From: O'Loughlin, James
Sent: Wednesday, July 05, 2006 3:39 PM
To: 'Bennett, Daniel'
Cc: Greene, Chris; D'Hollander, Ray; 'Kaczor, James'; Prohaska, Ronald; 'Chang.Young@epamail.epa.gov'; 'Gerard Burke'; 'David MacDougall'
Subject: RE: Revised clarification on barrier protection soil gradation

Dan,

Parsons has reviewed your request for clarification in light of the GDC planned for installation and provides the following revised response (Revised text underlined)

Field Change Order #009 provided a change in the hydraulic conductivity of the Barrier Protection Material (Specification 02260.2.02) to a maximum value of 1 x 10-4 cm/s. In effect, this modification superseded the gradation specification for the material. The gradation needed to achieve the new hydraulic conductivity value needs to have greater fines and/or a broader gradation. The broader gradation can include both the increased fines and larger particles than the original specified values. The barrier protection material for which DA Collins has provided the submittals meets the revised hydraulic conductivities but exceeds the percentage of materials passing the No. 200 sieve. It also has some materials as large as 2-inches. In our opinion, this material meets the intent of the design and will do so as long as it meets the maximum hydraulic conductivity values, the direct shear strength values, and does not have particles greater than <u>2-inches</u> in size.

Please contact me or Ray D'Hollander if we can provide further clarification.

Regards,

Jim

From: O'Loughlin, James
Sent: Wednesday, July 05, 2006 9:27 AM
To: 'Bennett, Daniel'
Cc: Greene, Chris; D'Hollander, Ray; Kaczor, James; Prohaska, Ronald; 'Chang.Young@epamail.epa.gov'; Gerard Burke; David MacDougall
Subject: RE: clarification on barrier protection soil gradation

Dan,

Parsons has reviewed your request for clarification and provides the following response:

Field Change Order #009 provided a change in the hydraulic conductivity of the Barrier Protection Material (Specification 02260.2.02) to a maximum value of 1 x 10-4 cm/s. In effect, this modification superseded the gradation specification for the material. The gradation needed to achieve the new hydraulic conductivity value needs to have greater fines and/or a broader gradation. The broader gradation can include both the increased fines and larger particles than the original specified values. The barrier protection material for which DA Collins has provided the submittals meets the revised hydraulic conductivities but exceeds the percentage of materials passing the No. 200 sieve. It also has some materials as large as 2-inches. In our opinion, this material meets the intent of the design and will do so as long as it meets the maximum hydraulic conductivity values, the direct shear strength values, and does not have particles greater than 4-inches in size.

Please contact me or Ray D'Hollander if we can provide further clarification.

Regards,

Jim

From: Bennett, Daniel [mailto:Daniel.Bennett@earthtech.com]
Sent: Thursday, June 29, 2006 1:59 PM
To: O'Loughlin, James
Cc: Greene, Chris; D'Hollander, Ray; Kaczor, James
Subject: clarification on barrier protection soil gradation

FCO #009 has a modified specification for maximum hydraulic conductivity, but does not mention anything about gradation. Specification 02260 has a gradation for barrier protection soil found in Section 2.02, Item 2. We interpret this to mean that Section 2.02, Item 2 remains valid. Is this correct? We ask because the recent BPM gradations do not all appear to meet the 100% passing the ¾-inch sieve criterion.

You can contact me at my Amherst, NY phone number if you have any questions.

Thanks,
Dan

Daniel A. Bennett
Environmental Engineer

Earth Tech, *A **tyco** INTERNATIONAL LTD. COMPANY*
100 Corporate Parkway
Suite 341
Amherst, NY 14226
716.836.4506 ext. 17 (phone)
716.834.8785 (fax)
daniel.bennett@earthtech.com

Long, William

From: O'Loughlin, James
Sent: Wednesday, July 05, 2006 8:48 AM
To: Bennett, Daniel; David MacDougall
Cc: 'Chang.Young@epamail.epa.gov'; Gerard Burke; Kaczor, James; 'waldo@amphenol.com'; 'jbianchi@amphenol-aao.com'; Galloway, Rich ; D'Hollander, Ray; Greene, Chris; Prohaska, Ronald
Subject: RHRL Drainage Swale Clarifications

Attachments: Benches_7_3.doc

Dan and Dave,

On behalf of Amphenol, Parsons has evaluated the recent as-built survey of current swale conditions at the site by Lawson Engineering subcontracted to DA Collins. We have performed a drainage capacity check as shown in the attached calculation and found that the swale capacity included in the original design provides sufficient capacity for the currently

Benches_7_3.doc

proposed final geometry at the site with an adequate margin of safety. We note that most of the as-built geometries of the swales meet or are close to the design swale geometry. We have provided some clarifications and guidance on interpreting the current approved design details as summarized below:

1. The swales should have a minimum hydraulic cross-sectional area of 5.0 feet which matches the hydraulic cross-sectional area of the original design detail.
2. The swale depth should be a minimum of 1 foot for both the geomembrane and final soil cover shapes.
3. We have set an internal limit of 2H:1V for the final swale interior slopes so that they can remain stable and vegetated.
4. Parsons recognizes that the first 100 feet of the swale are a transition from the normal slope to a bench swale configuration (typically in the middle of the landfill) and the swale will only be carrying a small percentage of the design flow in that stretch. The swale shape should have a minimum cross-sectional area of 2.0 feet and a height of 1 foot in this transition zone.
5. A geomembrane flap can be attached with an extrusion weld to augment the existing geomembrane shape where the existing geomembrane swale shape does not have the required 1 foot of height.
6. The swales should have a minimum slope of 1%.
7. The General Note on Drawing C-2 is clarified to make it clear that the intent of the rip-rap is to address the relatively steeply slope transition from the swale on the full barrier protection material thickness to the no barrier protection soil at the downchute swale. Latitude is provided for shortening or lengthening the transition as required by actual field geometries.

Please feel free to contact me or Ray D'Hollander if you have any questions concerning the calculations or the clarifications.

Regards,

Jim

James M. O'Loughlin
Project Manager

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(315) 451-9570 (fax)
(617) 279-3436 (cell)

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Direct: 617-449-1563
Main: 617-946-9400
Fax: 617-946-9777
(617) 279-3436 (cell)

SAFETY - MAKE IT PERSONAL

1.0 PURPOSE

To check the capacity and configurations of the drainage benches installed at the Richardson Hill Landfill Site and evaluate the as-built configurations for sizing to accommodate the anticipated run-off of a 25-year, 24-hour storm event.

2.0 ASSUMPTION

The following assumptions were made to support the calculations.

1. The design storm event will have a return period of 25 years and a duration of 24 hours.

2. A Manning's roughness coefficient of 0.030 similar to the one used as used in the O'Brien & Gere SWPP is applicable to the final grass surfaces of the benches at the RHLF site.

3. Stormwater calculations are for the completed design conditions.

3.0 APPROACH

The calculations were performed to address drainage benches on the landfill cap including Reach A, B, C, G, H, and I. Reaches J, E, and 8A will be addressed under a separate cover.

The configuration of the swales, slopes, and drainage areas were obtained from Lawson Surveying and Mapping of Schenectady, New York on June 1, 2006.

The peak rainfall for the 25 year return period for the landfill areas were obtained from Table 1-3 and Table 1-4 of O'Brien and Gere's, *Stormwater Pollution Prevention Plan,* dated May 2005 (Revised August 3, 2005).

The drainage swale capacities were checked and sized using Manning's equation ($Q = (1.49/n)*A*R^{2/3}*S^{1/2}$)to check flow capacities of "V" shaped channels.

Manning's equation was used to calculate the swale height. Manning's equation was solved first for all the knowns of the formula (Q, n, and S) which is equal to A(5/3)/P(2/3) and can be written as Q*(n/1.49)*S(-1/2)=A(5/3)/P(2/3). The first half of this equation was used to solve for A(5/3)/P(2/3). Next H was solved for using the equation:

H= [([Q*(n/1.49)*S(-1/2)]*(3.64+(1+B2)(1/2))(2/3))/(1.75+B/2)(5/3))](3/8).

The information on the peak flows and slopes of the benches are summarized below in Table 1.

Table 1 - Bench Information

O'Brien and Gere Report SWPP					
Area name	Peak Flow (25 year storm) (cfs)2	Reach	Swale Area per foot (ft^2)	Slope (%)	Slope (%)
Woods 4	33.22	CH-8A	See below	9.0%	NA[3]
Woods 2	7.9	E	See below	3.0%	NA[3]
Cap 8	4.8	J	See below	7.2%	13.0%
Cap 6	4.7	H	See below	3.6%	3.6%
Cap 7	3.7	I	See below	4.2%	4.7%
Cap 3	2.52	C	See below	2.7%	2.7%
Cap 5	2.89	G	See below	2.9%	3.0%
Cap 4	2.86	D	See below	1.0%	1.0%
Cap 1	2.54	A	See below	3.5%	3.1%
Cap 2	1.37	B	See below	4.5%	6.3%

Note

1. Swale area per foot estimated from Detail E on Design Drawing C-10. Assumed upgradient slope of 3.5 horizontal to 1 vertical.
2. Peak flow is from Table 1-3 of the O'Brien and Gere SWPP Report for the landfill covered with grass.
3. NA not included in the 2006 survey.

4.0 CALCULATIONS

To check the flow capacity of the reaches it was assumed that the upgradient side of the reach at a slope of 3.5 horizontal to 1 vertical which is conservative for the Reach D (Cap Area 4) condition used for the comparison due to its flat gradient. The conservatism of this evaluation is enhanced by comparing the capacity to the maximum expected flow for any swale on the landfill itself (Reach H below Cap Area 6). The down slope side of the reach was varied to check the capacities. The results are summarized below in Table 2.

Table 2 - Capacity Check of Cap Area Benches

As-built swales		Peak Flow (25 year storm) (cfs)[1]	Slope[2]	$A^{5/3}/P^{2/3}$	Swale (water) Height	Swale (water) Area per foot (ft^2)	Check back to Q
H	V						
2	1	4.7	1.0%	0.946	0.81	1.81	4.70
3	1	4.7	1.0%	0.946	0.76	1.86	4.70
4	1	4.7	1.0%	0.946	0.72	1.92	4.70
5	1	4.7	1.0%	0.946	0.68	1.98	4.70
6	1	4.7	1.0%	0.946	0.65	2.03	4.70
7	1	4.7	1.0%	0.946	0.63	2.07	4.70

Notes:
1. The peak flow used is from Cap Area 6 (Reach H)
2. The minimum slope is from Cap Area 4 (Reach D).

Based on the calculations presented above, the bench configurations evaluated have sufficient capacity for a 25-year, 24-hour storm event if they are a minimum of one foot in height.

5.0 CONCLUSIONS

1. The original Parsons swale design detail (E/C-6,7 on Sheet C-10 as modified in FCO #009) with a cross-sectional area of 5.0 feet to carry a 25-year, 24-hour storm flow of 3.3 cfs was supported by calculations for a flow capacity of 5.8 cfs in a swale with a depth of 1 foot, a minimum cross-sectional area of 1.1 square feet, a Manning "n" value of 0.33, and a minimum swale slope of 5%.

2. The subsequent re-evaluations of actual as-built conditions by O'Brien & Gere and Parsons show similar results with a maximum swale area of 2.1 square feet required for a conservative combination of flows and slopes. Parsons recommends that the 5.0 square foot area from the original design be maintained for the swales along with the minimum height of 1-foot to have sufficient capacity with an adequate safety factor for a 25-year, 24-hour storm event. Parsons recognizes that the geometry of the swale cross-section may vary from the original design detail. In our opinion, this is acceptable provided the 5.0 square foot area and 1-foot height are maintained except as indicated in paragraph 4 below.

3. To maintain sideslope stability and allow for good vegetation growth, the maximum acceptable finished slope of a swale side-slope is 2H:1V. where the geomembrane is currently steeper than that, the sidelsope can be built out with soil to achieve the 2H:1V soil slope.

4. Parsons expects that the beginning 100-feet of the swales (middle of the landfill) where the flows will be low may need to be transition zones from the adjacent slope geometry to the full swale size. The minimum acceptable geometry in this transition zone is a cross-sectional area of 2.0 square feet and a height of 1 foot.

5. Where the existing swales do not have a geomembrane peak height of 1-foot, the existing geomembrane can be augmented by a geomembrane flap attached with an extrusion weld.

6. The landfill swales should have a minimum slope of 1 percent towards the outlet swales. Low areas where ponding is observed should be repaired.

7. General Note 4 on Drawing C-2 states that the final 10-feet of the swales will have rip-rap placed over the permanent erosion control mat. The intent of this note is to provide for rip-rap on the sloping transition from the higher ground on top of the barrier protection material and the topsoil to the lower downchute swale. It is expected that this transition will be much steeper than the typical swale slope to that point. It is possible that this transition may be accomplished in a shorter or longer distance than 10 feet and the rip-rap should be placed accordingly. In any event, the swales and pipes daylighting from the swales must be free-flowing to the downchutes.

8. If conditions encountered differ from those anticipated or assumed, the conclusions and recommendations presented herein should be re-evaluated and modified or confirmed in writing.

6.0 REFERENCES:

1. United States Department of Agriculture Soil Conservation Service (USDASCS), 1986, *Urban Hydrology for Small Watersheds,* Technical Release 55

From:	O'Loughlin, James
Sent:	Friday, July 14, 2006 3:53 PM
To:	'Bennett, Daniel'; dmacdougall@dacollins.com
Cc:	'Kaczor, James'; 'Chang.Young@epamail.epa.gov'; 'jbianchi@amphenol-aao.com'; 'Sam Waldo'; 'Galloway, Rich'; Greene, Chris; D'Hollander, Ray
Subject:	Geotextile Overlap Clarification
Attachments:	20060714145959.pdf

Dan / Dave,

On behalf of Amphenol, please find attached clarifications to details from Figures 3 and 4 of FCO #009. The clarifications are summarized as follows:

- To promote free flow at the discharge point of the GDC, remove sufficient 7-oz fabric to allow the geonet to be in contact with the perforated 6" ADS N-12 HDPE Pipe.
- To prevent the migration of fines in the geonet place sufficient filter geotextile to separate the BPM from the Type 2 Crushed stone. The filter geotextile shall extent a minimum of 2 feet past where the 7-oz fabric is removed or peeled back.

Please contact Ray D'Hollander or Chris Green if you have any questions concerning this clarification.

Regards,

Jim

20060714145959.pdf

James M. O'Loughlin
Project Manager

PARSONS

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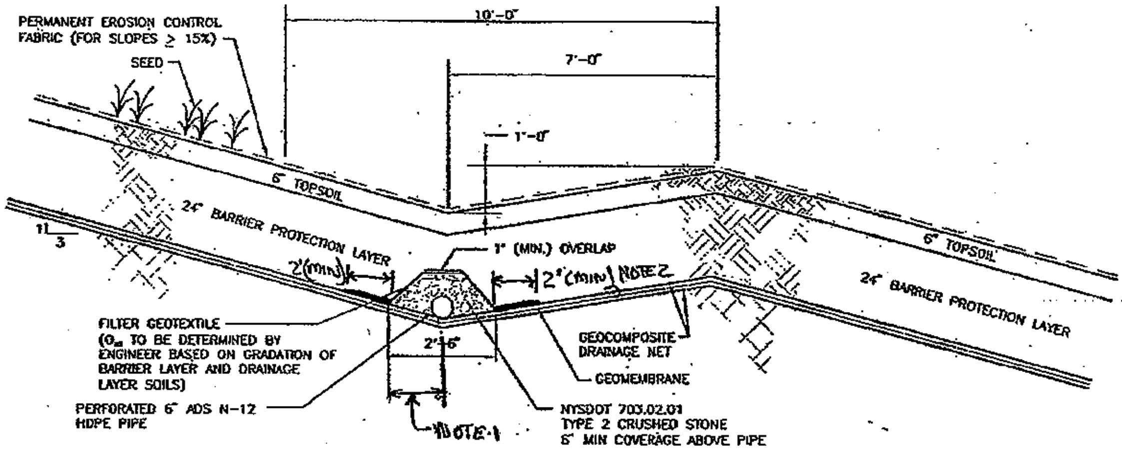
SAFETY - MAKE IT PERSONAL

PARSONS

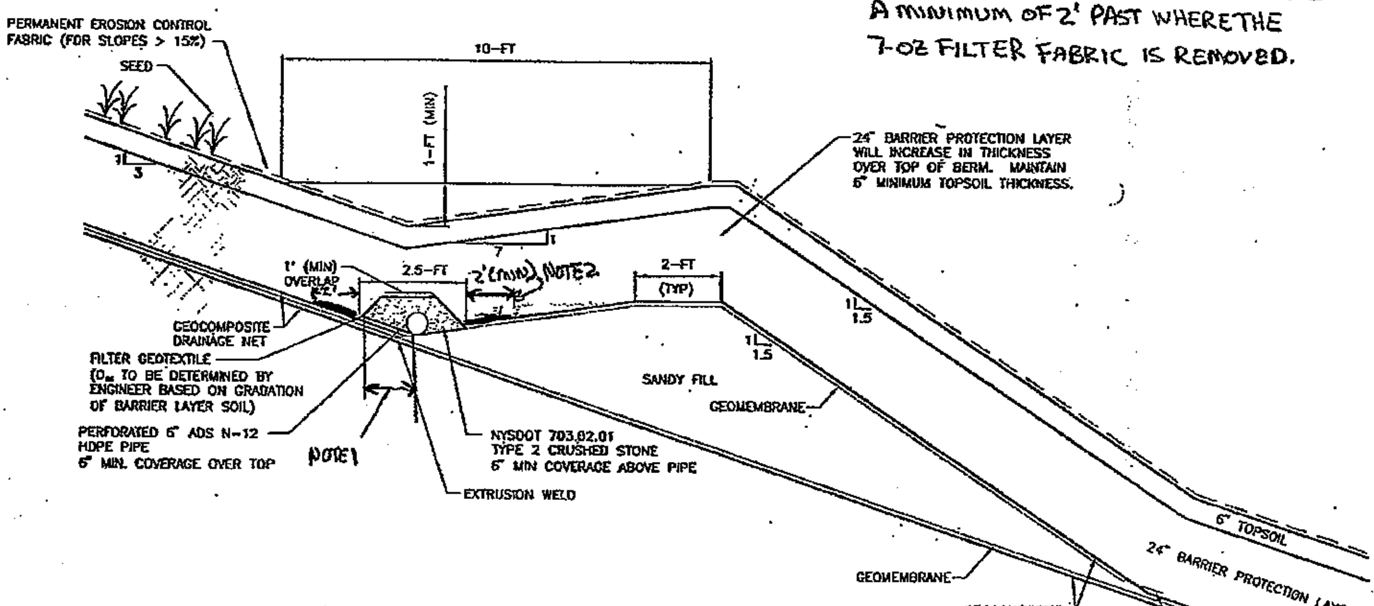
SUBJECT RHLF
Geotextile Overlap Clarification
JOB NO. 742577
SHEET NO. ____ OF ____
MADE BY CMG DATE 7/19/06 CHECKED BY CMG RDK DATE 7/14/06



LATERAL DRAIN WITH
GEOCOMPOSITE DRAINAGE LAYER

NOTES

1. 7-OZ FILTER FABRIC SHALL BE REMOVED TO ALLOW CONTACT BETWEEN THE GEONET AND 6" ADS N-12 HDPE PIPE
2. THE FILTER GEOTEXTILE SHALL EXTEND A MINIMUM OF 2' PAST WHERE THE 7-OZ FILTER FABRIC IS REMOVED.



LINER FLAP WITH
GEOCOMPOSITE DRAINAGE LAYER

Rev. 1/92

Long, William

From: O'Loughlin, James
Sent: Monday, July 17, 2006 10:44 AM
To: 'Bennett, Daniel'; dmacdougall@dacollins.com
Cc: 'Kaczor, James'; 'Chang.Young@epamail.epa.gov'; 'jbianchi@amphenol-aao.com'; Sam Waldo; 'Galloway, Rich'; Greene, Chris; D'Hollander, Ray; Prohaska, Ronald
Subject: Reach J, E, and 8B clarifications

Attachments: Benches_7_14.doc

Dave/Dan -

On behalf of Amphenol, Parsons has evaluated the recent as-built survey of the Reaches J, E, and 8A at the site by Lawson Engineering subcontracted to DA Collins. Based on a review of this information and drainage capacity checks the following recommendations are provided:

1. Parsons recommends that the 5.0 ft^2 area from the original design be maintained for Reach E along with the minimum height of 1-foot and a minimum slope of 3 percent to have capacity for a 25-year, 24-hour storm event. Parsons recognizes that the geometry of the swale cross-section may vary from the original design detail. In our opinion, this is acceptable provided the 5.0 ft^2 area and 1-foot of height are maintained with the following conditions of a maximum acceptable finished sideslopes of 3H:1V on the upgradient side and 2H:1V on the downgradient side.

2. Parsons recommends the southwestern landfill edge be completed with an intermediate reach (8b) installed along the eastern edge of the landfill between the existing Reach 8a and Cap Area 5 (west of the temporary access road described in FCO #010). The reach should start at the beginning of Reach E and flow south to discharge into the existing down shoot of Reach 8A. The Reach should be constructed to the configurations discussed above in Paragraph 1.

3. Parsons recommends that the landfill access road along Reach J be constructed with a minimum of 1-foot of elevation differential between the inside edge and the outside edge to promote runoff and provide sufficient capacity for a 25-year, 24 hour storm event using the present as-built configuration of Reach J (i.e., a portion of the access road will function as a swale). The 6-inch perforated pipe should still be installed in Reach J as required in FCO #009. At the bottom of the access road the crown of the road should be graded to prevent surface runoff across the road and maintain flow in Reach J as shown on the Contract Drawings. The final surface cover of the Reach J will be addressed separately following completion of subgrade repairs and liner installation.

Please feel free to contact Chris Green or Ray D'Hollander if you have any questions concerning the supporting calculations or the clarifications.

Regards,

Jim

Benches_7_14.doc

James M. O'Loughlin
Project Manager

PARSONS

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(315) 451-9570 (fax)
(617) 279-3436 (cell)

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Boston, Massachusetts 02110-1713
Direct: 617-449-1563
Main: 617-946-9400
Fax: 617-946-9777
(617) 279-3436 (cell)

SAFETY - MAKE IT PERSONAL

1.0 PURPOSE

Check the capacity and configuration of Reaches J and E at the Richardson Hill Landfill Site and determine if the as-built configurations are appropriately sized to accommodate the anticipated run-off of a 25-year storm event. Additionally, size an intermediate swale between Reach 8A and the edge of the landfill to remove surface runoff from Cap Area 5 and the downgradient area of Reach 8A.

2.0 ASSUMPTION

The following assumptions were made to support the calculations.

1. The design storm event will have a return period of 25 years.
2. A Manning's roughness coefficient of 0.030 similar to the one used as used in the O'Brien & Gere SWPP is applicable to the final grass surfaces of the benches at the RHLF site. A Manning's roughness coefficient of 0.033 is applicable for stone lined benches.
3. Stormwater calculations are for the completed design conditions.

3.0 APPROACH

The calculations were performed to address three benches on the landfill as follows:

- Case-1: Reach E, along the northwestern edge of the landfill.
- Case-2: Intermediate Reach (8b) between Reach 8A and edge of Cap Area 5/Temporary access road (FCO #010) along the southwestern edge of the landfill.
- Case-3: Drainage bench along the cap access road Reach J.

The configuration of the swales, slopes, and drainage areas were obtained from Lawson Surveying and Mapping of Schenectady, New York on June 1, 2006.

The peak rainfall for the 25 year return period for the landfill areas were obtained from Table 1-3 and Table 1-4 of O'brien and Gere's, *Stormwater Pollution Prevention Plan,* dated May 2005 (Revised August 3, 2005).

The drainage swale capacities were checked and sized using Manning's equation ($Q = (1.49/n)*A*R^{2/3}*S^{1/2}$)to check flow capacities of "V" shaped channels.

Manning's equation was used to calculate the swale height. The hand calculations are shown in Attachment B. Manning's equation was solved first for all the knowns of the formula (Q, n, and S) which is equal to A(5/3)/P(2/3) and can be written as Q*(n/1.49)*S(-1/2)=A(5/3)/P(2/3). The first half of this equation was used to solve for A(5/3)/P(2/3) in the Table 2. Next H is solved for and the reults of the previous equation are used in solving H. The resulting equation is H=[([Q*(n/1.49)*S(-1/2)]*(3.64+(1+B2)(1/2))(2/3))/(1.5+B/2)(5/3))](3/8).

The swale area per foot was next calculated using the found height of water for varying drainage bench slopes.

Table 1 - O'Brien and Gere SWPP Report

Area name	Peak Flow (25 year storm) (cfs)[1]	Reach	Slope (%)[2]	Slope (%)[3]
Woods 4	33.22	CH-8A	9.0%	NA[4]
Woods 2	7.9	E	3.0%	NA[4]
Cap 8	4.8	J	7.2%	13.0%

Notes

1. Peak flow is from Table 1-3 of the O'Brien and Gere SWPP Report for the landfill covered with grass.
2. Slope obtained from O'Brien and Gere SWPP Report.
3. Slope from 2006 LSW Survey.
4. NA not included in the 2006 LSW survey.

4.0 CALCULATIONS

Case 1 - Check capacities of Reach E

As-built swales		Peak Flow (25 year storm) (cfs)	Slope	$A^{5/3}/P^{2/3}$	Swale (water) Height	Swale (water) Area per foot (ft^2)	Check back to Q
H	V						
2	1	7.9	3.0%	1.010	0.88	1.94	8.36
3	1	7.9	3.0%	1.010	0.82	2.00	8.29
4	1	7.9	3.0%	1.010	0.77	2.05	8.24
5	1	7.9	3.0%	1.010	0.73	2.11	8.20
6	1	7.9	3.0%	1.010	0.69	2.16	8.17
7	1	7.9	3.0%	1.010	0.66	2.21	8.15

Reach E has sufficient capacity when constructed with a minimum height of 1-foot and maximum acceptable finished sideslopes of 3H:1V on the upgradient side and 2H:1V on the downgradient side.

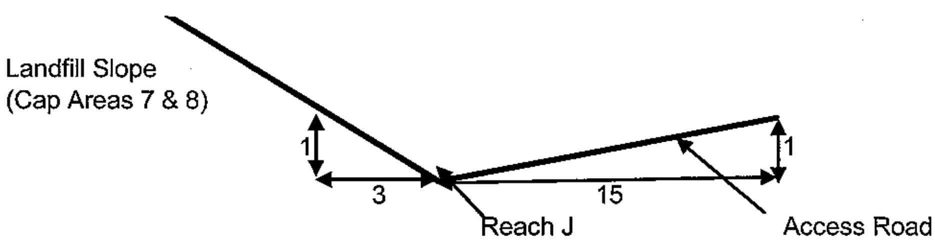
Case 2 – Sizing of intermediate reach (8b) between Reach 8A and the edge of Cap Area 5

As-built swales		Peak Flow (25 year storm) (cfs)(1)	Slope	$A^{5/3}/P^{2/3}$	Swale (water) Height	Swale (water) Area per foot (ft^2)	Check back to Q
H	V						
2	1	16.61	6.0%	1.502	1.02	2.62	17.58
3	1	16.61	6.0%	1.502	0.95	2.69	17.44
4	1	16.61	6.0%	1.502	0.89	2.76	17.33
5	1	16.61	6.0%	1.502	0.84	2.84	17.25
6	1	16.61	6.0%	1.502	0.80	2.90	17.18
7	1	16.61	6.0%	1.502	0.77	2.97	17.13

Note 1 – Reach 8A was designed by O'Brien and Gere with a peak 25-year storm flow of 32.2 cfs. The size of the contributing area for the proposed intermediate reach is significantly less than Wood Area 4. A peak flow of 16.6 cfs (approximately 50%) was selected for the sizing of this intermediate reach.

The proposed intermediate reach between Reach 8A and Cap Area 5 has sufficient capacity when constructed with a minimum height of 12-inches and maximum acceptable finished slopes of 3H:1V on the upgradient side and 2H:1V on the downgradient side.

Case 3 – Reach J along the landfill access road



Note: Dimensions are in feet

Height of Water =	1 ft
Area (A) =	9 ft^2
Wetted Perimeter (P) =	18.20 ft
Hydraulic Radius[1] (R) =	0.49
n =	0.033
Capcity (Q) =	91.6 ft^3/sec

OK This is greater than the design flow of 4.8 cfs.

5.0 CONCLUSIONS

1. Parsons recommends that the 5.0 ft^2 area from the original design be maintained for Reach E along with the minimum height of 1-foot and a minimum slope of 3 percent to have capacity for a 25-year, 24hour storm event. Parsons recognizes that the geometry of the swale cross-section

may vary from the original design detail. In our opinion, this is acceptable provided the 5.0 ft^2 area and 1-foot of height are maintained with the following conditions of a maximum acceptable finished sideslopes of 3H:1V on the upgradient side and 2H:1V on the downgradient side.

2. Parsons recommends the southwestern landfill edge be completed with an intermediate reach (8b) installed along the eastern edge of the landfill between the existing Reach 8a and Cap Area 5. The reach should start at the beginning of Reach E and flow south to discharge into the existing down shoot of Reach 8A. The Reach should be constructed to the configurations discussed above in Paragraph 1.

3. Parsons recommends that the landfill access road along Reach J be constructed with a minimum of 1-foot of elevation differential between the inside edge and the outside edge to promote drainage into Reach J and provide sufficient capacity for a 25-year, 24 hour storm event using the present as-built configuration of Reach J. The 6-inch perforate pipe should still be installed in Reach J as required in FCO #009. The surface cover of the Reach J (i.e. grass or stone) is presently being evaluated and will be address in a separate calculation. At the bottom of the access road the crown of the road should be grade to prevent surface runoff across the road and maintain flow in Reach J as shown on the Contract Drawings.

4. If conditions encountered differ from those anticipated or assumed, the conclusions and recommendations presented herein should be re-evaluated and modified or confirmed in writing.

6.0 REFERENCES:

1. United States Department of Agriculture Soil Conservation Service (USDASCS), 1986, *Urban Hydrology for Small Watersheds,* Technical Release 55

Rev	Date	By		Ck	Title
A	9/8/2006	SMA		RDD	Honeywell Richardson Hill Landfill

Objective:

Calculate the riprap sizing for Reach J at the Richardson Hill Landfill.

Assumptions:

The following assumptions were made to support the calculations.

1 The design storm event based on a return period of 25 years.

2 A flow rate of 4.8 cubic feet per second (cfs) was calculated based on the Drainage Bench J design calculations (dated 7/24/06, CMG of Parsons).

Approach:

1. The riprap sizing was based on the HEC-15 USDOT.

Reach J Riprap Sizing Calculations

Mannings (Blodgett Equation)

Calculate Mannings Roughness

$n=\alpha d^{1/6}/2.25+4.23\log(d_a/D_{50})$

Given:

Q=	4.8	cfs
B=	2	ft
z=	3	
So=	0.13	ft/ft
D50k=	0.25	ft
Assume:		
Initial trial depth=	0.341	ft

A	Area of Swale=	$Bd+zd^2$	1.03	ft^2
Pw	Wetted Perimeter=	$B+2d(Z^2+1)^{0.5}$	4.16	ft^2
R	Hydraulic Radius=	A/Pw	0.25	ft
T	Top of Swale Width=	B+2dz	4.05	ft
da	Average Depth=	A/T	0.25	ft

n Mannings Coeff.= $n=\alpha d^{1/6}/2.25+4.23\log(d_a/D_{50})$ = 0.046

Q Estimated Flow= 4.80 cfs

Conclusion:

Depth is within stone therefore, use Bathurst Equation

Mannings (Bathurst Equation)

Where:

Q=	4.8	cfs	L=	200	ft
B=	2	ft			
z=	3				
So=	0.13	ft/ft			
D50k=	0.25				
Assume:					

d	trial depth=	0.256 ft		
A	Area of Swale=	$Bd+zd^2$	0.71	ft^2
Pw	Wetted Perimeter=	$B+2d(Z^2+1)^{0.5}$	3.62	ft
R	Hydraulic Radius=	A/Pw	0.20	ft

T	Top of Swale Width=	B+2dz	3.54 ft
da	Average Depth=	A/T	0.20 ft

n= Manning's roughness coefficient

D_{50}= median riprap/gravel size (ft) 0.25 ft

α= unit conversion constant (0.262)

Calculate Mannings (Bathurst)

Formula:

$n=\alpha d^{1/6}/((32.2)^{1/2} f(Fr)\, f(REG)\, f(CG))$

$f(Fr)= (0.28Fr/b)^{\log(0.755/b)}$ Fr= V/(gL)^.5 Where: V=Q/A= 6.77 ft/s

f(Fr)= 0.35 = 0.08

$f(REG)= 13.434(T/D_{50})^{0.492} b^{1.025(T/D50)^{0.118}}$ where: T= 3.536 ft

Where T= top width of swale

f(REG)= 8.59

$f(CG)=(T/d_a)^{-b}$

f(CG)= 0.44

$b= 1.14(D_{50}/T)^{0.453}(d_a/D_{50})^{0.814}$

b= 0.29 ft

Q Estimated Flow= 4.81 cfs

Calculate Shear Velocity

Vx Shear Velocity= $(gdS)^{0.5}$

= 0.98 ft/sec

Re=VxD50/v

20156.4

Re</=4 x 10^4 therefore, Fx= 0.047 and SF= 1

n= 0.027

Slope is Greater than 10 %. Therefore:

D50>/= SF*D*S*△/Fx(SG-1)

D_{50} 0.39 ft for △ =1

△ is function of geometry and riprap size and is required for final D50 determination

$\Delta= K_1(1+\sin(\alpha+\beta)\tan\phi/2(\cos\theta\tan\phi - SF\sin\theta\cos\beta)$

α= Channel bottom slope 7.41 deg for 13%

β= Angle between weight vector and weight resultant vector in the plane of the sideslope

ϕ= Angle of repose = 38 deg

θ= Angle of channel sideslope = 18.43 deg for 2:1

$1.5 < z < 5$ therefore:

$K_1 = 0.066z + 0.67$

$= 0.87$

$K2 = (1 - (\sin\theta/\sin\phi)^2)^{0.5}$

$= 0.87$

Calculate Stability Number (η):

$\eta = \tau_s/(Fx(\lambda_s - \lambda)D_{50}$

where:

$\tau_d = \gamma dSo$ and

$= 2.08$ lbs/ft^2

$\tau_s = K_1 \tau_d$

$= 1.80$ lbs/ft^2

$\eta = \tau_s/(Fx(\gamma_s - \gamma)D_{50}$

$\eta = 1.50$

$\beta = \tan^{-1}(\cos\alpha/(2\sin\theta/\eta\tan\phi + \sin\alpha)$

$\beta = 60.49$ deg

$\triangle = K_1(1+\sin(\alpha + \beta)\tan\phi/2(\cos\theta\tan\phi - SF\sin\theta\cos\beta)$

$= 0.83$

D50>/= SF*d*S*$\triangle$/Fx(SG-1)

D50 for Riprap 0.36 ft or 4 inches

Long, William

From: Greene, Chris
Sent: Thursday, September 14, 2006 2:24 PM
To: 'dblodgett@dacollins.com'; dmacdougall@dacollins.com; 'James T. Mickam, P.G.'; 'Joe Bianchi'; Prohaska, Ronald
Cc: 'Bennett, Daniel'; D'Hollander, Ray; O'Loughlin, James
Subject: RHLF - Site Fence, Turnaround, and CH-2

All -

Below is a summary of our discussions yesterday pertaining to the site fence, turnaround along RHR, and additional rip rap placement along CH-2.

Site Fence

Parsons does not take exception to DACs proposal to drive some of the site perimeter fence posts directly into the subsurface as opposed to setting them with concrete bases, as show on Detail D of Drawing C-10. It is recommended that the spacing of concrete bases be determined by DAC's fencing subcontractor in the field based on site conditions and the fence configuration. Sufficient concrete bases and diagonal bracing shall be installed to support the permanent site fence. Concrete bases are recommended at fence corners and at gates and for posts adjacent to the corners and gates. As discussed, the existing fence at the north and south ends of the landfill have driven posts as well as the majority of the fence posts at the Sidney Landfill fence. To date neither of them has had a problem with performance. Additionally, the fence will not be installed with barbed wire on top.

RHR Turnaround

Parsons does not take exception to the proposal to install a stone turnaround on the former work platform of the extraction trench for the use of town maintenance vehicle. The location of the turnaround will be across the street from present landfill entrance. As discussed, the turnaround will have crush stone placed over a non-woven geosynthetic fabric. The limits of the stone will be determined in the field with Amphenol and DAC.

Rip-Rap along CH-2

Parsons does not take exception to DACs proposal to install medium rip rap at the edge of CH-2 along cap areas 8, 7, and 6. This edge has continued to have erosion issues prior to the establishment vegetation. The placement of medium rip rap will minimize these issues. In these areas the geombrane and GDC, installed on top of the liner extend, to the edge of CH-2. The medium rip rap can be placed directly on the GDC in this area.

If you have any questions or comments please do not hesitate to contact me.

Chris

Chris Greene
Environmental/Geotechnical Engineer

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chris.greene@parsons.com

O'BRIEN & GERE

December 7, 2006

Mr. James M. O'Loughlin
Project Manager
Parsons
290 Elwood Davis Road,
Liverpool, NY 13088

Re: SWPPP – Swale
Certification Letter

File: 8653/36252 #2

Dear Mr. O'Loughlin,

O'Brien & Gere Engineers has prepared this Engineering Certification letter associated with the NYSDEC approved Storm Water Pollution Prevention Plan (SWPPP) developed for the Richardson Hill Road Site on behalf of the Parsons. Based upon limited field observations made by O'Brien & Gere during the required site SWPPP inspections, O'Brien & Gere Engineers hereby certifies, as requested by Parsons, that the swales associated with the SWPPP appear to have been constructed in general conformance with the NYSDEC-approved SWPPP.

Very truly yours,

O'BRIEN & GERE ENGINEERS, INC.

James R Heckathorne, P.E.
Vice President

STATE OF NEW YORK
JAMES R. HECKATHORNE
56609
LICENSED PROFESSIONAL ENGINEER

I:/Div83/projects/8653-Parsons Eng./36252/2-Corr/Rip-Rap Swale Certification Letter.doc

cc: Ray D'Hollander – Parsons
Dave Farber – O'Brien & Gere