Waste Management of Carolinas, Inc.

Response To Regulatory Technical Comments

Transition Application
And
Permit Renewal

Piedmont Landfill And Recycling Center

Kernersville, North Carolina

April 1994

RUST ENVIRONMENT & INFRASTRUCTURE

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and Recycling Center
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A Waste Management Company

APR 1994
Received
Solid Waste
Section

April 15, 1994

Ms. Sherri Hoyt
Environmental Engineer
North Carolina Dept of Environment, Health, and Natural Resources
Department of Solid Waste Management (DSWM)
401 Oberlin Road
Raleigh, NC 27611

Re: Response to DSWM Letter Dated March 7, 1994 Concerning "Permit Renewal/Transition Application for the Piedmont Landfill & Recycling Center (PLFRC), Kernersville, NC"; Solid Waste Permit 34-06

Dear Ms. Hoyt:

Enclosed please find the following document that is being submitted in response to your above referenced letter:

- ***Response to Regulatory Technical Documents, Transition Application and Permit Renewal: Piedmont Landfill & Recycling Center*** by RUST E & I, dated April 1994

For ease of review, we have provided Instructions (attached) and have included in the front of the document a chart describing your specific comment and where in the document the response can be found. Changes to the original submittal documents dated December 2, 1993 have been made in the following manner: information that has been removed has been ~~struck through~~, and the additions have been underlined.

As you already know, airspace is at the critical stage for the PLFRC and we need to construct a new landfill cell this summer (estimated start is June 1994). We again want to thank you for your past expedient review and hope that you will continue to provide this prompt service.

Please feel free to contact either Ed Gibson or myself should you have any questions or comments. Eagerly awaiting your response, I am

Sincerely,

William R. Lewis, P.E.
Division President & General Manager

Fac/Perm/Co ID # 34-06
Date 8/9/13
Doc ID#
DIN

cc: Ed Gibson

RUST ENVIRONMENT & INFRASTRUCTURE

I hereby affirm that the information provided by RUST Environment and Infrastructure, Inc. and contained within this document is accurate and complete to the best of my knowledge and belief.

Clarke M. Lundell, P.E.
Department Manager

Date 4/13/94

North Carolina Professional Engineering Registration Number: 17778

PIEDMONT LANDFILL AND RECYCLING CENTER	
SUMMARY OF REVISIONS TO THE PERMIT RENEWAL/TRANSITION APPLICATION	
NCDEHNR Comment	**Location Where Comment is Addressed**
FACILITY PLAN - Need clarification on future development of the 38 acres.	INTRODUCTION - Page 1 OPERATIONS PLAN - Section 3.9 DESIGN DRAWINGS - Drawing Nos. 4A & 4B
FACILITY PLAN - Define waste stream and landfill capacity.	FACILITY PLAN - Pages 4, 5 and 6, Appendix B.
FACILITY PLAN - Better define equipment availability and type.	FACILITY PLAN - Page 5.
FACILITY PLAN - Define the soil needs and availability.	FACILITY PLAN - Pages 6 and 7, Appendix B OPERATIONS PLAN - Section 3.8
FACILITY PLAN - Define locations proposed for stormwater flaps.	FACILITY PLAN - Pages 8, 9 and 13.
FACILITY PLAN - Define sump capacity. Describe pumping system.	FACILITY PLAN - Page 12 and 13, Appendix D.
FACILITY PLAN - Describe the drainage of the 25-year, 24-hour storm from cells.	FACILITY PLAN - Page 14, Appendix D.
FACILITY PLAN - Define the operating capacity of the leachate line components.	FACILITY PLAN - Pages 15, 16 and 17, Appendix R.
FACILITY PLAN - Show the granular drainage material in contact with waste.	DESIGN DRAWINGS - Drawing Nos. 10, 11 and 12 of 21.
ENGINEERING PLAN - Clarify the clay used in the base liner and final cover.	FACILITY PLAN - Page 7 ENGINEERING PLAN - Page 31.
ENGINEERING PLAN - Clarify site life calculations.	FACILITY PLAN - Page 6 ENGINEERING PLAN - Appendix B OPERATION PLAN - Section 2.7.
ENGINEERING PLAN - Demonstrate long term performance of the geonet.	FACILITY PLAN - Pages 10 and 11, Appendix C.
ENGINEERING PLAN - Evaluate stresses in the base liner and sideslope drainage in the geonet.	ENGINEERING PLAN - Pages 35 and 36, Appendix K. FACILITY PLAN - Pages 11 and 12.
CQA PLAN - specify the minimum performance standards of system components.	CQA PLAN - Tables 1-5
CQA PLAN - Division should be notified of pre-construction meetings.	CQA PLAN - Section 2.6

PIEDMONT LANDFILL AND RECYCLING CENTER	
SUMMARY OF REVISIONS TO THE PERMIT RENEWAL/TRANSITION APPLICATION	
NCDEHNR Comment	**Location Where Comment is Addressed**
CQA PLAN - Clearly define specifications of the test pad.	CQA PLAN - Section 5.4
CQA PLAN - Define more clearly repairs to test areas.	CQA PLAN - Section 5.4.2.1and 6.10
CQA PLAN - CQA documentation must be submitted to the NCDEHNR.	CQA PLAN - Sections 5.4.4 and 11.0
CLOSURE/POST-CLOSURE PLAN - Define the maximum waste inventory.	ENGINEERING PLAN - Appendix B, CLOSURE/POST-CLOSURE PLAN - Section 2.2, OPERATION PLAN - Section 2.7.
CLOSURE/POST-CLOSURE PLAN - Define the vegetation plan.	CLOSURE/POST-CLOSURE PLAN - Sections 2.6 and 3.1.1, Appendix C.
CLOSURE/POST-CLOSURE PLAN - Provide a summary chart for post-closure activities.	CLOSURE/POST-CLOSURE PLAN - Section 3.1, Table 1.
CLOSURE/POST-CLOSURE PLAN - Provide further detail on the cost estimates.	CLOSURE.POST-CLOSURE PLAN - Appendices A and B.
OPERATION PLAN - Better describe the methods of waste placement.	OPERATION PLAN - Sections 3.3, 4.1.6 and 4.12.
OPERATION PLAN - Better describe the methods of incoming waste screening.	OPERATION PLAN - Section 4.1.1
OPERATION PLAN - Describe procedures for proper handling of asbestos or putrid wastes.	OPERATIONS PLAN - Section 4.1.2
OPERATION PLAN - Describe the alternative daily cover to be used at the facility.	OPERATION PLAN - Section 4.2, Appendix C.
OPERATION PLAN - Describe the logic for spacing of gas monitoring probes.	OPERATION PLAN - Section 4.5.1. DESIGN DRAWINGS - Drawing No. 17 of 21.
OPERATION PLAN - Further clarify the fire fighting and prevention plan.	OPERATION PLAN - Section 4.6.
OPERATION PLAN - Develop a safety manual for the facility.	OPERATION PLAN - Section 4.7.2, Appendix D.

PIEDMONT LANDFILL AND RECYCLING CENTER	
ADDITIONAL REVISIONS TO THE PERMIT RENEWAL/TRANSITION APPLICATION	
Issue of Revision	**Location Where Revision is Addressed**
Revised Groundwater Countours.	DESIGN DRAWINGS - Drawing No. 2, 3, 4, 5, 6, 7, 8 and Cover Sheet. ENGINEERING PLAN - Page 3. FACILITY PLAN - Pages 14 and 46, Appendix A, B, C, D and L.

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WASTE MANAGEMENT OF CAROLINAS, INC.

TRANSITION APPLICATION AND PERMIT RENEWAL

FOR

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, NORTH CAROLINA

Prepared By:
RUST ENVIRONMENT & INFRASTRUCTURE
GREENVILLE, SOUTH CAROLINA
NAPERVILLE, ILLINOIS

DECEMBER 1993

REVISED: APRIL 1994

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INTRODUCTION/STATEMENT OF PURPOSE - NCAC 13B .1603(b)(1)(B)

Waste Management of Carolinas, Inc. (WMCI) currently owns and operates the Piedmont Landfill and Recycling Center (PLRC) in Kernersville, North Carolina. The facility operates under permit number 34-06 and was originally permitted in 1989. This is an approximately 108 acre site in Forsyth county bordering Guilford county. The current landfill area is approximately 30 acres. This area is comprised of modules in Phases I and II of the landfill as originally permitted which are partially completed.

In response to the recently approved North Carolina Solid Waste Regulations, this document contains both a transition application and a permit renewal. The transition application is for an operation continuance in the existing units (Phases I and II). The permit renewal is in accordance with .1603(a)(5)(C) which requires a permit renewal 180 days prior to the commencement date for base liner construction. ~~In the attached letter,~~ Based on daily tonnage rates, construction of the next cell is anticipated for ~~April~~ June 1994. The remainder of the existing landfill footprint has been redesigned and renamed Phase III - Cells 1 and 2 to be consistent with Section .1600 of the regulations. Phase III - Cells 1 and 2 are included in the permit renewal.

Also, in accordance with the existing Solid Waste Permit 34-06, condition General 6., the application is being submitted to the North Carolina Department of Environment, Health, and Natural Resources (NCDEHNR) in order to perform the applicable 5 year review.

Modules 1, 2, a portion of 3, 5, and 6 of Phase I have been constructed and are being filled. Modules 1 and 2 of this phase utilized a composite liner for containment. Modules 3, 5 and 6 incorporate a double geosynthetic liner system.

Phase II - Modules 1 and 2 are also being filled. Both of these modules contain a double geosynthetic liner system.

The current landfill design contains Phase III - Modules 1, 2 and 3 and Phase IV - Modules 1, 2 and 3. These phases have not been constructed and are part of this permit renewal. In this renewal, Phase III modules will become Phase III Cell 1. Phase IV along with the remainder of Module 3 and Module 4 of Phase I will become Cell 2 of Phase III (see Drawing 1 for cell layout). These new proposed cells will be designed with a Subtitle D composite liner system in accordance with NCAC 13B .1624 of the regulations.

TRANSITION APPLICATION

AND

PERMIT RENEWAL

TRANSITION APPLICATION - NCAC 13B .1617(d)

A. OPERATIONS PLAN - NCAC 13B .1617(d)(1)

The operations plan for the existing landfill is incorporated into the operations plan for the permit renewal. See the Operation Plan section for a detailed explanation of this plan.

B. CLOSURE/POST CLOSURE PLAN - NCAC 13B .1617(d)(2)

The existing facility will be closed in the same manner as the proposed cells in the permit renewal. See the Closure and Post Closure Plan section for an explanation of the closure plan.

C. WATER QUALITY MONITORING PLAN - NCAC 13B .1617(d)(3)

The existing facility will be monitored in conjunction with the proposed new phases of the landfill. ~~See the~~ A revised Water Quality Monitoring Plan ~~section for a description of the water quality monitoring plan for the entire site~~ for the Transition Application has been prepared by RUST Environment & Infrastructure, Inc., dated March 1994 and submitted to NCDEHNR for review.

D. CONTINUED OPERATING PLAN

NCAC 13B .1617(d)(4) - *A report that defines the owner's or operator's plans for continued operation of the existing facility or a new facility for a minimum five year period and incorporates:*

(A) A closure date for the existing MSWLF unit

(B) A schedule for submitting the required permit applications for a new facility, permit renewal or planned use of any MSWLF facility which meets the requirements of Subparagraph (b)(1) of Rule .1624

At the present waste volume, the existing constructed cells at the PLRC have sufficient airspace to continue operating for approximately 1 year. In response to the new regulations, along with this Transition Application is a Permit Renewal for the remaining 38 ± acres of landfill disposal area. This area which is known as Phase III will extend the site life for approximately 3.1 additional years. The Phase III area has been redesigned from the original permit to meet the current regulations.

The existing disposal area will continue to be operated as currently permitted. The existing units compliance with the location standard of rule .1622 is provided in the

Engineering Plan Section. The operations of the existing units is provided in the Operation Plan Section.

PERMIT RENEWAL

A. FACILITY PLAN

1. Waste Stream

NCAC 13B .1619(e)(1) - *A discussion of the characteristics of the wastes received at the facility and facility specific management plans shall incorporate:*

(A) The types of waste specified for disposal

(B) Average monthly disposal rates and estimated variance

(C) The area served by the facility

(D) Procedures for segregated management at different on-site facilities

(E) Equipment requirements for operation of the MSWLF unit

The PLRC is designed to accept the following types of wastes: household waste, commercial waste, and industrial solid waste. PLRC will not accept hazardous, radioactive or PCB wastes as defined in Rule .1626(1)(b). Liquid wastes will also be excluded unless managed according to Rule .1626(9).

The PLRC is also designed to accept the following types of wastes:

chemical laboratory waste	sludge waste
chemical-containing equipment	animal wastes and parts
"empty" chemical containers	asbestos demolition waste
discontinued chemical products	
solid waste and medical waste incinerator ash	

~~PLRC anticipates receiving an average monthly rate of 52,000 tons/month. This disposal rate is subject to variances due to the addition of new customers and one time disposal occurrences. This rate is expected to increase up to 50% annually to account for growth in the service area of the landfill.~~ As you can see from Appendix B, the PLRC is estimating that the average monthly disposal rate will be approximately 62,000 gate tons. You will also notice that, compared to the average monthly disposal rate for the most recent six

(6) months, the variance is 135%. The reason for this variance is due to mainly two factors:

- Known and expected waste streams that are expected (better than 50% chance of receiving the waste) for 1994. This is expected to be approximately 600 tons per day.

- Unknown waste streams that may enter the PLRC, particularly in the area of special waste. As you know, the PLRC is open for business in the service area designated. This service area contains a wide open area for special waste customers not only in long term commercial/industrial contracts but also for the one-time clean-up jobs.

This service area is currently comprised of areas in Kentucky, North Carolina, South Carolina, Tennessee, Virginia, and West Virginia.

The PLRC has no other waste operations at the facility that would require segregated management.

Based upon the anticipated disposal rate and area to be filled, it is expected that the following equipment will be utilized on a daily basis.

1 Articulated Truck
1 Scraper
1 Bulldozer
2 Compactors
1 Motor Grader
1 Water Truck
1 Trac Backhoe

The quantity of equipment may vary due to fluctuations in the disposal rates or the receipt of special waste types which may necessitate special disposal procedures.

At the present time the PLRC is using two compactors to handle the working face: Rex Model 390, weighing 96,000 lbs, and a Caterpillar Model 826, weighing 70,000 lbs. Also, the PLRC has a backup compactor on site: Rex Model 355, weighing 50,000 lbs. These type compactors are subject to change as the needs of the landfill vary.

Should the need arise to obtain additional equipment, the PLRC will be able to lease the appropriate equipment from local equipment suppliers/service

firms. Historically, the PLRC has been able to obtain this equipment from these suppliers within 48 hours of notification.

2. Landfill Capacity

NCAC 13B .1619(e)(2) - *An analysis of landfill capacity and soil resources shall be performed.*

(A) The data and assumptions used in the analysis shall be:
- *(i) Consistent with the facility drawings and disposal rates specified in the facility plan*
- *(ii) Representative of operational requirements and conditions*

(B) The conclusions shall provide accurate volumetric estimates of:
- *(i) Total operating capacity*
- *(ii) Operating capacity for each phase of development*
- *(iii) In-place ratio of waste to soil*
- *(iv) Available soil resources from on-site or specific off-site sources*
- *(v) Required quantities of soil for landfill construction, operation, and closure*
- *(vi) The estimated operating life of all MSWLF units in years*

The proposed Phase III area included in this permit renewal has a maximum estimated design volume of approximately ~~4.3~~ 4.1 million cubic yards. This volume is comprised of a combination of waste and daily/intermediate cover soils. The in-place density of the waste with daily and intermediate cover soils ~~is anticipated to be~~ varies due to the type waste disposed of and also the compaction equipment used but it is anticipated to be approximately 1,200 lb/cy. The cover soils for daily and intermediate cover will be obtained from soil excavated on-site. See Appendix B for the soil balance calculations.

Based upon the cell configurations, soil to waste ratio, and final closure area, quantities of soil materials have been estimated. These quantities include liner materials, daily and intermediate cover materials and final cover materials. See ~~Table 1~~ Appendix B for a summary of these estimates.

The net ~~refuse~~ airspace for Phase III is estimated to be ~~3.5~~ 4.1 million cubic yards. Using the anticipated waste disposal rate of ~~2200~~ 2,600 tons/day, this phase should have an operating life of approximately ~~3.4~~ 3.5 years. Combined with the remaining airspace in the existing cells, this extends the total operating life of the site to approximately 4.5 ~~4.6~~ years (see revised Appendix B for site life calculations).

The daily and intermediate cover soil, protective soil and compacted soil in the final cover, compacted soil liner amended with bentonite, and structural

fill for perimeter berms is expected to be obtained from on-site sources. Soil available from on-site sources totals approximately 1.59 million cy. On-site soils required is approximately 890,069 cy. The liner protective cover soils are expected to be obtained from off-site commercial sources.

The compacted soil liner may not be amended with bentonite and therefore will be obtained from off-site sources. Bentonite admixing is discussed further in the CQA Plan.

3. Containment and Environmental Control Systems

NCAC 13B .1619(e)(3) - *A general description of the systems designed for proper landfill operation, system components, and corresponding function shall be provided.*

Phase III of the PLRC will be constructed with a Subtitle D composite liner system. This system includes two feet of clay overlain by a 60-mil High Density Polyethylene (HDPE) geomembrane. The composite liner system is overlain by a leachate collection system which consists of a geonet, perforated pipe, and two feet of a protective/drainage layer.

The compacted clay material will have a maximum permeability of $1x10^{-7}$ cm/sec. This layer minimizes any leakage through the geomembrane from migrating into the groundwater. Above the clay layer, the 60 mil HDPE liner will be placed. This material will minimize leachate leakage into the underlying clay liner. The subbase will slope at a minimum 2% post settlement grade directing leachate to a leachate collection pipe or sump.

The leachate collection system has several components. The first is a geonet which is directly above the HDPE geomembrane. This net provides a conduit layer for the leachate to flow over the bottom slope of the landfill cell. Above the geonet is the protective/drainage layer. This layer will allow percolation of leachate from the waste down to the geonet. A geotextile filter will be placed between the geonet and the protective soils to minimize the infiltration of fine grained soil particles into the geonet.

The base of each cell is sloped towards the leachate collection trenches. The trenches, located either down the center of the cell or at the toes of slopes, contain a perforated pipe which transports the leachate to a sump. Riser pipes extend down the exterior berm into the sumps containing pumps which transfer the leachate from the sump to the leachate forcemain. The exterior berm surrounds the landfill and is a minimum 5 feet high to prevent leachate from migrating out of the landfill area. The liner system extends up the

interior slope of the berm and is held in place by an anchor trench at the top of the berm.

4. Leachate Management

NCAC 13B .1619(e)(4) - *An analysis of the leachate management requirements and plans for the MSWLF facility shall incorporate the information required under this Subparagraph.*

(A) The performance of and design concepts for the leachate collection system within active areas of the MSWLF unit and any storm water segregation included in the engineering design shall be described.

(B) Normal operating conditions shall be defined and must consider:

(i) Average monthly values for leachate generation representative of the landfill's environment and operation using:

(I) empirically derived estimates: or

(II) for landfill expansions, actual leachate generation data from the existing landfill.

(ii) Surge volumes generated by storm events

The PLRC was designed with a leachate collection system within the landfill. The leachate collection system consists of the following components: a drainage layer, collection trenches with perforated piping, collection sumps and leachate discharge and removal components. ~~In addition, stormwater flaps may be used to segregate stormwater falling upon constructed but unfilled areas, thus minimizing the generation of leachate. The location of stormwater flaps within a cell will be determined in accordance with operational needs at time of construction.~~ Refer to the design drawing for locations and details of the leachate collection system components.

Temporary stormwater flaps/berms may be proposed for use at a future date to segregate stormwater falling upon constructed but unfilled waste disposal cells from active waste disposal cells, thereby minimizing the generation of leachate. Stormwater contained by these structures which does not come in contact with waste would be pumped into the facility stormwater management system. Stormwater coming in contact with waste will be treated as leachate. The locations

of stormwater flaps/berms within an unfilled cell would be determined based upon operational needs at the time of construction. The stormwater flaps/berms would be removed as waste disposal advances across the cell and toward the location of the structure.

Presently, stormwater flaps/berms are not proposed for use at the PLRC because of anticipated high rate of waste disposal would not allow the use of these components for an extended period of time. This is because a lift of waste will rapidly be placed across the floor of the newly constructed cells, thereby not facilitating the installation and subsequent removal of the structure. However, in the event that stormwater flaps/berms are determined practical for future use at the PLRC, the NCDEHNR will be notified of the intended locations and times of construction. Refer to the leachate collection system details provided in the Design Drawings, located in Volume 2, for a typical detail of the temporary stormwater flap/berm proposal by the facility.

The leachate drainage layer will be located over the entire base liner system, directly above the 60 mil HDPE geomembrane. The drainage/protective layer components in the currently developed areas of the landfill consist either of two feet of granular drainage material with a saturated hydraulic conductivity of 1×10^{-2} cm/sec or a geonet with a geotextile and two feet of protective layer. A HDPE geonet secondary drainage layer exists in part of the currently developed landfill areas.

The proposed leachate drainage layer for the undeveloped portions of the PLRC consists of a HDPE geonet drainage layer overlain by a 16 oz/sq yd nonwoven geotextile filter/cushion, which is overlain by two feet of protective/drainage layer. The primary function of the protective/drainage layer is to convey liquids which have infiltrated down through the landfill laterally to leachate collection trenches. The leachate collection trenches will then convey the liquids to the collection sumps for removal from the landfill. In addition, this layer serves as a minimum two foot thick barrier to protect the integrity of the low permeability components of the landfill base liner.

The drainage layer was designed to ensure that the leachate head on the barrier components of the base liner does not exceed one foot under normal operating conditions. Calculations were performed to evaluate the maximum head anticipated during operations. A peak monthly impingement rate of 6.1 inches was assumed to be collected in the drainage layer. This value represents the anticipated peak

monthly precipitation rate, as determined by local climatic background data. The peak monthly value was confirmed by the Hydrologic Evaluation of Landfill Performance (H.E.L.P.) computer software model, as developed by the U.S. Environmental Protection Agency and the U.S. Army Corps of Engineers. The impingement rate reflects "open cell" conditions, with little or no waste on the landfill floor. Based on the calculation, the head anticipated under this impingement rate is minimal (i.e., less than 0.1 inches). The low head value reflects the high permeability inherent to a HDPE geonet. As waste is deposited in the landfill and development advances, the impingement rate will greatly decrease and even lower head values should be ensured.

In order to ensure that the assumptions regarding the long-term performance of the geonet drainage layer are appropriate, a series of additional analyses were made. As mentioned previously, leachate head was analyzed under "open cell" conditions, which represents a worst case scenario for leachate drainage. To verify this as correct, the performance of the drainage layer was further analyzed under interim fill conditions. The HELP model was used to estimate the peak daily leachate generation under interim conditions (i.e., 60 feet of waste with one foot of intermediate cover). The resulting peak daily leachate drainage was .0288 inches per day (.88 inches per month). In addition, the geonet permeability was recalculated to reflect the effects of the waste overburden. As a conservative measure, the permeability was calculated for the maximum waste height designed at final closure conditions (i.e., 120 feet). The resulting geonet permeability was 29.5 centimeters per second (cm/sec). Comparing the results of the various calculations, the leachate generation rate decreased greatly in going from "open cell" to interim conditions (i.e., 6.1 inches per month to .88 inches per month), while the geonet permeability decreased only slightly (i.e., 35 cm/sec to 29.5 cm/sec). Based on this information, analyses of leachate head under "open cell" conditions does represent worst case conditions for leachate head. Refer to Appendix C for the additional evaluation.

A factor of safety of 2.0 was applied to the original geonet permeability of 35 cm/sec when performing the leachate head analyses. This was to account for potential decreases in the permeability resulting from siltation. Although this is an adequate value to use for a factor of safety, the leachate head was further evaluated under "open cell" conditions with a factor of safety of 4.0. The resulting maximum leachate head level anticipated, using a conservative factor of safety of

4.0, was calculated to be 0.19 inches. This maximum anticipated head value remained very minimal, and will maintain the leachate levels within the 0.2 inch thickness of the geonet layer. Once again, the low head value reflects the high permeability inherent to a HDPE geonet. Refer to Appendix C for the additional evaluation.

It should be noted that the factor of safety applied to the geonet permeability under "open cell" conditions is a very conservative approach. Siltation and biological growth are not a threat under "open cell" conditions. Rather, potential decreases in geonet permeability are more likely under interim and final development conditions, when the infiltration of leachate through the collection system may promote siltation and biological growth. Because of this, further analysis was performed to determine the factor of safety of the geonet permeability in regard to maintaining leachate head levels under interim fill conditions (refer to Appendix C). Based on this analysis, a minimum geonet permeability of 1.25 cm/sec is required under peak leachate drainage flows under interim conditions in order to maintain head levels within the 0.2-inch geonet thickness. The initial geonet permeability value under maximum landfill loads is 29.5 cm/sec. Therefore, a factor of safety of 23.6 (i.e., 29.5 cm/sec ÷ 1.25 cm/sec) is available to maintain leachate head levels. As final cover is applied to the landfill, leachate generation levels will decrease, thus further facilitating the maintenance of leachate head levels.

The initial permeability of the geotextile fabric overlying the geonet layer is 0.62 cm/sec. The geotextile will tend to retain particles which pass through the waste. The geotextile may also be subject to the development of biological growth. However, the true function of the leachate collection system will be maintained as long as the geonet permeability remains greater than that of the overlying waste and protective soils. The permeability of the waste is typically found to be $1x10^{-4}$ cm/sec, which matches the minimum initial permeability of the protective soils. In order to maintain effective leachate collection, the geotextile permeability must remain greater than $1x10^{-4}$ cm/sec. Based on this, the factor of safety against geotextile failure is approximately 6,200 (i.e., .62 cm/sec ÷ $1x10^{-4}$ cm/sec). Based on this information, the geotextile should provide adequate filtering capability while maintaining adequate permeability, without the potential threat of clogging.

The geonet drainage layer was further evaluated to ensure that sufficient drainage would be available on the sideslopes to maintain

stability of the protective soils, prior to filling within the cell. Appendix K presents a calculation which demonstrates that adequate drainage capacity is provided. It is expected that during operations within each cell, waste disposal will occur at a rate which will result in the sideslopes being covered within a relatively short period of time. Therefore, a check of the drainage capacity of the geonet was performed assuming a 5-year, 24-hour storm even (i.e., 4.5 inches). Although it is likely that a large portion of the precipitation will run off and not infiltrate the protective soils, the full intensity of the precipitation was considered to drain through the protective soils and into the geonet. Based on the results of the calculation, the geonet is capable of draining approximately 307 $ft^3/ft/day$ of liquids. The selected precipitation event will result in 11.25 $ft^3/ft/day$ of drainage requirement. The resulting factor of safety of 27.3 demonstrates that the drainage ability of the geonet is more than adequate in facilitating drainage without protective soil instability.

Leachate collected within the drainage layer is laterally conveyed to "V" shaped leachate collection trenches. The trenches are one foot deep and six feet wide at the top. Six inch diameter perforated piping is placed on the bottom of the trenches. The piping will be made of either SCH 80 PVC or SDR 17 HDPE materials. The perforations of the piping will be 3/8 inch in diameter. The leachate collection piping will be accessible from the perimeter of the landfill for cleaning as needed by cleanout riser pipes.

Calculations were performed which demonstrate that both the SCH 80 PVC and the SDR 17 HDPE piping is structurally capable of bearing the maximum landfill loads without crushing, buckling or developing excessive ring deflection (see appendices in Engineering Plan).

After the leachate collection piping is placed in the trenches, the trenches are backfilled with suitable aggregate. This aggregate is selected because it is large enough to not pose a threat of clogging the pipe perforations. The entire leachate collection trench is wrapped in geotextile to filter the smaller protective soils out of the aggregate, and to provide an added cushion above the 60 mil HDPE geomembrane liner.

Leachate collected in the collection trenches is conveyed to leachate collection sumps. The sumps serve to provide temporary collection of leachate prior to discharge. Calculations provided in Appendix D of the Engineering Report demonstrate that the leachate collection sumps

provide a temporary liquid storage volume of approximately 2,800 gallons. This assumes a porosity of the drainage stone within the sump of 35 percent. Leachate will be discharged from the sump either ~~manually of via~~ with a manual pumping system or with an automatic forcemain system. The pump size for the manual pumping systems may range anywhere from 40 to 100 gallons per minute (gpm) as determined necessary at the time of use. The pump size for the automatic forcemain system will typically be 40 gpm. The manual pumping system will be utilized prior to incorporation of the automatic forcemain system within a sump (i.e., during the early stages of filling within a landfill area). Leachate will be manually pumped from the sumps and into the leachate storage tanks when liquid levels reach elevations no higher than 12 inches above the top of the sumps. The forcemain pump is located within an 18 inch diameter perforated SDR 11 HDPE riser pipe. The pumps within the automatic forcemain system will be activated via pressure transducers. The pressure transducers will be set to activate the pumps when liquid levels within the sumps reach elevations no higher than 12 inches above the top of sump. The pipe perforations are 3/8 inch diameter, and begin at the bottom of the sump. The perforations end as the pipe exits the sump area. The sump area is backfilled with suitable aggregate to provide adequate drainage for liquids. The entire sump is then wrapped in geotextile, in a manner similar to the leachate collection trench, to provide filter and cushion properties. Care will be exercised to ensure that no gaps remain between the geotextile filter and the sump riser pipe, thereby minimizing the infiltration of fine grained particles into the sump area.

The forcemain system will discharge leachate when levels reach a predetermined elevation, generally no greater than 12 inches above the crest of the sump in each cell. To provide added protection to the primary geomembrane a second 60 mil HDPE geomembrane will be welded to the primary geomembrane in the sump area. Additional, added geotextile cushions will be placed in the sump floor to protect the geomembrane liner components.

In order to minimize the amount of leachate generated during initial filling at the facility, temporary stormwater flaps/berms may be constructed to segregate stormwater within unfilled areas from active fill areas. These structures are not currently proposed for use at the facility because the high rate of waste disposal would not made stormwater flaps/berms practical for use. However, in the event that the structures are determined practical in the future, the NCDEHNR

will be notified of all proposed locations and times of construction. Should the stormwater flaps/berms be used, they will be constructed on the landfill floor and utilized during "open cell" conditions. As landfilling advances, the stormwater flaps/berms will be removed.

The landfill geometry will be capable of containing the volume of ~~leachate~~ liquids generated by the 25-year, 24-hour storm. Calculations were performed to demonstrate that the landfill cells are capable of containing the aforementioned design storm (see Appendix D ~~appendices in Engineering Plan~~). The calculations were performed under "open cell" conditions, with no waste in the cells. Therefore, the contained water would be pumped out as stormwater. The storm value was chosen as six inches which is greater than the 25-year, 24-hour storm event of 5.5 inches. This is a conservative approach. As an additional measure of conservatism, no evaporation was included in the calculation. Based on the results, ~~Phase 3 Cell 2 Subcell 1 will require pumping at a minimum rate of 54 gpm to contain the storm. The remaining cells will each contain the storm without the use of pumping~~ each of the subcells within Phase 3 will be capable of containing the 25-year, 24-hour design storm of six inches without overtopping. Analysis prior to the April 1994 base grade redesign has also demonstrated that the subcells can contain the 100-year, 24-hour design storm of 8.4 inches. The use of manual pumping to maintain containment of the 100-year, 24-hour storm of 8.4 inches was required in Phase 3 - Cell 2 - Subcell 1.

The time to drain the 25-year, 24-hour storm will vary depending on a number of conditions, including the open cell volume and pumping rate. Dewatering time calculations were performed for Phase 3 - Cell 2 - Subcell 1, because this subcell has the largest area for the 6-inch design storm to fall upon (refer for Appendix D of the Engineering Plan). Based on these calculations, at the minimum required pumping rate of 40 gpm, it will take a maximum of 17.6 days to completely pump the stormwater from the subcell. The dewatering time can be decreased further through the use of larger pumps, or multiple pumps. It should be noted that as waste is disposed within the newly constructed subcells, the volume available for liquid containment will decrease, and dewatering times will decrease. However, under those conditions, all liquids coming in direct contact with waste will be collected for treatment rather than pumping directly into the surface water management system. At no time will liquids be impounded over the waste. The presence of daily, intermediate and final cover soils, as well as adequate grading of the fill areas, will

promote surface water runoff to the perimeter surface water management systems.

If the need to pump from ~~Phase 3 Cell 2 Subcell 1~~ a subcell does arise under the "open cell" conditions previously described , and if the liquid does come in contact with waste, the liquid will be treated as leachate. The leachate will be removed and disposed if a manner consistent with the leachate removed from other collection sumps. Refer to Section 4, Leachate Management, NCAC 13B.1619(e)(4)(D) of this Facility Plan, for a description of the contingency plan for storm surges. Liquids which do not come in contact with waste will be pumped into the facility surface water management system. Storage requirements for leachate were determined based on historic rainfall data from the site.

(C) *A description of the leachate management system components and their engineered function shall be provided, including:*

(i) Leachate pipeline operating capacity

(ii) Capacity of the storage and if applicable, the treatment facilities

(iii) Final disposal plans and applicable discharge limits, including documented prior approval of the waste water treatment plant which may be designated in the plan

Leachate Management System

Leachate collected within individual cells is pumped into a common forcemain encircling the lined portion of the landfill and connected to the leachate storage tanks on Freeman Road until removed by truck to a treatment facility.

Collection Sump

Within each cell's sump, a submersible pump is installed in the sideslope riser. These pumps operate automatically by level sensors or pressure transducers responding to leachate level fluctuations within the sump. Sensors cause the pump to turn on when the level of leachate is high enough for proper pump operations, but not above the permitted level for hydrostatic head limits. Additionally, sensors will turn pump off when the liquid level is down to the minimum allowable for proper pump operations. The pump on/off positions can be adjusted to suit the individual sump, but the former is set to limit

hydrostatic head on the liner and the latter is limited by the need to protect submersible pumps from pumping dry. A practical consideration is that the pump may lower the leachate down to a level of six inches in the sump.

Leachate pumps within individual cells are sized as follows:

- Evacuate liquid collection in sump within a reasonable time.
- Minimize the number of start/stop cycles.
- Maintain minimum velocity in pipeline of at least two (2) feet per second.
- Capable of generating sufficient dynamic head to move the leachate to the storage tanks.

Additionally, the pumps would need to be sized so that when more than one sump pump is operating and discharging into the common forcemain, liquid is being moved and pumps are not overloaded.

For this particular case, the following combination of pump operations represent the "worst case" scenarios:

- Sump B and C, Phase 3, Cell 1 pumping simultaneously.
- Sump C, Phase 3, Cell 1 pumping to storage tanks.

Included in Appendix R are calculations that show:

- Pump in Sump C transfer leachate to storage tanks (Graph 1).
- Sump B transferring leachate to storage (Graph 2).
- Sump C and Sump B both pumping simultaneously (Graph 3).

Capacities

When Sump B or C are pumping alone, capacities are approximately 80 gpm each.

When both sumps are operating simultaneously, the rate is 110 gpm.

Forcemain

The leachate forcemain is a dual contained HDPE piping system. The carrier pipe is 3" in diameter and the containment pipe is 6" in diameter. This piping system is designed to transport the leachate from the sumps to the storage tanks.

Storage

The current storage capacity at PLRC is for 12,400 gallons. This storage is comprised of two leachate storage tanks which receive leachate from the existing forcemain. In consideration of the proposed Phase 3 expansion, PLRC will add more leachate storage tanks to those already in operation to increase the total storage capacity to a minimum of 25,000 gallons. Appropriate secondary containment will be provided for additional storage. Should the flow increase or decrease drastically, the schedule at which the tanks are emptied may be varied. Additional storage tanks will be added as operational needs require to include the need to store leachate for recirculation purposes.

The tanks are emptied by a tanker truck which transports the leachate to an approved treatment facility. At the present time, the NCDEHNR has approved two private WWTP's for leachate disposal. One, Environmental Compliance Corporation (ECC), is located in Winston-Salem. The other, Terradyne, is located at 818 Ann Street, Fayetteville, North Carolina. Details of the leachate disposal permit and contract agreement with Environmental Compliance Corporation (ECC) and NCDEHNR approval of Terradyne, Inc. are included in the Leachate Management Plan located in the appendices of the Operation Plan Section.

System Capacity

The leachate generation is estimated to be at a rate that requires a leachate sump pump to turn on and pump the sump down once in every 24 hours. The system capacity is controlled by:

- Capacity of individual sump pumps.
- Leachate storage tanks.
- Rate at which trucks are able to remove leachate from storage for disposal.

Pipeline capacities may range between 44 and 180 gpm simply as a function of velocities of liquid in the line.

(D) *A contingency plan shall be prepared for storm surges or other considerations exceeding design parameters for the storage or treatment facilities.*

In case of a storm surge or the ultimate disposal facility ceases operation temporarily, PLRC has several operating options. First, the proposed landfill cells are designed to store a 24-hour, 25-year storm within each cell. If a storm of this magnitude occurs, temporary storage can be provided in the cell until pumping can catch up with the inflow of water into the cell. Secondly, the leachate can be pumped to the on-site storage tanks. In the event of a storm surge, the tanks can be emptied by tanker trucks on a more frequent basis. In the event of the ultimate disposal facility ceasing operation, PLRC will utilize the backup disposal facility.

B. ENGINEERING PLAN

1. Analysis of Facility Design - NCAC 13B .1620 (d)(1)

a. Foundation Standards

(A) - The standards for the foundation and the base liner system set forth in Rule .1624 of this Section

A description of the foundation standards and the base liner system are contained in the Landfill Subgrade section.

b. Cap System

(B) - The standards for the cap system set forth in paragraph (c) of Rule .1627 of this Section

The cap system for PLRC has been designed to minimize the infiltration of stormwater through the closed landfill. The soil portion of the cap will have a permeability equal to or less than $1x10^{-5}$ cm/sec. A geomembrane overlain with a geotextile/geonet/geotextile composite will overlay the soil component. Twenty-four inches of protective cover with the upper 6" soil layer capable of supporting vegetation will overlay the geosynthetic composite.

c. Leachate Storage

(C) - The standards for the leachate storage facilities set forth in Rule .1680 of this Section

PLRC currently has two 6200 gallon leachate storage tanks. These tanks are located on the south side of the landfill site. The tanks are constructed of HDPE and are located on a concrete slab with a 4' concrete block wall for secondary containment. Additional leachate storage tanks are proposed to increase the storage capacity to approximately 25,000 gallons. Since the tanks will not be interconnected, the existing secondary containment will continue to be provided as required.

2. Summary of the Facility Design - NCAC 13B .1620(d)(2)

a. Analytical Methods

(A) - A discussion of the analytical methods used to evaluate the design

Through the course of this design several analytical methods have been used to insure that the design is practical, environmentally safe, and that it complies with the NCDEHNR rules. Analytical methods and associated calculations are identified in the attached appendices.

b. Critical Conditions

(B) - Definition of the critical conditions evaluated and assumptions made

Several critical conditions were analyzed during the design of Phase 3 of PLRC. As a minimum, but not limited to, the following critical conditions include:

In the design of the landfill base grades, the landfill cells shall be able to store a 24-hour, 25-year storm event to contain leachate when initially open. The base grades shall be sloped at a minimum post-settlement slope of 2% and maintain a minimum separation to seasonal high groundwater table of 4 feet from the bottom of the liner system.

In the design of the leachate collection system, the base grades must be sloped sufficiently, the collection system should have adequate permeability, pipes be spaced appropriately, and the sump pump must be able to transport the leachate at an adequate rate such that the leachate does not exceed one foot of head over the liner system under normal operating conditions.

The base, side, and top slopes of the landfill must be stable to support the landfill during seismic events.

Other conditions analyzed include:

location standards	foundation standards
pipe strength	anchor trench
cover stability	geotextile clogging
cap settlement	surface water management
gas management	

All critical conditions were analyzed with supporting calculations provided in the appendices to this section.

c. Technical References

(C) - A list of technical references used in the evaluation

Technical references for the critical conditions analyzed are provided with the calculations in the appendices.

d. Location Restrictions

(D) - Completion of any applicable location restriction demonstrations in accordance with Rule .1622 of this Section

1. Airport Safety - the PLRC is not located within 5,000 feet of a runway used by piston-powered aircraft or within 10,000 feet of a runway used by turbine-powered aircraft. The landfill site is also not within a five-mile radius of any airport which supports either piston-powered or turbine-powered aircraft. The location map in the appendix verifies this information.

2. Floodplains - the PLRC landfill footprint is not located within the 100-year floodplain. It has no impact upon the flow or water storage capacity for this storm event. A map included in the appendix shows the location of the landfill site in relation to the 100-year floodplain.

3. Wetlands - Per a letter from Charles Hollis of the Army Corps of Engineers to Edward Gibson of PLRC dated March 9, 1988, the entire landfill site is "constructed on high ground with the exception of Sedimentation Pond No. 3". This sedimentation pond is located in "waters of the U.S." and is authorized by nationwide Permit No. 26. A copy of the correspondence between PLRC and the Corps is included in the appendix for this section. The facility is not located in wetlands.

4. Fault Areas - the PLRC is not located within 200 feet of a Holocene fault area. References for the fault area restriction are included in a letter to PLRC dated October 5, 1993. A copy of this letter is included in the appendices for this section.

5. Seismic Impact Zones - the PLRC is located within a seismic impact zone. A demonstration that the facility will not be adversely impacted by a seismic event in accordance with the regulations is included in Appendix A.

6. Unstable Areas - the landfill site has been analyzed to insure that the soil conditions are sufficient to provide adequate foundation support for the landfill area. Bearing capacity, bottom heave and settlement calculations have been previously performed for the site. Copies of these calculations are included in the appendices.

3. Construction Practices

NCAC 13B .1620(d)(3) - *A description of the materials and construction practices that conforms to the requirements set forth in Rule .1624 of this Section, and is consistent with the analysis of the facility design prepared in accordance with this Part.*

3.1 Base Liner System

NCAC 13B .1624 (b)(1) - The base liner system is constructed on the landfill subgrade and shall be designed to efficiently contain, collect, and remove leachate generated by the MSWLF unit. At a minimum, the components of the liner system shall consist of the following:

(A) A composite liner. The composite liner is one liner which consists of two components; a geomembrane liner installed above and in direct and uniform contact with a compacted clay liner. The composite liner shall be designed and constructed in accordance with Subparagraphs (8) and (9).

(B) A leachate collection system (LCS). The LCS is constructed directly above the composite liner and shall be designed to effectively collect and remove leachate from the MSWLF unit. The secondary function of the LCS is to establish a zone of protection between the composite liner and the waste. The LCS shall be designed and constructed in accordance with Subparagraphs (2),(10),(11), and 12.

A base liner system will be constructed on the PLRC subgrade. The main purpose of this system is to efficiently contain, collect, and remove leachate generated by the landfill. The liner system will consist of a composite liner and a leachate collection system. The composite liner consists of the following two components in ascending order: a two foot thick compacted cohesive soil layer with a maximum saturated hydraulic conductivity of 1×10^{-7} cm/sec overlain by a 60 mil HDPE geomembrane.

The leachate collection system will be constructed directly above the composite liner, and will consist of a HDPE geonet drainage layer , overlain by a geotextile filter/cushion, overlain by two feet of drainage/ protective soil with a minimum saturated hydraulic conductivity of $1x10^{-4}$ cm/sec. This collection system will be designed to effectively collect and remove leachate generated by the landfill. The function of the drainage/protective soil is to establish a zone of protection between the composite liner and the waste.

3.2 Leachate Collection System Design

NCAC 13B .1624 (b)(2)

(A) The leachate collection system shall be hydraulically designed to remove leachate from the landfill and ensure that the leachate head on the composite liner does not exceed one foot. A means of quantitatively assessing the performance of the leachate collection system under uniform conditions must be provided in the engineering plan. The performance analysis must evaluate the flow capacities of the pipe network necessary to convey leachate to the storage facility or off-site transport location. The engineering evaluation shall incorporate the following criteria:

(i) At a minimum, the impingement rate on the drainage layer shall be equal to the peak monthly precipitation rate to evaluate the relationship between base slope, drainage layer permeability, and collector pipe spacing.

(ii) At a minimum, the geometry of the landfill shall be designed to control and contain the volume of leachate generated by the 24-hour, 25-year storm.

(iii) Collection pipe flow capacity shall be sized to drain the critical volume of leachate generated by the 24-hour, 25-year storm in a specified period of time.

(B) The leachate collection system shall be designed to provide a zone of protection at least 24 inches thick separating the composite liner from landfilling activities.

(C) The leachate collection system shall include a drainage layer, a pipe network with clean-outs, and the necessary filters designed to

prevent physical clogging and promote leachate collection and removal from the landfill.

The leachate collection system for the PLRC is designed to be capable of hydraulically removing leachate from the landfill while ensuring that the leachate head on the composite liner is one foot or less under normal operating conditions.

Calculations were performed in order to determine the performance of the leachate collection system. These calculations included determining the maximum head anticipated on the liner and determining whether the geometry of the cells are capable of containing the 25-year, 24-hour storm (see attached appendix).

The evaluation of the leachate collection system incorporates several design criteria. The leachate collection system design criteria include provisions for leachate impingement rate, leachate volume, pipe flow capacity, zone of protection, and system components. For design purposes, the impingement rate on the drainage layer was equal to the peak monthly precipitation rate from on-site historic precipitation data and the HELP model. The relationship between this value and the base slope, drainage layer permeability, and collector pipe spacing was evaluated to determine leachate head. The geometry of the landfill was evaluated to control and contain the volume of leachate generated by the 24-hour, 25-year storm event. In addition, the collection pipe is a size capable of draining the volume of leachate generated by the 24-hour, 25-year storm event. The zone of protection separating the composite liner from landfilling activities is at least 24 inches thick. The collection system design will also provide for pipe network clean-outs necessary to prevent clogging and promote leachate collection and removal from the landfill.

Refer to the design drawings and the engineering calculations in the appendices for further detail.

3.3 Horizontal Separation Requirements

NCAC 13B .1624 (b)(3)

(A) Property line buffer. New MSWLF units at a new facility shall establish a minimum 300-foot buffer between the MSWLF unit and all property lines.

(B) *Private residences and wells. All MSWLF units at a new facility shall establish a minimum 500-foot buffer between the MSWLF unit and existing private residences and wells.*

(C) *Surface waters. All MSWLF units at new facilities shall establish a minimum 50-foot buffer between the MSWLF unit and any stream, river, or lake unless the owner or operator can demonstrate:*

(i) *To the Division that the alternative management of the water and any discharge will adequately protect the public health and environment; and*

(ii) *That the construction activities conform to the requirements of Sections 404 and 401 of the Clean Water Act.*

(D) *Existing landfill units. An adequate buffer distance shall be established between a new MSWLF unit and any existing landfill units to establish a ground-water monitoring system as set forth in Rule .1631 of this Section.*

(E) *Existing facility buffers. At a minimum, a lateral expansion or new MSWLF unit at an existing facility shall conform to the requirements of the effective permit.*

The PLRC is considered an existing facility with new units subject to the requirements of the existing facility. The currently permitted PLRC has a minimum 100 foot buffer between the limit of refuse and property boundary and 50 feet between the limit of refuse and Freeman Road right-of-way. This buffer extends around the entire permitted landfill area and will be maintained. Refer to the design drawings for disposal area locations.

3.4 Vertical Separation Requirements

NCAC 13B .1624 (b)(4) - *A MSWLF unit shall be constructed so that the post settlement bottom elevation of the base liner system is a minimum of four feet above the seasonal high ground-water table and bedrock. The nature of the materials establishing this separation shall be subject to Division approval.*

The unconstructed areas of the PLRC have been designed in a manner such that the post settlement bottom elevation of the base liner system

is a minimum of four feet above the seasonal high ground water table. According to the hydrogeologic investigation, it is not anticipated that competent bedrock will be encountered during excavation of the subgrade.

3.5 Survey Control

NCAC 13B .1624 (b)(5) - *One permanent benchmark of known elevation measured from a U.S. Geological Survey benchmark shall be established and maintained for each 50 acres of developed landfill, or part thereof, at the landfill facility. This benchmark shall be the reference point for establishing vertical elevation control.*

Permanent benchmarks of known elevations have been established at the PLRC. Benchmarks, which must be measured from a U.S. geological survey benchmark, will be established and maintained for every 50 acres of developed landfill. Therefore, at least two permanent vertical benchmarks will be established onsite. For a detailed description of the elevations, locations, and quantity of permanent benchmarks, see Sheet 1 - Facility Plan of the design drawings.

3.6 Location Coordinates

NCAC 13B .1624 (b)(6) - *The North Carolina State Plane (NCSP) Coordinates shall be established and one of its points shall be the benchmark of known NCSP coordinates.*

The North Carolina State Plane (NCSP) coordinates are established at the PLRC. The NCSP coordinates system is established for the facility benchmarks. The landfill design was based from the NCSP coordinate system. Refer to the design drawings for the NCSP coordinates for the facility.

3.7 Landfill Subgrade

NCAC 13B .1624 (b)(7) - *The landfill subgrade is the in-situ soil layer(s), constructed embankments, and select fill providing the foundation for construction of the unit. A foundation analysis shall be performed to determine the structural integrity of the subgrade to support the loads and stresses imposed by the weight of the landfill and to support overlying facility components and maintain their integrity of the components. Minimum post-settlement slope for the subgrade shall be two percent.*

Safety factors should be adequately specified for facilities located in a Seismic Impact Zones.

(A) Materials required. The landfill subgrade shall be adequately free of organic material and consist of in-situ soils or a select fill if approved by the Division.

(B) Construction requirements.

(i) The landfill subgrade shall be graded in accordance with the Division approved plans and specifications.

(ii) The owner or operator of the MSWLF units may be required by the permit to notify the Division's hydrogeologist and inspect the subgrade when excavation is completed or if bedrock or other unpredicted subsurface conditions are encountered during excavation.

(C) Certification requirements. At a minimum, the subgrade surface shall be inspected in accordance with the following requirements:

(i) before beginning construction of the base liner system, the project engineer shall visually inspect the exposed surface to evaluate the suitability of the subgrade and document that the surface is properly prepared and that the elevations are consistent with the Division approved engineering plans;

(ii) the subgrade shall be proof-rolled using specified procedures and equipment; and

(iii) the subgrade shall be tested for density and moisture content at a minimum frequency specified in the Division approved plans.

The PLRC subgrade consists of the in-situ soil and rock layers, constructed embankments, and select fill. The function of the subgrade is to provide the foundation for construction of the base liner system. In addition, the foundation supports the loads and stresses imposed by the weight of the refuse and miscellaneous facility structures. A foundation analysis of the subgrade at PLRC has been performed. The analysis includes calculations for bearing capacity, settlement and bottom heave. Refer to attached appendices. The

settlement analysis demonstrates that the post-settlement slopes will not fall below 2% minimum.

3.7.1 Material Requirements

In order to insure the structural integrity of the subgrade, the materials will be adequately free of organic material. The soil will consist of in-situ soils, bedrock, or select fill obtained onsite. When unpredicted subsurface conditions are encountered during the excavation, the landfill manager will notify the Division's hydrogeologist.

3.7.2 Construction/Certification Requirements

The landfill subgrade will be excavated to the grades presented on Sheet 2 of the design drawings. After the subgrade surface has been completed, it will be certified before base liner construction will begin. This certification will be by a registered professional engineer. The CQA consultant will evaluate the suitability of the subgrade and document that the surface is properly prepared and that the elevations are consistent with the approved design plans. In addition, the subgrade will be proof-rolled and tested for density and moisture content at a frequency specified in the facility Construction Quality Assurance (CQA) plan. Documentation of all inspections will also be completed in accordance with the CQA plan.

Refer to the facility CQA plan for further information on subgrade preparation.

3.8 Compacted Clay Liners

NCAC 13B .1624 (b)(8) - *Compacted clay liners are low permeability barriers designed to control fluid migration in a cap liner system or base liner system.*

(A) Materials required. The soil materials used in constructing a compacted clay liner may consist of on-site or off-site sources, or a combination of sources; sources may possess adequate native properties or may require bentonite conditioning to meet the permeability requirement. The soil material shall be free of particles greater than three inches in any dimension.

(i) *For the base liner system, the compacted clay liner shall be constructed with a minimum thickness of 24 inches (0.61 m) and a permeability of no more than 1 X 10^{-7} cm/sec.*

(ii) *For the cap system, the compacted clay liner shall be constructed with a minimum thickness of 18 inches (0.46 m) and a permeability of no more than 1 X 10^{-5} cm/sec.*

(B) *Construction requirements. Construction methods for the compacted clay liner shall be based upon the type and quality of the borrow source and shall be verified in the field by constructing test pad(s). The project engineer shall ensure that the compacted clay liner installation conforms with the Division approved plans including following minimum requirements:*

(i) *A test pad shall be constructed prior to beginning installation of the compacted clay liner and whenever there is a significant change in soil material properties. The area and equipment, liner thickness, and subgrade slope and conditions shall be representative of full scale construction. Acceptance and rejection criteria shall be verified for the tests specified in accordance with Subparagraph (C) of this Paragraph. For each lift, a minimum of three test locations shall be established for testing moisture content, density, and a composite sample for recompacted lab permeability. At least one shelby tube sample for lab permeability testing, or another in situ test approved by the Division, shall be obtained per lift.*

(ii) *Soil conditioning, placement, and compaction shall be maintained within the range identified in the moisture-density-permeability relation developed in accordance with Subparagraph (C) of this Paragraph.*

(iii) *The final compacted thickness of each lift shall be a maximum of six inches.*

(iv) *Prior to placement of successive lifts, the surface of the lift in place shall be scarified or otherwise conditioned to eliminate lift interfaces.*

(v) *The final lift shall be adequately protected from environmental degradation.*

(C) *Certification requirements. The project engineer shall include in the construction quality assurance report a discussion of all quality assurance and quality control testing required in this Subparagraph. The testing procedures and protocols shall be submitted in accordance with Rule .1621 of this Section and approved by the Division. The results of all testing shall be included in the construction quality assurance report including documentation of any failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of all retesting performed in accordance with the division approved plans including the following requirements:*

(i) *At a minimum, the quality control testing for accepting materials prior to and during construction of a compacted clay liner should include: particle size distribution analysis, Atterberg limits, triaxial cell laboratory permeability, moisture content, percent bentonite admixed with soil, and the moisture-density-permeability relation. The project engineer shall certify that the materials used in construction were tested according to the Division approved plans.*

(ii) *At a minimum, the quality assurance testing for evaluating each lift of the compacted clay liner shall include: moisture content and density, and permeability testing. For each location the moisture content and density shall be compared to the appropriate moisture-density-permeability relation. The project engineer shall certify that the liner was constructed using the methods and acceptance criteria consistent with test pad construction and tested according to the Division approved plans.*

(iii) *Any tests resulting in the penetration of the compacted clay liner shall be repaired using bentonite or as approved by the Division.*

Compacted clay liners consist of low-permeability soils which are designed to prevent fluid from entering or leaving the landfill unit. These liners will be used as components in both the final cover system and the base liner system.

3.8.1 Material Requirements

Soil materials used in constructing compacted clay liners for the landfill base liner and final cover will meet certain requirements. For the base liner system, the compacted clay liner will be constructed with a minimum thickness of 24 inches and a maximum hydraulic conductivity of 1×10^{-7} cm/sec. The liner will be free of particles greater than 2 inches in size in any dimension. For the final cover system, the compacted clay liner will be at least 18" thick, and have a maximum permeability of 1×10^{-5} cm/sec. Again, the clay liner will be free of any particles greater than 2 inches in size in any dimension. In general, the soils used for constructing the compacted clay liners may be obtained from on-site, off-site, or a combination of sources. In some cases, meeting the permeability requirements for the clays may involve bentonite conditioning.

It is anticipated that the final cover material will be obtained from on-site sources. The clay liner material is anticipated to be amended with bentonite. Four percent bentonite admixing with on-site soils has been utilized successfully on-site in the past. Past practices will be used with test pad verification to achieve amendment and construction requirements for density and permeability.

3.8.2 Construction Requirements

Varying construction methods for the compacted clay liner may be utilized depending upon the type and quality of the borrow source. Construction methods will be verified in the field by constructing test pads. A test pad will be constructed prior to beginning installation of the compacted clay liner and whenever there is a significant change in soil material properties. The equipment used, surface area, liner thickness and subgrade slope and conditions will be representative of full scale construction. After each test pad is constructed, a registered professional engineer will review its performance based on acceptance and rejection criteria as discussed in the next section. Once the test pad has been approved, construction of the appropriate full-scale liner may proceed. The test pad may be constructed in the cell area and be used as part of the compacted clay liner if it passes. If future soil materials remain consistent, a test pad may not be constructed, at the discretion of the Owner.

In addition to the verification of test pad performance, the engineer will ensure that the full-scale liner installation conforms with the approved design drawings and construction specifications. Refer to the CQA plan for minimum testing requirements for compacted clay liner installation and further test pad information. At a minimum, three test locations will be established for each clay liner lift for testing moisture content, density, and a composite sample for recompacted lab permeability. At least one shelby tube sample for lab permeability testing, or another in-situ test approved by the Division, shall also be obtained per lift. Soil conditioning, placement, and compaction will be maintained within the appropriate moisture-density-permeability range.

The maximum compacted clay liner lift thickness will be six inches. Prior to the placement of successive lifts, the surface of the lift in place will be scarified or otherwise conditioned to eliminate lift interfaces. Also, the final lift will be adequately protected from environmental degradation by covering with the remaining liner components or by utilizing other suitable means.

3.8.3 Certification Requirements

Compacted clay liner testing will be performed and documented pursuant to the facility CQA plan. The results of the testing will be provided to the Division in a CQA report and will include documentation of failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of retesting performed in accordance with the approved design drawings. Specific testing procedures will include provisions for accepting materials and lift evaluation.

The quality control tests for accepting materials prior to and during construction of a compacted clay liner will include the following:

1) Particle size distribution analysis;
2) Atterberg limits;
3) Triaxial cell laboratory permeability;
4) Moisture content;
5) Percent bentonite admixed with soil;
6) Moisture-density-permeability relation.

A registered professional engineer will certify that the materials used in construction were tested according to the approved CQA plan.

The quality assurance testing for evaluating each lift of the compacted clay liner will include moisture content, density, and permeability testing. For each test location, the moisture content and density will be compared to the appropriate moisture-density-permeability relation. A registered professional engineer will certify that the liner was constructed using the methods and acceptance criteria consistent with test pad construction and tested according to the CQA plan.

Testing procedures which result in the penetration of the compacted clay liner (such as shelby tube sampling) will be repaired using bentonite or a Division-approved alternative.

Refer to the facility CQA plan for further information on compacted clay liner installation.

3.9 Geomembrane Liners

NCAC 13B .1624(b)(9) - *Geomembrane liners are geosynthetic hydraulic barriers manufactured in sheets and installed by field seaming techniques.*

(A) Materials required. The geomembrane liner material shall have a demonstrated water vapor transmission rate of not more than 0.03 gm/m ^{2}-day. The liner material and any seaming materials shall have chemical and physical resistance not adversely affected by environmental exposure, waste placement and leachate generation. The type of geomembrane shall be acceptable to the Division.

(i) High density polyethylene geomembrane liners shall have a minimum thickness of 60 mils.

(ii) The minimum thickness of any geomembrane approved by the Division shall be greater than 30 mils.

(B) Construction requirements. The project engineer shall ensure that the geomembrane installation conforms to the requirements of the manufacturer's recommendations and the Division approved plans including the following:

(i) *the surface of the supporting soil upon which the geomembrane will be installed shall be reasonably free of stones, organic matter, protrusions, loose soil, and any abrupt changes in grade that could damage the geomembrane;*

(ii) *field seaming preparation and methods, general orientation criteria, and restrictive weather conditions;*

(iii) *anchor trench design;*

(iv) *critical tensile forces and slope stability;*

(v) *protection from environmental damage; and*

(vi) *physical protection from the materials installed directly above the geomembrane.*

(C) *Certification requirements. The project engineer shall include in the construction quality assurance report a discussion of the approved data resulting from the quality assurance and quality control testing required in this Subparagraph. The testing procedures and protocols for field installation shall be submitted in accordance with Rule .1621 of this Section and approved by the Division. The results of all testing shall be included in the construction quality assurance report including documentation of any failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of all retesting performed in accordance the Division approved plans including the following:*

(i) *quality control testing of the raw materials and manufactured product;*

(ii) *at a minimum, test seams shall be made upon each start of work for each seaming crew, upon every four hours of continuous seaming, every time seaming equipment is changed or if significant changes in geomembrane temperature and weather conditions are observed;*

(iii) *nondestructive testing of all seams; and*

(iv) field and independent laboratory destructive testing of seam samples.

Geomembrane liners are installed above the compacted clay liners to provide hydraulic barriers. The liners are installed utilizing field seaming techniques. The geomembrane liners will be used in both the final cover system and the base liner system.

3.9.1 Materials Requirements

The PLRC proposes the use of an HDPE geomembrane as a component of the base liner with a nominal thickness of 60 mils. A coextruded HDPE and VLDPE geomembrane is proposed as a component of the final cover. The coextruded geomembrane has a nominal thickness of 40 mils and consists of a core of VLDPE geomembrane layered between HDPE geomembrane. The VLDPE geomembrane occupies approximately sixty percent of the total thickness while each HDPE geomembrane layer occupies approximately twenty percent of the total thickness. The coextruded geomembrane is textured on both sides. The liner materials and any seaming materials will be protected from environmental exposure and waste placement. The materials will be compatible with the landfill leachate; both chemically and physically.

Stresses may be imposed on the geomembrane liners as a result of differential settlement of the landfill base or loading of the 2-foot thick base liner protective soils on the base grade sideslopes. The settlement calculations provided in Appendix G indicate that the maximum differential settlement anticipated under the landfill loading is insignificant (i.e., approximately ten inches of differential settlement over a 900-foot distance). This will result in very insignificant stresses on the base liner geomembrane. Therefore, the worst case loading on the geomembrane would be due to the initial application of protective soils to the base liner, prior to waste disposal. The weight of the protetive soils may transfer tensile forces to the underlying geomembrane liner.

Appendix K provides an analysis of the highest elevation to which the protective soils can be applied to the base liner without imposing tensile stresses on the geomembrane. The

results of the analysis indicate that protective soil can be applied to an elevation of approximately eight feet before stresses are imposed on the geomembrane. However, the maximum excavation depth is approximately ten feet (refer to the Base Grade Plan in the design drawings). This maximum depth occurs in small, localized areas within Phase 3 - Cell 1 - Subcells 1 and 3. Therefore, if protective soils were loaded on the entire sideslope, minor stresses may result in the geomembrane under full loading.

To prevent the development of tensile stresses within the geomembrane liner at these areas, the protective soils will not be applied over the entire sidewall. The soils will be applied in stages based upon the progression of filling within the cells. However, because filling is anticipated to occur at a rapid pace, and the initial waste lift height may exceed the sideslope height, it may be necessary to apply protective soils over the entire sideslope. If this need arises, extra protective soils will be mounded at the toe of slope to provide additional support for the soils on the sideslope.

Based on this information, the 60-mil HDPE geomembrane liner has sufficient strength and thickness for use in the landfill base liner under the varying landfill conditions.

3.9.2 Construction Requirements

Construction methods for the geosynthetics will be verified in the field by a professional engineer. The engineer will ensure that the geomembrane installation conforms to the requirements of the manufacturer's recommendations and the facility CQA plan. Included in the CQA plan are measures to ensure the following:

1) The surface upon which the geomembrane will be installed will be reasonably free of stones, organic matter, protrusions, loose soil, and abrupt grade changes;

2) Proper field seaming preparation and methods are applied, including general orientation criteria, and restrictive weather conditions;

3) Proper anchor trench construction;

4) Protection from environmental damage; and

5) Physical protection from the materials installed directly above the geomembrane.

3.9.3 Certification Requirements

Geomembrane testing will be performed and documented pursuant to the CQA plan. The results of the testing will be provided to the Division in a CQA report and will include documentation of failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of retesting performed in accordance with the approved design plans. Specific testing procedures will include the following:

(1) Quality control testing of the raw materials and manufactured product;

(2) Test seams shall be made upon each start of work for each seaming crew, upon every 4 hours of continuous seaming, every time seaming equipment is changed or if significant changes in geomembrane temperature and weather conditions are observed;

(3) Non-destructive testing of all seams; and

(4) Field and independent laboratory destructive testing of seam samples.

Refer to the facility CQA plan for detailed information on geomembrane liner installation procedures and testing.

3.10 Leachate Collection Pipes

NCAC 13B .1624 (b)(10) - A leachate collection pipe network shall be a component of the leachate collection system and shall be hydraulically designed to convey leachate from the MSWLF unit to an appropriately sized leachate storage or treatment facility or a point of off-site transport. Leachate collection piping shall comply with the following:

(A) Materials required.

(i) *The leachate collection piping shall have a minimum nominal diameter of six inches.*

(ii) *The chemical properties of the pipe and any materials used in installation shall not be adversely affected by waste placement or leachate generated by the landfill.*

(iii) *The physical properties of the pipe shall provide adequate structural strength to support the maximum static and dynamic loads and stresses imposed by the overlying materials and any equipment used in construction and operation of the landfill. Specifications for the pipe shall be submitted in the engineering report.*

(B) *Construction requirements.*

(i) *Leachate collection piping shall be installed according to the Division approved plan.*

(ii) *The location and grade of the piping network shall provide access for periodic cleaning.*

(iii) *The bedding material for the leachate collection pipe shall consist of a coarse aggregateinstalled in direct contact with the pipe. The aggregate shall be chemically compatible with the leachate generated and shall be placed to provide adequate support to the pipe. The bedding material for main collector lines should be extended to and in direct contact with the waste layer or a graded soil or granular filter.*

(C) *Certification requirements. The project engineer shall include in the construction quality assurance report a discussion of the quality assurance and quality control testing to ensure that the material is placed according to the approved plans. The testing procedures and protocols for field installation shall be submitted in accordance with Rule .1621 of this Section and approved by the Division. The results of all testing shall be included in the construction quality assurance report including documentation of any failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of all retesting performed in accordance the Division approved plans including the following:*

(i) *All leachate piping installed from the MSWLF unit to the leachate storage or treatment facility shall be watertight.*

(ii) *The seal where the piping system penetrates the geomembrane shall be inspected and non-destructively tested for leakage.*

A significant component of the leachate collection system at the PLRC is the leachate collection pipe network. This pipe network is designed to convey leachate from the landfill to a leachate storage or treatment facility on the site, or to a point of off-site transport.

3.10.1Material Requirements

The leachate collection pipe on the landfill floor will be six inch diameter and perforated. The pipe will be either Schedule 80 PVC or SDR 17 HDPE pipe. These materials are widely accepted and used in landfill applications. Tests have demonstrated that these materials are compatible with leachate typically encountered in the landfill. These materials have been used effectively in the existing units of this facility. Physical properties of the piping which are of concern involve structural strength. Calculations in the attached appendices have demonstrated that the leachate piping will support the maximum stresses imposed by the overlying materials at closed conditions and under maximum landfill height. Specifications for the leachate piping are provided in the appendices.

3.10.2Construction Requirements

The leachate collection piping layout is provided in Sheet 4 of the design drawings. The piping will be installed in accordance with these plans. As shown on the design drawings, the location and grade of the piping network provide access for periodic cleaning through the pipe "clean-out" risers. In addition, the drawings show that the pipe bedding material is a coarse aggregate, which will be installed in direct contact with the pipe. This will provide lateral support for the pipes under landfill loads. The aggregate also provides an additional pathway for leachate collection in the collection system. The calcium carbonate content in the aggregate will be limited to ensure the aggregate is chemically compatible with the leachate generated in the landfill. The bedding material for the main collector

lines will be extended to and in direct contact with the 2 foot protective soil/drainage layer. A geotextile filter is wrapped around the aggregate to prevent the infiltration of fine grained materials.

3.10.3Certification Requirements

Leachate collection pipe installation testing will be performed and documented pursuant to the CQA plan. The results of the testing will be provided to the Division in a CQA report and will include documentation of failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of all retesting performed in accordance with the approved design plans. Specific testing procedures will include provisions for leak prevention. The leachate transfer piping (i.e., forcemain) installed outside the landfill transporting leachate to the storage facility will be watertight and double contained. The leachate collection piping will not penetrate the geomembrane liner in a way which might compromise the integrity of the liner.

Refer to the facility CQA plan for further information on pipe installation.

3.11 Drainage Layers

NCAC 13B .1624 (b)(11) - *Any soil, granular, or geosynthetic drainage nets used in the leachate collection system shall conform to the following requirements:*

(A) Materials required.

(i) The chemical properties of the drainage layer materials shall not be adversely affected by waste placement or leachate generated by the landfill.

(ii) The physical and hydraulic properties of the drainage layer materials shall promote lateral drainage of leachate through a zone of relatively high permeability or transmissivity under the predicted loads imposed by overlying materials.

(B) Construction requirements.

(i) The drainage layer materials shall be placed according to the Division approved plans and in a manner which prevents equipment from working directly on the geomembrane.

(ii) The drainage layer materials shall be stable on the slopes specified on the engineering drawings.

(C) Certification requirements. The project engineer shall include in the construction quality assurance report a discussion of the quality assurance and quality control testing to ensure that the drainage layer material is placed according to the approved plans. The testing procedures and protocols for field installation shall be submitted in accordance with Rule .1621 of this Section and approved by the Division. The results of all testing shall be included in the construction quality assurance report including documentation of any failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of all retesting performed in accordance the Division approved plans.

Soil or granular layers, or geosynthetic drainage nets used in the leachate collection system at the PLRC are referred to as drainage layers. The drainage layers will conform to specific material, construction, and certification requirements.

3.11.1Material Requirements

The PLRC proposes to use both HDPE drainage net and granular drainage material for leachate drainage. Material requirements for the landfill drainage layers involve both chemical, physical, and hydraulic properties. The chemical properties of the drainage layer materials used will be such that they will not be adversely affected by waste placement or leachate generated by the landfill. The physical and hydraulic properties of the drainage layer material will promote lateral drainage of leachate through a zone of high permeability under the predicted loads imposed by the landfill facility. The lateral drainage of leachate will be performed while maintaining liquid head levels below one foot.

3.11.2Construction Requirements

Drainage layer materials will be placed according to the facility CQA plan. In addition, the installation will proceed in a manner which prevents equipment from working directly on the geomembrane. Evaluations have been performed to ensure that the landfill drainage layers will be stable at critical slopes and loads.

3.11.3Certification Requirements

Drainage layer placement documentation will be performed and documented pursuant to the CQA plan. The results of the testing will be provided to the Division in a CQA report and will include documentation of failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of retesting performed in accordance with the approved plans.

Refer to the facility CQA plan for further information on drainage layer installation.

3.12 Filter Layer Criteria

NCAC 13B .1624 (b)(12) - *All filter collection layers used in the leachate collection system shall be designed to prevent the migration of fine soil particles into a courser grained material, and permit water or gases to freely enter a drainage medium (pipe or drainage layer) without clogging.*

(A) Materials required.

(i) Graded cohesionless soil filters. The granular soil material used as a filter shall have no more than five percent by weight passing the No. 200 sieve and no soil particles larger than three inches in any dimension.

(ii) Geosynthetic filters. Geosynthetic filter materials shall demonstrate adequate permeability and soil particle retention, and chemical and physical resistance which is not adversely affected by waste placement, any overlying material or leachate generated by the landfill.

(B) Construction requirements. All filter layers shall be installed in accordance with the approved engineering plan and specifications.

Geosynthetic filter materials shall not be wrapped directly around leachate collection piping.

(C) Certification requirements. The project engineer shall include in the construction quality assurance report a discussion of the quality assurance and quality control testing to ensure that the filter layer material is placed according to the approved plans. The testing procedures and protocols for field installation shall be submitted in accordance with Rule .1621 of this Section and approved by the Division. The results of all testing shall be included in the construction quality assurance report including documentation of any failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of all retesting performed in accordance with the Division approved plans.

Filter layers used in the leachate collection system serve two basic purposes. One purpose of the filter collection layer is to prevent the migration of fine grained soil particles into courser grained materials or HDPE drainage net. The second purpose of the filter is to permit liquid or gases to enter a drainage medium without clogging. An additional purpose is to provide a cushion for the overburden of the landfill above the 60 mil HDPE geomembrane on the base liner.

3.12.1Material Requirements

A granular, cohesionless soil may used as a filter. The granular soil material used as a filter will have a maximum of 5% by weight passing the No. 200 sieve, and no particles larger than 3 inches in any dimension.

Geosynthetic filter materials (i.e., geotextile) will demonstrate adequate permeability and soil particle retention. In addition, the geotextile will have chemical and physical resistance to the adverse effects of waste placement, overlying material, and leachate generation. The geotextile proposed for the PLRC will be non-woven and will have a weight of 16 oz/sq yd. Geotextiles of this nature will function as adequate filters (see geotextile clogging calculation in appendices) and have adequate permeability.

3.12.2Construction Requirements

All filter layer materials will be installed according to the facility CQA plan and design drawings. In addition, geosynthetic filter materials will not be wrapped directly around leachate collection piping.

3.12.3Certification Requirements

Drainage layer placement documentation will be performed pursuant to the CQA plan. The results of the testing will be provided to the Division in a CQA report and will include documentation of failed test results, descriptions of the procedures used to correct the improperly installed material, and statements of retesting performed in accordance with the approved plans.

Refer to the facility CQA plan for further information on filter media installation.

3.13 Special Engineering Structures

NCAC 13B .1624 (b)(13) - *Engineering structures incorporated in the design and necessary to comply with the requirements of this Section shall be specified in the engineering plan. Material, construction, and certification requirements necessary to ensure that the structure is constructed according to the design and acceptable engineering practices shall be included in the Division approved plan.*

No special engineering structures have been incorporated into the design. If special engineering structures were to be incorporated in the future, material, construction, and certification requirements necessary to ensure compliance with the design and acceptable engineering practices will be added to the Engineering Report upon Division approval.

3.14 Sedimentation and Erosion Control

NCAC 13B .1624 (b)(14) - *Adequate structures and measures shall be designed and maintained to manage the run-off generated by the 24-hour, 25-year storm event, and conform to the requirements of the Sedimentation Pollution Control Law (15A NCAC 4).*

Stormwater Management

The proposed stormwater management system modifications are consistent with the existing stormwater management system plan in that stormwater will be routed to existing on-site sedimentation basins.

Calculation and runoff modeling were performed to confirm that the sediment basins are adequately sized. Since the landfill cover and footprint were not changed significantly the existing stormwater management facilities are adequate. The post-development peak flows were calculated using a stormwater management/sediment control computer program called SEDCAD. SEDCAD is based on the same criteria as the U.S. Soil Conservation Service TR-55. Hydrographs are calculated for a design storm by input of a drainage area, time of concentration, and SCS curve number. Times of concentration are calculated using the Upland Curve Method.

Following is a summary of the peak flows and maximum stage for the 25-year, 24-hour storm. See Appendix Q for computer model results.

SED BASIN NO.	PEAK OUTFLOW (CFS)	PEAK STAGE FLOW (FT)
1	59	744.3
2	35	753.9
3	55.7	758.7

Landfill operating practices and design features will prevent the contact of surface water runoff and groundwater with landfill waste. Operating cover will be provided to prevent rainfall and runoff from contacting waste to the extent possible. Any rainfall which contacts the waste will not be allowed to enter the surface water retention system and will be treated as leachate. Surface water will be conveyed to sedimentation basins with water elevations no higher than the top of the liner. Also, the landfill is constructed with a composite liner system to prevent the migration of leachate into groundwater. Thus, any water reaching the stormwater retention areas will be considered rainfall and will not contaminate the groundwater from percolation.

Erosion Control Measures

Erosion will be controlled in a manner consistent with the existing Sediment and Erosion Control Plan. Surface water will be routed to the retention areas from the final cover through a series of drainage ditches. The three types of ditches are: final cover drainage swales,

access road ditches, and perimeter ditches. Each of these ditches are designed to carry peak flows for the 25-year, 24-hour design storm event without overtopping. The location of the drainage features is shown on the final development plan. Calculations for the drainage features are included in Appendix Q.

The final cover drainage swales will be constructed on the final cover to intercept runoff on the 4H to 1V side slopes of the landfill cover. The drainage swales will be spaced every 40 feet vertically and sloped to access road ditches or perimeter ditches at 1 percent. The drainage swales are designed to convey the peak flow for the 25-year storm event. All permanent swales will be grass lined to prevent erosion.

3.15 Construction Quality Assurance (CQA) Report

NCAC 13B .1617(e)(3) - A construction quality assurance plan prepared in accordance with Rule .1621 of this Section.

(see Construction Quality Assurance Plan)

4. Design Hydrogeologic Report

NCAC 13B .1620(d)(4) - *A copy of the design Hydrogeologic Report prepared in accordance with Paragraph (b) of Rule .1623.*

~~A Phase II Hydrogeologic Investigation was performed by Golder Associates, Inc., in June 1988 for the PLRC. This report was submitted and approved by NCDEHNR. The site conditions at the landfill have changed since this report was completed; however, soil data information is still applicable and used for this design.~~

~~Revised groundwater information was used for the design of the new units based on site specific groundwater elevations taken over a 4 year period from the onsite monitoring wells to develop a seasonal high groundwater table map. See the Water Quality Monitoring plan for more detailed discussion.~~

A design Hydrogeologic Report prepared by RUST Environment and Infrastructure, Inc., dated March 1994, was submitted to NCDHENR under separate cover.

APPENDIX A

Location Restriction Standards

Airport Demonstration
Floodplain Demonstration
Wetlands Demonstration
Seismic Impact Zones, Unstable Conditions Demonstration and Fault Areas

Seismic Impact Zones
Unstable Conditions Demonstration
and
Fault Areas

ADDITIONAL SEISMIC STABILITY ANALYSES

ADDENDUM TO APPENDIX A OF THE TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER

INTRODUCTION

Waste Management of Carolinas, Inc., currently owns and operates the Piedmont Landfill and Recycling Center (Piedmont, or Site) in Kernersville, North Carolina. In response to the recently approved North Carolina Solid Waste Regulations, a Transition Application and Permit Renewal (Permit) was submitted to the State of North Carolina in December, 1993.

Appendix A of this Permit contains the Location Restrictions Standards demonstration regarding seismic impact zones, unstable conditions, and fault areas. Compliance with RCRA Subtitle D requirements regarding seismic impact zones was demonstrated in the Seismic Stability Report prepared by RUST Environment & Infrastructure. This report was dated May, 1993 and was included in Appendix A.

In the May, 1993 report, pseudo-static slope stability analyses were performed for three cross sections of the landfill. These cross sections were selected to represent the range of typical conditions that exist at the facility. The results of the analyses indicate that adequate factors of safety exist under static conditions. However, factors of safety under seismic conditions were found to be unacceptably low, below 1.0 for two of the cross sections.

As a result of the factors of safety being below 1.0, a displacement analysis was performed to determine how much movement the landfill would undergo during the design seismic event. The calculations indicated that the landfill would undergo less than 0.1 inches of displacement during the design earthquake. The report concluded that this small amount of displacement could be readily accommodated by the landfill without endangering the liner or leachate collection systems.

LANDFILL REDESIGN

After submittal of the Permit to the State, ongoing hydrogeologic work determined that the seasonal high groundwater level below parts of the Phase 3 area were higher than originally believed. To maintain the separation between the liner system and seasonal high groundwater specified in State Regulations, the base grades in parts of the Phase 3 area were modified. Because of this, a review of the stability analysis contained in the Permit has been performed to determine the effect of the change in base grades on slope stability factors of safety.

Cross Section 1. Along Cross Section 1, the base grades were not significantly altered. The grades below the higher portion of the cross section were slightly flattened. Because of this grade flattening, the base grades do not slope as sharply towards the perimeter berm. This is expected to lead to a more stable configuration, resulting in higher factors of safety. Therefore, it was determined that a reanalysis was not necessary for Cross Section 1.

Cross Section 2. Along Cross Section 2, the floor was raised fairly uniformly. This resulted in a reduced thickness of waste as well as a reduced height of perimeter berm. The reduced waste height would lead to less driving force and, hence, a more stable situation. However, the reduced height of perimeter berm would provide less containment for the waste, leading to a less stable configuration. Due to the uncertainty regarding which effect would predominate, it was determined that a reanalysis should be performed.

Cross Section 3. Only minor changes occurred along Cross Section 3. These changes were expected to have an insignificant effect on stability, so it was determined that a reanalysis was not necessary.

SLOPE STABILITY REANALYSIS

Computer slope stability analysis of Cross Section 2 was performed using PCSTABL 5M, a slope stability program developed at Purdue University. As with the analysis performed in the Permit, stability analysis was performed by specifying blocks to force the failure plane through the intraliner components. PCSTABL 5M uses the Simplified Janbu method to evaluate potential block failures.

PCSTABL 5M allows the use of earthquake coefficients (expressed as a percentage of gravity) in order to generate a pseudo-static representation of earthquake effects within the limit equilibrium model. A direct relationship is assumed to exist between the pseudo-static earthquake force acting on the sliding mass and the weight of the sliding mass. As in the calculations performed in the Permit, a horizontal earthquake coefficient of 15 percent of gravity directed outward from the slope face was used for the seismic analyses.

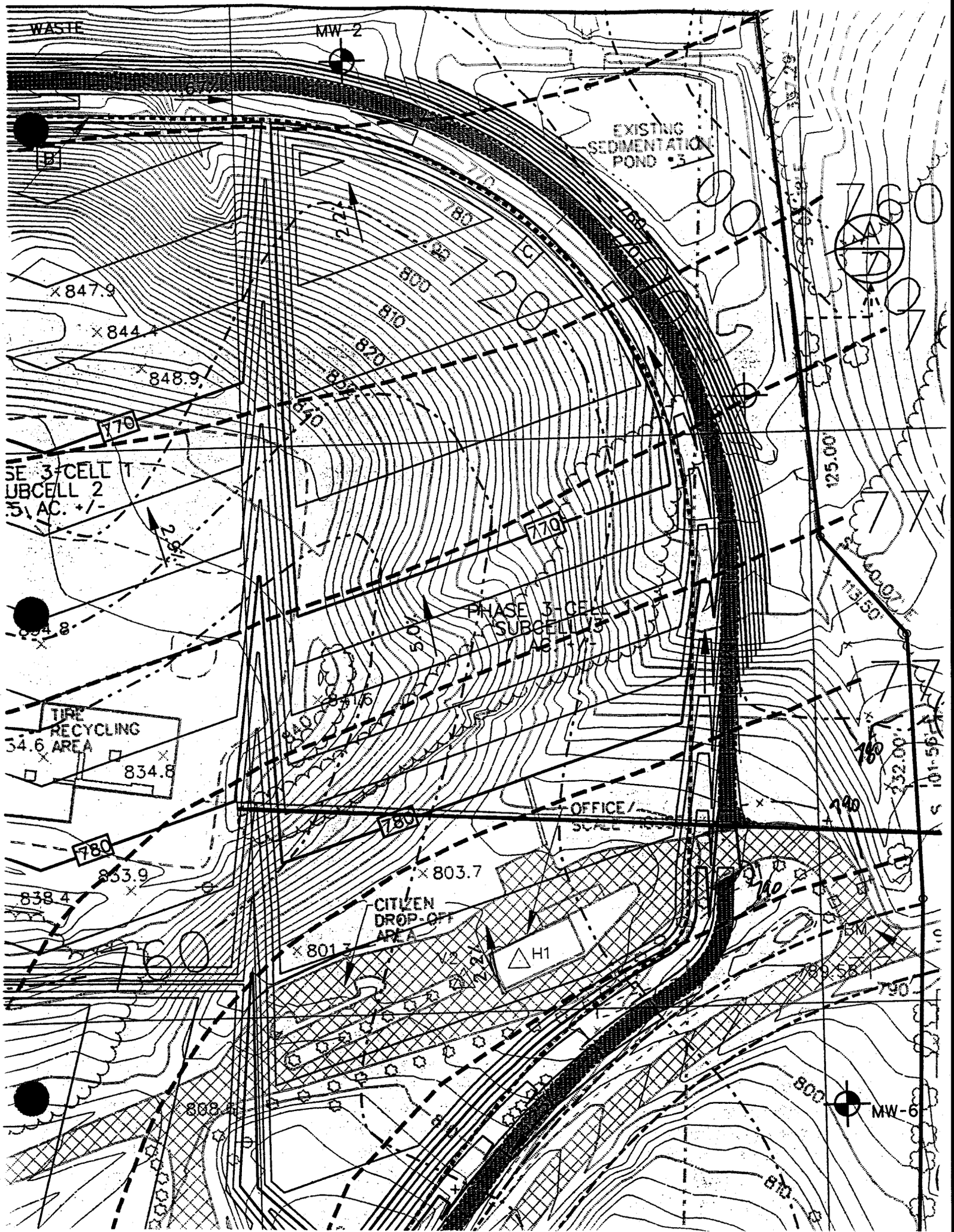
As mentioned above, the analysis was performed by placing blocks to direct the failure plane through the intraliner components. Active and passive zones are generated by the program to complete the failure surface. The active and passive zones were generated using Rankine earth pressure theory. The failure surface is divided into slices and the interslice shear forces are assumed to be zero. The factor of safety is defined as the ratio of the resisting forces and the driving forces. Moment equilibrium is generally not satisfied.

Parameters used to represent waste, soil, and liner interface strength were the same as those used for the calculations contained in the Permit. A full discussion of parameter selection is contained in that document.

RESULTS AND CONCLUSIONS

The factor of safety for Cross Section 2 calculated for this study was 1.9 for the static case and 0.9 for seismic conditions. Computer runs for static and seismic cases are attached. These factors of safety are slightly higher than those calculated in the Permit. This appears to be the result of smaller driving forces caused by a reduced thickness of waste.

Since the factors of safety calculated for this study are higher than the factors of safety calculated in the Permit, the conclusions in the Permit regarding the ability of the liner and leachate collection systems to accommodate motions associated with the design seismic event are unchanged.


WASTE
MW-2
EXISTING SEDIMENTATION POND #3
847.9
844.4
848.9
770
780
800
810
820
840
SE 3-CELL 1
UBCELL 2
AC. +/-
PHASE 3-CELL 1
SUBCELL 3
AC. +/-
TIRE RECYCLING AREA
834.8
OFFICE/
SCALE
803.7
CITIZEN DROP-OFF AREA
801.3
H1
833.9
838.4
808.4
125.00'
113.50'
232.00'
MW-6

Reviewed By:
Patrick J. Flood
4-8-94

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	4/8/94
Time of Run:	11:30 AM
Run By:	EJG
Input Data Filename:	87221-2A.DAT
Output Filename:	87221-2C.OUT

PROBLEM DESCRIPTION PIEDMONT LANDFILL; NEW BASEGRADES; DYNAMIC INTERLINER BLOCK FAILURE

BOUNDARY COORDINATES

12 Top Boundaries
48 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	82.50	100.00	76.00	1
2	100.00	76.00	157.00	70.00	1
3	157.00	70.00	176.00	90.00	1
4	176.00	90.00	178.00	90.00	4
5	178.00	90.00	184.00	90.00	3
6	184.00	90.00	260.00	100.00	2
7	260.00	100.00	469.00	140.00	2
8	469.00	140.00	521.00	150.00	2
9	521.00	150.00	635.00	180.00	2
10	635.00	180.00	698.00	190.00	2
11	698.00	190.00	1000.00	197.00	2
12	1000.00	197.00	1200.00	197.00	2
13	184.00	90.00	190.00	88.00	3
14	190.00	88.00	205.00	88.00	3
15	205.00	88.00	222.00	82.00	3
16	222.00	82.00	430.00	80.00	3
17	430.00	80.00	540.00	78.00	3
18	540.00	78.00	569.00	78.00	3
19	569.00	78.00	635.00	78.00	3
20	635.00	78.00	960.00	80.00	3
21	960.00	80.00	975.00	84.00	3
22	975.00	84.00	995.00	78.00	3
23	995.00	78.00	1045.00	76.00	3
24	1045.00	76.00	1200.00	73.50	3

25	178.00	90.00	190.00	86.00	4
26	190.00	86.00	205.00	86.00	4
27	205.00	86.00	222.00	80.00	4
28	222.00	80.00	430.00	78.00	4
29	430.00	78.00	540.00	76.00	4
30	540.00	76.00	569.00	76.00	4
31	569.00	76.00	635.00	76.00	4
32	635.00	76.00	960.00	78.00	4
33	960.00	78.00	975.00	82.00	4
34	975.00	82.00	995.00	76.00	4
35	995.00	76.00	1045.00	74.00	4
36	1045.00	74.00	1200.00	71.50	4
37	176.00	90.00	190.00	85.00	1
38	190.00	85.00	205.00	85.00	1
39	205.00	85.00	222.00	79.00	1
40	222.00	79.00	430.00	77.00	1
41	430.00	77.00	540.00	75.00	1
42	540.00	75.00	569.00	75.00	1
43	569.00	75.00	635.00	75.00	1
44	635.00	75.00	960.00	77.00	1
45	960.00	77.00	975.00	81.00	1
46	975.00	81.00	995.00	75.00	1
47	995.00	75.00	1045.00	73.00	1
48	1045.00	73.00	1200.00	70.50	1

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	135.0	135.0	.0	35.0	.00	.0	1
2	60.0	75.0	334.0	34.0	.00	.0	1
3	120.0	120.0	.0	36.0	.00	.0	1
4	100.0	100.0	.0	9.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 5 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	80.00
2	350.00	75.00
3	645.00	70.00

4	960.00	65.00
5	1200.00	62.00

A Horizontal Earthquake Loading Coefficient
Of .150 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

6 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 5.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	178.00	90.00	178.00	90.00	.00
2	190.00	85.50	190.00	85.50	1.00
3	205.00	85.50	205.00	85.50	1.00
4	222.00	79.50	222.00	79.50	1.00
5	430.00	77.50	430.00	77.50	1.00
6	540.00	75.50	635.00	75.50	1.00

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.94
3	205.00	85.89
4	222.00	79.86

5	430.00	77.20
6	617.25	75.01
7	619.48	79.49
8	622.97	83.07
9	626.14	86.94
10	629.61	90.54
11	631.68	95.09
12	634.69	99.08
13	638.21	102.63
14	641.75	106.17
15	643.40	110.89
16	644.68	115.72
17	646.60	120.34
18	649.70	124.26
19	651.96	128.72
20	655.48	132.27
21	659.02	135.81
22	659.18	140.81
23	662.70	144.35
24	663.15	149.33
25	664.87	154.02
26	667.45	158.31
27	668.97	163.07
28	672.42	166.69
29	674.64	171.16
30	675.61	176.07
31	678.21	180.34
32	680.98	184.50
33	683.21	187.65

*** .901 ***

Individual data on the 42 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	.0	.0	.0	.0	.0	.0	.0	.0	.0
2	6.0	729.2	.0	.0	.0	.0	109.4	.0	.0
3	6.0	1969.8	.0	.0	.0	.0	295.5	.0	.0
4	15.0	7131.0	.0	.0	.0	.0	1069.6	.0	.0
5	17.0	13356.4	.0	.0	.0	.0	2003.5	.0	.0
6	38.0	45638.4	.0	.0	.0	.0	6845.8	.0	.0
7	170.0	411370.6	.0	.0	.0	.0	61705.6	.0	.0
8	39.0	144465.7	.0	.0	.0	.0	21669.8	.0	.0
9	52.0	220911.2	.0	.0	.0	.0	33136.7	.0	.0
10	19.0	89565.9	.0	.0	.0	.0	13434.9	.0	.0
11	29.0	148308.9	.0	.0	.0	.0	22246.3	.0	.0
12	48.3	278368.9	.0	.0	.0	.0	41755.3	.0	.0
13	.5	3015.9	.0	.0	.0	.0	452.4	.0	.0
14	1.0	5964.1	.0	.0	.0	.0	894.6	.0	.0
15	.7	4326.4	.0	.0	.0	.0	649.0	.0	.0
16	3.5	19889.5	.0	.0	.0	.0	2983.4	.0	.0
17	3.2	17536.7	.0	.0	.0	.0	2630.5	.0	.0

18	3.5	18603.5	.0	.0	.0	.0	2790.5	.0	.0
19	2.1	10702.8	.0	.0	.0	.0	1605.4	.0	.0
20	3.0	14887.1	.0	.0	.0	.0	2233.1	.0	.0
21	.3	1499.3	.0	.0	.0	.0	224.9	.0	.0
22	3.2	15274.6	.0	.0	.0	.0	2291.2	.0	.0
23	3.5	16194.3	.0	.0	.0	.0	2429.1	.0	.0
24	1.7	7197.7	.0	.0	.0	.0	1079.7	.0	.0
25	1.3	5231.8	.0	.0	.0	.0	784.8	.0	.0
26	1.9	7353.8	.0	.0	.0	.0	1103.1	.0	.0
27	3.1	11109.5	.0	.0	.0	.0	1666.4	.0	.0
28	2.3	7607.2	.0	.0	.0	.0	1141.1	.0	.0
29	3.5	11081.2	.0	.0	.0	.0	1662.2	.0	.0
30	3.5	10497.2	.0	.0	.0	.0	1574.6	.0	.0
31	.2	436.6	.0	.0	.0	.0	65.5	.0	.0
32	3.5	8787.6	.0	.0	.0	.0	1318.1	.0	.0
33	.4	999.1	.0	.0	.0	.0	149.9	.0	.0
34	1.7	3416.1	.0	.0	.0	.0	512.4	.0	.0
35	2.6	4454.8	.0	.0	.0	.0	668.2	.0	.0
36	1.5	2228.5	.0	.0	.0	.0	334.3	.0	.0
37	3.5	4311.3	.0	.0	.0	.0	646.7	.0	.0
38	2.2	2291.5	.0	.0	.0	.0	343.7	.0	.0
39	1.0	735.6	.0	.0	.0	.0	110.3	.0	.0
40	2.6	1321.8	.0	.0	.0	.0	198.3	.0	.0
41	2.8	774.5	.0	.0	.0	.0	116.2	.0	.0
42	2.2	186.7	.0	.0	.0	.0	28.0	.0	.0

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.62
3	205.00	85.40
4	222.00	79.24
5	430.00	77.73
6	588.91	75.21
7	590.58	79.93
8	592.03	84.71
9	595.57	88.25
10	598.84	92.02
11	602.07	95.84
12	605.28	99.68
13	608.46	103.53
14	611.83	107.23
15	614.36	111.55
16	617.85	115.12
17	621.26	118.78
18	622.67	123.57
19	626.05	127.26
20	626.93	132.18
21	630.06	136.08
22	631.96	140.71
23	635.48	144.26
24	638.59	148.17
25	641.09	152.50
26	642.87	157.17
27	644.03	162.03
28	647.18	165.92

29	649.68	170.25
30	652.02	174.67
31	655.35	178.40
32	657.84	182.74
33	658.86	183.79

*** .903 ***

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Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.91
3	205.00	85.48
4	222.00	79.95
5	430.00	77.02
6	624.56	75.16
7	627.92	78.86
8	630.33	83.24
9	631.80	88.02
10	635.08	91.80
11	638.58	95.36
12	638.76	100.36
13	641.38	104.62
14	644.88	108.18
15	648.31	111.83
16	649.03	116.77
17	649.70	121.73
18	652.91	125.56
19	656.23	129.30
20	659.72	132.88
21	661.98	137.34
22	663.73	142.03
23	667.25	145.58
24	670.33	149.52
25	673.20	153.61
26	676.72	157.16
27	679.83	161.08
28	682.74	165.15
29	686.20	168.76
30	689.72	172.30
31	690.38	177.26
32	693.90	180.81
33	696.77	184.90
34	699.64	189.00
35	700.43	190.06

*** .912 ***

Failure Surface Specified By 31 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.38
3	205.00	85.26
4	222.00	79.85
5	430.00	78.00
6	559.27	75.30
7	562.71	78.92
8	566.22	82.48
9	569.07	86.60
10	571.89	90.72
11	575.13	94.53
12	578.39	98.32
13	580.45	102.88
14	583.92	106.48
15	587.26	110.20
16	590.38	114.11
17	592.99	118.37
18	596.52	121.92
19	598.20	126.62
20	601.73	130.17
21	602.88	135.03
22	603.25	140.02
23	605.91	144.25
24	609.42	147.81
25	612.69	151.59
26	615.20	155.92
27	618.35	159.81
28	621.72	163.50
29	623.72	168.08
30	625.76	172.65
31	625.81	177.58

*** .921 ***

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Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.82
3	205.00	85.98
4	222.00	79.97
5	430.00	77.00
6	624.84	75.04
7	626.89	79.60
8	628.17	84.44

9	631.38	88.27
10	632.84	93.06
11	633.14	98.05
12	636.59	101.67
13	639.68	105.60
14	640.01	110.59
15	643.40	114.27
16	644.55	119.13
17	645.92	123.94
18	649.21	127.71
19	652.60	131.38
20	655.22	135.64
21	658.45	139.46
22	661.03	143.74
23	662.95	148.36
24	666.47	151.90
25	668.38	156.52
26	671.19	160.66
27	674.20	164.66
28	677.70	168.22
29	679.66	172.82
30	680.78	177.69
31	684.27	181.27
32	687.22	185.31
33	690.74	188.85

*** .921 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.60
3	205.00	85.81
4	222.00	79.94
5	430.00	77.60
6	568.68	75.13
7	570.88	79.63
8	573.15	84.08
9	576.18	88.06
10	576.65	93.04
11	579.20	97.34
12	581.40	101.83
13	582.65	106.67
14	586.16	110.23
15	588.33	114.74
16	591.40	118.68
17	594.72	122.43
18	597.84	126.33
19	599.96	130.86
20	602.25	135.30
21	604.92	139.53
22	607.68	143.70

23	610.86	147.56
24	614.37	151.12
25	617.59	154.95
26	620.95	158.65
27	623.87	162.70
28	626.48	166.97
29	629.47	170.98
30	632.65	174.83
31	634.45	179.50
32	634.60	179.90

*** .922 ***

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Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.76
3	205.00	85.30
4	222.00	79.53
5	430.00	77.04
6	617.30	75.16
7	618.18	80.08
8	621.72	83.62
9	625.22	87.19
10	628.27	91.15
11	631.10	95.27
12	631.45	100.26
13	633.35	104.88
14	636.88	108.42
15	640.22	112.14
16	642.45	116.61
17	645.63	120.48
18	647.51	125.11
19	647.82	130.10
20	649.14	134.92
21	650.57	139.71
22	653.88	143.46
23	657.10	147.29
24	660.50	150.95
25	662.01	155.72
26	664.91	159.79
27	668.40	163.37
28	670.78	167.77
29	674.30	171.32
30	674.31	176.32
31	677.82	179.87
32	679.61	184.54
33	681.87	187.44

*** .923 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.31
3	205.00	85.35
4	222.00	79.17
5	430.00	77.11
6	579.77	75.26
7	581.64	79.90
8	584.51	84.00
9	586.71	88.49
10	589.91	92.32
11	590.39	97.30
12	593.82	100.94
13	597.34	104.49
14	600.82	108.08
15	604.23	111.73
16	607.77	115.27
17	610.90	119.17
18	613.84	123.21
19	617.17	126.95
20	618.97	131.61
21	622.11	135.50
22	622.30	140.50
23	623.16	145.42
24	626.69	148.96
25	628.18	153.73
26	630.99	157.87
27	632.43	162.66
28	634.41	167.25
29	637.53	171.15
30	639.28	175.84
31	640.97	180.54
32	641.29	181.00

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Failure Surface Specified By 31 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.81
3	205.00	85.66
4	222.00	79.88

5	430.00	77.56
6	563.77	75.56
7	566.97	79.40
8	569.90	83.45
9	573.43	86.99
10	575.87	91.35
11	577.24	96.16
12	580.76	99.71
13	581.74	104.62
14	583.47	109.30
15	586.91	112.94
16	590.35	116.57
17	593.15	120.71
18	594.85	125.41
19	598.21	129.11
20	600.90	133.33
21	603.01	137.86
22	606.16	141.75
23	607.97	146.40
24	609.03	151.29
25	612.51	154.88
26	615.71	158.72
27	618.83	162.63
28	619.47	167.59
29	621.62	172.10
30	624.62	176.10
31	624.71	177.29

*** .924 ***

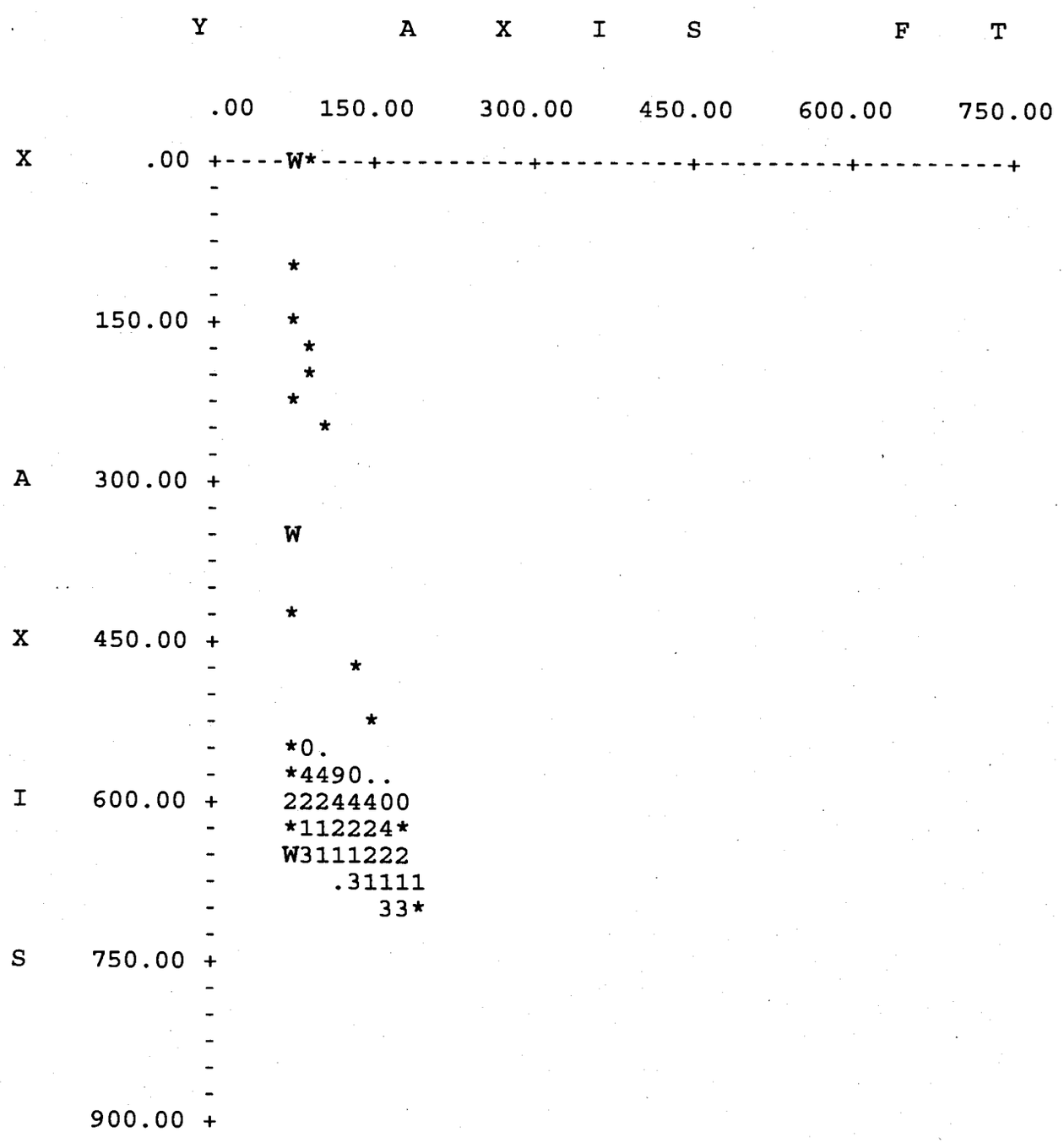
Failure Surface Specified By 30 Coordinate Points

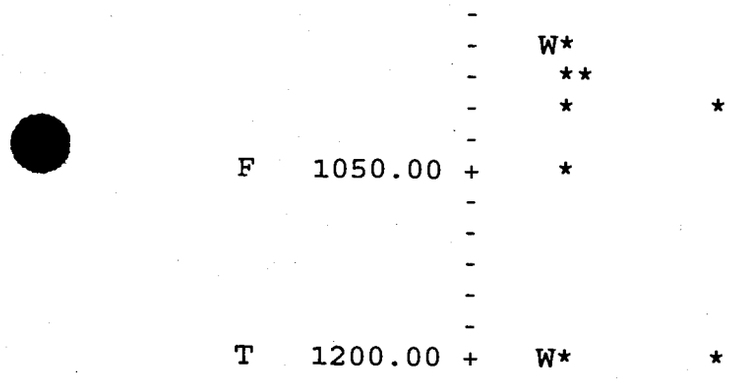
Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.32
3	205.00	85.13
4	222.00	79.33
5	430.00	77.29
6	548.78	75.08
7	552.19	78.75
8	554.75	83.04
9	558.26	86.60
10	561.50	90.41
11	564.38	94.50
12	566.36	99.09
13	567.84	103.86
14	569.33	108.64
15	572.66	112.37
16	575.63	116.39
17	576.44	121.32
18	579.82	125.01
19	582.97	128.90
20	585.63	133.13

21	589.08	136.74
22	592.09	140.74
23	594.83	144.92
24	596.55	149.61
25	599.70	153.50
26	603.21	157.06
27	606.14	161.11
28	607.45	165.94
29	610.83	169.62
30	610.83	173.64

*** .928 ***

1




W*
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F 1050.00 +
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T 1200.00 +
W*
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PROFIL
PIEDMONT LANDFILL; NEW BASEGRADES; DYNAMIC INTERLINER BLOCK FAILURE
48 12
0. 82.5 100. 76. 1
100. 76. 157. 70. 1
157. 70. 176. 90. 1
176. 90. 178. 90. 4
178. 90. 184. 90. 3
184. 90. 260. 100. 2
260. 100. 469. 140. 2
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1000. 197. 1200. 197. 2
184. 90. 190. 88. 3
190. 88. 205. 88. 3
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222. 82. 430. 80. 3
430. 80. 540. 78. 3
540. 78. 569. 78. 3
569. 78. 635. 78. 3
635. 78. 960. 80. 3
960. 80. 975. 84. 3
975. 84. 995. 78. 3
995. 78. 1045. 76. 3
1045. 76. 1200. 73.5 3
178. 90. 190. 86. 4
190. 86. 205. 86. 4
205. 86. 222. 80. 4
222. 80. 430. 78. 4
430. 78. 540. 76. 4
540. 76. 569. 76. 4
569. 76. 635. 76. 4
635. 76. 960. 78. 4
960. 78. 975. 82. 4
975. 82. 995. 76. 4
995. 76. 1045. 74. 4
1045. 74. 1200. 71.5 4
176. 90. 190. 85. 1
190. 85. 205. 85. 1
205. 85. 222. 79. 1
222. 79. 430. 77. 1
430. 77. 540. 75. 1
540. 75. 569. 75. 1
569. 75. 635. 75. 1
635. 75. 960. 77. 1
960. 77. 975. 81. 1
975. 81. 995. 75. 1
995. 75. 1045. 73. 1
1045. 73. 1200. 70.5 1
SOIL
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135. 135. 0. 35. 0. 0. 1
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0. 120. 0. 36. 0. 0. 1
100. 100. 0. 9. 0. 0. 1
WATER
1 62.4
5
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0. 80.
350. 75.
645. 70.
960. 65.
1300. 62.
EQUAKE
.15 0. 0.
BLOCK
0
100 6 5.
178. 90. 178. 90. 0.
190. 85.5 190. 85.5 1.
205. 85.5 205. 85.5 1.
222. 79.5 222. 79.5 1.
430. 77.5 430. 77.5 1.
540. 75.5 635. 75.5 1.
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Reviewed By:
Patrick T. Flood
4-8-94

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	4/7/94
Time of Run:	5:30 PM
Run By:	EJG
Input Data Filename:	87221-1A.DAT
Output Filename:	87221-1E.OUT

PROBLEM DESCRIPTION PIEDMONT LANDFILL; NEW BASEGRADES; STATIC INTERLINER BLOCK FAILURE

BOUNDARY COORDINATES

12 Top Boundaries
48 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	82.50	100.00	76.00	1
2	100.00	76.00	157.00	70.00	1
3	157.00	70.00	176.00	90.00	1
4	176.00	90.00	178.00	90.00	4
5	178.00	90.00	184.00	90.00	3
6	184.00	90.00	260.00	100.00	2
7	260.00	100.00	469.00	140.00	2
8	469.00	140.00	521.00	150.00	2
9	521.00	150.00	635.00	180.00	2
10	635.00	180.00	698.00	190.00	2
11	698.00	190.00	1000.00	197.00	2
12	1000.00	197.00	1200.00	197.00	2
13	184.00	90.00	190.00	88.00	3
14	190.00	88.00	205.00	88.00	3
15	205.00	88.00	222.00	82.00	3
16	222.00	82.00	430.00	80.00	3
17	430.00	80.00	540.00	78.00	3
18	540.00	78.00	569.00	78.00	3
19	569.00	78.00	635.00	78.00	3
20	635.00	78.00	960.00	80.00	3
21	960.00	80.00	975.00	84.00	3
22	975.00	84.00	995.00	78.00	3
23	995.00	78.00	1045.00	76.00	3
24	1045.00	76.00	1200.00	73.50	3
25	178.00	90.00	190.00	86.00	4

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0. 80.
350. 75.
645. 70.
960. 65.
1200. 62.
BLOCK
0
100 6 5.
178. 90. 178. 90. 0.
190. 85.5 190. 85.5 1.
205. 85.5 205. 85.5 1.
222. 79.5 222. 79.5 1.
430. 77.5 430. 77.5 1.
540. 75.5 635. 75.5 1.
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26	190.00	86.00	205.00	86.00	4
27	205.00	86.00	222.00	80.00	4
28	222.00	80.00	430.00	78.00	4
29	430.00	78.00	540.00	76.00	4
30	540.00	76.00	569.00	76.00	4
31	569.00	76.00	635.00	76.00	4
32	635.00	76.00	960.00	78.00	4
33	960.00	78.00	975.00	82.00	4
34	975.00	82.00	995.00	76.00	4
35	995.00	76.00	1045.00	74.00	4
36	1045.00	74.00	1200.00	71.50	4
37	176.00	90.00	190.00	85.00	1
38	190.00	85.00	205.00	85.00	1
39	205.00	85.00	222.00	79.00	1
40	222.00	79.00	430.00	77.00	1
41	430.00	77.00	540.00	75.00	1
42	540.00	75.00	569.00	75.00	1
43	569.00	75.00	635.00	75.00	1
44	635.00	75.00	960.00	77.00	1
45	960.00	77.00	975.00	81.00	1
46	975.00	81.00	995.00	75.00	1
47	995.00	75.00	1045.00	73.00	1
48	1045.00	73.00	1200.00	70.50	1

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	135.0	135.0	.0	35.0	.00	.0	1
2	60.0	75.0	334.0	34.0	.00	.0	1
3	120.0	120.0	.0	36.0	.00	.0	1
4	100.0	100.0	.0	9.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 5 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	80.00
2	350.00	75.00
3	645.00	70.00
4	960.00	65.00

5 1200.00 62.00

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

6 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 5.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	178.00	90.00	178.00	90.00	.00
2	190.00	85.50	190.00	85.50	1.00
3	205.00	85.50	205.00	85.50	1.00
4	222.00	79.50	222.00	79.50	1.00
5	430.00	77.50	430.00	77.50	1.00
6	540.00	75.50	635.00	75.50	1.00

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.62
3	205.00	85.40
4	222.00	79.24
5	430.00	77.73
6	588.91	75.21
7	590.58	79.93
8	592.03	84.71
9	595.57	88.25
10	598.84	92.02
11	602.07	95.84
12	605.28	99.68
13	608.46	103.53
14	611.83	107.23
15	614.36	111.55

16	617.85	115.12
17	621.26	118.78
18	622.67	123.57
19	626.05	127.26
20	626.93	132.18
21	630.06	136.08
22	631.96	140.71
23	635.48	144.26
24	638.59	148.17
25	641.09	152.50
26	642.87	157.17
27	644.03	162.03
28	647.18	165.92
29	649.68	170.25
30	652.02	174.67
31	655.35	178.40
32	657.84	182.74
33	658.86	183.79

*** 1.945 ***

Individual data on the 41 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	6.0	776.6	.0	.0	.0	.0	.0	.0	.0
2	6.0	2112.0	.0	.0	.0	.0	.0	.0	.0
3	15.0	7730.8	.0	.0	.0	.0	.0	.0	.0
4	17.0	14295.2	.0	.0	.0	.0	.0	.0	.0
5	38.0	47599.4	.0	.0	.0	.0	.0	.0	.0
6	170.0	410415.1	.0	.0	.0	.0	.0	.0	.0
7	39.0	142736.1	.0	.0	.0	.0	.0	.0	.0
8	52.0	219581.5	.0	.0	.0	.0	.0	.0	.0
9	19.0	89358.3	.0	.0	.0	.0	.0	.0	.0
10	29.0	148279.3	.0	.0	.0	.0	.0	.0	.0
11	19.9	110253.9	.0	.0	.0	.0	.0	.0	.0
12	.3	1579.5	.0	.0	.0	.0	.0	.0	.0
13	.7	3913.5	.0	.0	.0	.0	.0	.0	.0
14	.7	3656.7	.0	.0	.0	.0	.0	.0	.0
15	1.5	7523.5	.0	.0	.0	.0	.0	.0	.0
16	3.5	17523.1	.0	.0	.0	.0	.0	.0	.0
17	3.3	15719.6	.0	.0	.0	.0	.0	.0	.0
18	3.2	14912.1	.0	.0	.0	.0	.0	.0	.0
19	3.2	14246.6	.0	.0	.0	.0	.0	.0	.0
20	3.2	13554.2	.0	.0	.0	.0	.0	.0	.0
21	3.4	13753.1	.0	.0	.0	.0	.0	.0	.0
22	2.5	9823.7	.0	.0	.0	.0	.0	.0	.0
23	3.5	12948.1	.0	.0	.0	.0	.0	.0	.0
24	3.4	12067.8	.0	.0	.0	.0	.0	.0	.0
25	1.4	4690.5	.0	.0	.0	.0	.0	.0	.0
26	3.4	10485.6	.0	.0	.0	.0	.0	.0	.0
27	.9	2524.4	.0	.0	.0	.0	.0	.0	.0
28	3.1	8305.7	.0	.0	.0	.0	.0	.0	.0

29	1.9	4615.1	.0	.0	.0	.0	.0	.0	.0
30	3.0	6821.2	.0	.0	.0	.0	.0	.0	.0
31	.5	1028.6	.0	.0	.0	.0	.0	.0	.0
32	3.1	6371.0	.0	.0	.0	.0	.0	.0	.0
33	2.5	4572.2	.0	.0	.0	.0	.0	.0	.0
34	1.8	2803.1	.0	.0	.0	.0	.0	.0	.0
35	1.2	1514.7	.0	.0	.0	.0	.0	.0	.0
36	3.2	3347.7	.0	.0	.0	.0	.0	.0	.0
37	2.5	2100.7	.0	.0	.0	.0	.0	.0	.0
38	2.3	1416.3	.0	.0	.0	.0	.0	.0	.0
39	3.3	1284.3	.0	.0	.0	.0	.0	.0	.0
40	2.5	426.5	.0	.0	.0	.0	.0	.0	.0
41	1.0	27.2	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.94
3	205.00	85.89
4	222.00	79.86
5	430.00	77.20
6	617.25	75.01
7	619.48	79.49
8	622.97	83.07
9	626.14	86.94
10	629.61	90.54
11	631.68	95.09
12	634.69	99.08
13	638.21	102.63
14	641.75	106.17
15	643.40	110.89
16	644.68	115.72
17	646.60	120.34
18	649.70	124.26
19	651.96	128.72
20	655.48	132.27
21	659.02	135.81
22	659.18	140.81
23	662.70	144.35
24	663.15	149.33
25	664.87	154.02
26	667.45	158.31
27	668.97	163.07
28	672.42	166.69
29	674.64	171.16
30	675.61	176.07
31	678.21	180.34
32	680.98	184.50
33	683.21	187.65

*** 1.986 ***

Failure Surface Specified By 31 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.38
3	205.00	85.26
4	222.00	79.85
5	430.00	78.00
6	559.27	75.30
7	562.71	78.92
8	566.22	82.48
9	569.07	86.60
10	571.89	90.72
11	575.13	94.53
12	578.39	98.32
13	580.45	102.88
14	583.92	106.48
15	587.26	110.20
16	590.38	114.11
17	592.99	118.37
18	596.52	121.92
19	598.20	126.62
20	601.73	130.17
21	602.88	135.03
22	603.25	140.02
23	605.91	144.25
24	609.42	147.81
25	612.69	151.59
26	615.20	155.92
27	618.35	159.81
28	621.72	163.50
29	623.72	168.08
30	625.76	172.65
31	625.81	177.58

*** 1.994 ***

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.89
3	205.00	85.58
4	222.00	79.48
5	430.00	77.44
6	562.98	75.06
7	566.29	78.80
8	568.40	83.34
9	571.88	86.93

10	575.02	90.82
11	578.55	94.36
12	582.08	97.90
13	582.34	102.89
14	585.80	106.50
15	588.15	110.91
16	591.57	114.56
17	594.47	118.63
18	597.90	122.27
19	599.71	126.93
20	603.25	130.47
21	606.36	134.38
22	609.78	138.03
23	613.26	141.62
24	616.72	145.23
25	619.60	149.31
26	619.92	154.30
27	622.36	158.67
28	625.34	162.69
29	628.18	166.80
30	631.61	170.43
31	632.73	175.31
32	635.27	179.62
33	635.75	180.12

*** 2.033 ***

Failure Surface Specified By 31 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.81
3	205.00	85.66
4	222.00	79.88
5	430.00	77.56
6	563.77	75.56
7	566.97	79.40
8	569.90	83.45
9	573.43	86.99
10	575.87	91.35
11	577.24	96.16
12	580.76	99.71
13	581.74	104.62
14	583.47	109.30
15	586.91	112.94
16	590.35	116.57
17	593.15	120.71
18	594.85	125.41
19	598.21	129.11
20	600.90	133.33
21	603.01	137.86
22	606.16	141.75

23	607.97	146.40
24	609.03	151.29
25	612.51	154.88
26	615.71	158.72
27	618.83	162.63
28	619.47	167.59
29	621.62	172.10
30	624.62	176.10
31	624.71	177.29

*** 2.037 ***

Failure Surface Specified By 31 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.23
3	205.00	85.59
4	222.00	79.91
5	430.00	77.39
6	545.61	75.02
7	546.61	79.91
8	549.42	84.05
9	552.88	87.66
10	556.18	91.42
11	559.70	94.96
12	561.96	99.42
13	565.10	103.32
14	567.91	107.45
15	569.76	112.10
16	572.27	116.42
17	575.23	120.45
18	578.39	124.33
19	580.70	128.76
20	584.22	132.31
21	587.72	135.88
22	589.90	140.38
23	592.63	144.57
24	593.74	149.45
25	597.20	153.05
26	600.70	156.62
27	604.18	160.21
28	606.24	164.76
29	609.51	168.55
30	612.29	172.71
31	613.86	174.44

*** 2.038 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.60
3	205.00	85.81
4	222.00	79.94
5	430.00	77.60
6	568.68	75.13
7	570.88	79.63
8	573.15	84.08
9	576.18	88.06
10	576.65	93.04
11	579.20	97.34
12	581.40	101.83
13	582.65	106.67
14	586.16	110.23
15	588.33	114.74
16	591.40	118.68
17	594.72	122.43
18	597.84	126.33
19	599.96	130.86
20	602.25	135.30
21	604.92	139.53
22	607.68	143.70
23	610.86	147.56
24	614.37	151.12
25	617.59	154.95
26	620.95	158.65
27	623.87	162.70
28	626.48	166.97
29	629.47	170.98
30	632.65	174.83
31	634.45	179.50
32	634.60	179.90

*** 2.040 ***

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.91
3	205.00	85.48
4	222.00	79.95
5	430.00	77.02
6	624.56	75.16
7	627.92	78.86
8	630.33	83.24

9	631.80	88.02
10	635.08	91.80
11	638.58	95.36
12	638.76	100.36
13	641.38	104.62
14	644.88	108.18
15	648.31	111.83
16	649.03	116.77
17	649.70	121.73
18	652.91	125.56
19	656.23	129.30
20	659.72	132.88
21	661.98	137.34
22	663.73	142.03
23	667.25	145.58
24	670.33	149.52
25	673.20	153.61
26	676.72	157.16
27	679.83	161.08
28	682.74	165.15
29	686.20	168.76
30	689.72	172.30
31	690.38	177.26
32	693.90	180.81
33	696.77	184.90
34	699.64	189.00
35	700.43	190.06

*** 2.043 ***

1

Failure Surface Specified By 30 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.32
3	205.00	85.13
4	222.00	79.33
5	430.00	77.29
6	548.78	75.08
7	552.19	78.75
8	554.75	83.04
9	558.26	86.60
10	561.50	90.41
11	564.38	94.50
12	566.36	99.09
13	567.84	103.86
14	569.33	108.64
15	572.66	112.37
16	575.63	116.39
17	576.44	121.32
18	579.82	125.01
19	582.97	128.90

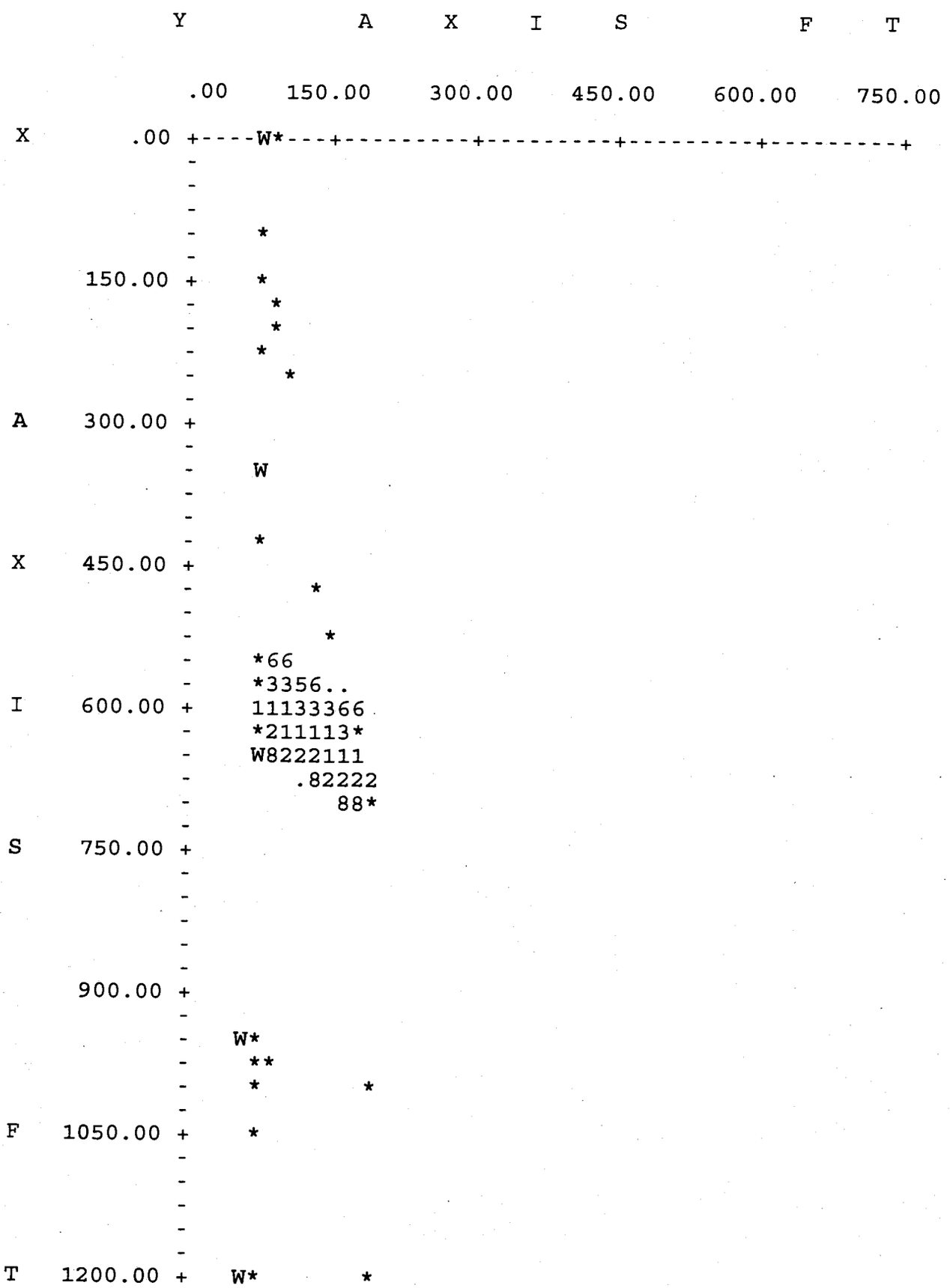
20	585.63	133.13
21	589.08	136.74
22	592.09	140.74
23	594.83	144.92
24	596.55	149.61
25	599.70	153.50
26	603.21	157.06
27	606.14	161.11
28	607.45	165.94
29	610.83	169.62
30	610.83	173.64

*** 2.056 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.00	90.00
2	190.00	85.31
3	205.00	85.35
4	222.00	79.17
5	430.00	77.11
6	579.77	75.26
7	581.64	79.90
8	584.51	84.00
9	586.71	88.49
10	589.91	92.32
11	590.39	97.30
12	593.82	100.94
13	597.34	104.49
14	600.82	108.08
15	604.23	111.73
16	607.77	115.27
17	610.90	119.17
18	613.84	123.21
19	617.17	126.95
20	618.97	131.61
21	622.11	135.50
22	622.30	140.50
23	623.16	145.42
24	626.69	148.96
25	628.18	153.73
26	630.99	157.87
27	632.43	162.66
28	634.41	167.25
29	637.53	171.15
30	639.28	175.84
31	640.97	180.54
32	641.29	181.00

*** 2.059 ***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I 600.00
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900.00
F 1050.00
T 1200.00

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PROFIL
PIEDMONT LANDFILL; NEW BASEGRADES; STATIC INTERLINER BLOCK FAILURE
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184. 90. 260. 100. 2
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521. 150. 635. 180. 2
635. 180. 698. 190. 2
698. 190. 1000. 197. 2
1000. 197. 1200. 197. 2
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190. 88. 205. 88. 3
205. 88. 222. 82. 3
222. 82. 430. 80. 3
430. 80. 540. 78. 3
540. 78. 569. 78. 3
569. 78. 635. 78. 3
635. 78. 960. 80. 3
960. 80. 975. 84. 3
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995. 76. 1045. 74. 4
1045. 74. 1200. 71.5 4
176. 90. 190. 85. 1
190. 85. 205. 85. 1
205. 85. 222. 79. 1
222. 79. 430. 77. 1
430. 77. 540. 75. 1
540. 75. 569. 75. 1
569. 75. 635. 75. 1
635. 75. 960. 77. 1
960. 77. 975. 81. 1
975. 81. 995. 75. 1
995. 75. 1045. 73. 1
1045. 73. 1200. 70.5 1
SOIL
4
135. 135. 0. 35. 0. 0. 1
60. 75. 334. 34. 0. 0. 1
[illegible]. 120. 0. 36. 0. 0. 1
[illegible]0. 100. 0. 9. 0. 0. 1
WATER
1 62.4
5
```

```
0. 80.
350. 75.
645. 70.
960. 65.
1200. 62.
BLOCK
0
100 6 5.
178. 90. 178. 90. 0.
190. 85.5 190. 85.5 1.
205. 85.5 205. 85.5 1.
222. 79.5 222. 79.5 1.
430. 77.5 430. 77.5 1.
540. 75.5 635. 75.5 1.
```

APPENDIX B

Site Life Calculation

APPENDIX B

PIEDMONT LANDFILL AIRSPACE TRACKING REPORT AS OF 1/31/94

Basis of Airspace Volumes

A) Remaining BCY*, Modules 1,2,5,6,1,2,3 as of 1/9/94, (no final cover):	1,099,749
B) Remaining BCY, full site, as of 1/9/94 (no final cover):	5,278,659
C) BCY consumed between 7/1/93 and 1/9/94** :	229,996
D) Gate Tons Received between 7/1/93 and 1/9/94,Tons:	147,318
E) Refuse/Int/Daily Cover Density Factor, (D/C)*2000,Gate lbs/Bank CY:	1,281

* Bank cubic yards (in-place cubic yards)

** Dates represent when topographic surveys took place

PRESENT ANALYSIS

F) Total Tons Landfilled between 1/9/94 and 1/31/94:	24,747
G) Total Gate Tons Received, This Month :	32,920
H) Tons Landfilled since 1/9/94, tons:	24,747
I) Remaining BCY Mods 1,2,5,6,1,2,3 as of this date, A-(H/(E/2000)):	1,061,113
J) Gate Tons Available, Modules 1,2,5,6,1,2,3, I*(E/2000):	679,669
K) Monthly Refuse Gate Rate, (for site life calc), TONS***:	61,967
L) MONTHS REMAINING, (J / K):	11
M) REMAINING TOTAL SITE LIFE, YRS , ((B-(H+(2000/E)))/(K*(2000/E)))/12:	4.5
N) REMAINING BCY, FULL SITE, AS OF THIS DATE, B-(H+(2000/E)):	5,240,023
0) Total Available Site Gate tons, (N+((Tons rec'd to date*2000))/E:	6,213,322
P) Monthly Average gate tons for last 6 months:	26,404
Q) Daily average gate tons for past 6 months:	1,108

*** Calculation based upon the following criteria:

a) Gate tons per day:	2,600
b) Days per week:	5.5
c) Weeks per year:	52
d) Latest density:	1,281

Piedmont Landfill & Recycling Center

April 1994

Revision 0

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 3
PROJECT NO. 87221.400

CLIENT REDMONT LF
SUBJECT SOIL BALANCE
PROJECT TRANSITION PLAN & PERMIT RENEWAL

Prepared By JKR Date 3/29/94
Reviewed By [illegible] Date 4/13/94
Approved By _____ Date _____
Revised By JKR 4/11/94

Problem: Determine quantities of soil material available on-site and needed for borrow to meet the soil requirement needs for the remaining site life.

Given: Approximately 30 acres of landfill filled ≈ 1,306,800 ft²
Approximately 38 acres remain to be filled. ≈ 1,655,280 ft²

Assumptions: Final Cover Profile (Soils only) 2' Protective Soil, 18" Compacted Soil ($K = 10^{-5}$)

Base Liner Profile (Soils only) 2' Protective Cover ($K = 10^{-4}$), 2' Compacted Soil ($K = 10^{-7}$) plus internal berms (See Attached)

Refuse to D/I Cover Rate: @ 2600 Tpd use 260 cy soil/day
Site historically used soil at a rate 10% of daily waste intake

Calculations: D/I Cover needs for existing fill area = 1,099,749 CY remain (from 1/9/94)
@ 1281 gate lb/cy ≈ 704,389 Gate Tons
∴ Need 70,439 cy soil

D/I Cover needs for proposed cells = 4,118,910 CY remain
@ 1281 gate lb/cy = 2638162 Gate Tons ∴ Need 263,816 CY so

Excavation Volume (Soil) = 1,587,245 CY
+ 60,400 CY
1,647,645 CY

[Available on site - suitable for D/I cover, protective cover (cap), compacted soil (liner) amended, compacted soil (cap)]

Existing Cells Soil Needs

D/I Cover = 70,440 cy (on site sources)

Final Cover: 2' Protective Soil ⇒ $(2')(1{,}306{,}800\ ft^2)\left(\frac{1}{27}\right) = 96{,}800$ cy (on site sources)
1.5' Compacted Soil (processed) ⇒ $96{,}800\ cy\left(\frac{1.5}{1}\right) = 72{,}600$ cy (on site sources)

∴ Needs from on site sources 239,840 cy

Proposed Cells Soils Needs

D/I Cover = 263,816 cy (on-site sources)

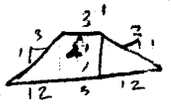
Final Cover = 2' Protective Soil ⇒ $(2')(1{,}655{,}280\ ft^2)(\frac{1}{27}) = 122{,}613$ cy (onsite source)
1.5' Compacted Soil ⇒ $122{,}613(\frac{1.5}{2}) = 91{,}960$ cy (on site source)
(processed)

Base Liner = 2' Protective Cover ⇒ 122,613 cy (obtained offsite)

2' Compacted Soil ⇒ 122,613 cy (.96) = 117,708 cy say 120,000 cy
(amended with approx. 4% by weight bentonite)
w/ compacted intercell berms = 127,800 CY
2' Compacted Soil ⇒ 122,613 cy (obtained offsite if not amended)
w/ compacted intercell berms = 200,613 cy

Structural Fill = 44,040 CY (computed from surface models developed in the CADD system)
(onsite sources)

Intercell Berm Volume = 3500 LF (60 SF) ≅ 7800 CY



$A = (4)(12) + 3(4) = 60$ SF

∴ Needs from on site sources 263,816 + 122,613 + 91,960 + 127,800 + 44040
= 650,229 CY

SOIL BALANCE: 1,647,645 − [239840 + 650229] = 757,576 CY EXCESS

Needs off site = 10^{-4} Protective Cover 122,613 cy
* 10^{-7} Compacted Soil 122,613 cy

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 3 OF 8
PROJECT NO. 81857.000

CLIENT WASTE MGT
PROJECT PIEDMONT

SUBJECT VOLUME CALCS
SOIL EXCAVATION

Prepared By ELK Date 9/12/93
Reviewed By CM Date 4/13/94
Approved By ____ Date ____

PHASE III CELL 1

ELEV (ft)	AREA (ft^2)	VOL CY	CUM VOL CY
848	2990 + 249.2	—	—
840	34140	5,537.7	5,537.7
830	234,000	49,655.6	55,193.3
820	360,840	110,155.6	165,348.9
810	523,070	163,687.0	329,035.9
800	[illegible]	205,764.8	534,800.7
790	645,675	228,469.4	763,270.1
780	494,164	211,081.3	974,351.4
770	28160 + 214063	136,368.0	1,110,719.4
760	500 + 90,210	61,654.3	1,172,373.7
756	5,732	7,143.9	1,179,517.5

TOTAL VOL CUT FOR PHASE I = 1,178,000 CY

SURFACES: JULY, 1993 TOPOGRAPHIC MAP
OLD BOTTOM OF REFUSE GRADE (PERMITTED)

PHASE III CELL II

756	14204	—	—
760	69030	6165.5	6,165.5
770	259,650	60,866.7	67,032.2
780	354115	113,660.2	180,692.4
790	290815	119,431.5	300,123.9
800	371800	122,706.5	422830.4
810	27412 + 78500	88,465.2	511295.6
820	7225	20,951.3	532246.9
823	1740	500	532745

TOTAL CUT PHASE II = 532,745 C.Y.

SURFACES: JULY 1993 TOPOGRAPHIC MAP
OLD BOTTOM OF REFUSE GRADE (PERMITTED

ADDITIONAL EXCAVATION

PIEDMONT LANDFILL ~~AIRSPACE GAIN~~

CONTOUR	AREA	AREA	DEPTH	VOLUME
754	0.55	5500		
			2	23148.3
756	1.90	19000		
			2	43863.0
758	2.50	25000		
			2	50996.7
760	2.60	26000		
			2	48969.4
762	2.30	23000		
			2	38254.1
764	1.55	15500		
			2	38254.1
766	2.30	23000		
			2	87031.6
768	6.80	68000		
			2	168197.6
770	10.13	101300		
			2	166368.2
772	6.63	66300		
			2	151873.5
774	8.60	86000		
			2	252178.9
776	17.1	171000		
			2	338494.0
778	16.75	167500		
			2	320898.2
780	15.35	153500		
			2	291362.5
782	13.8	138000		
			2	251101.3
784	11.35	113500		
			2	205124.1
786	9.2	92000		
			2	155155.5
788	6.4	64000		
			2	108443.8
790	4.5	45000		
			2	69583.2
792	2.55	25500		
			2	41132.7
794	1.6	16000		
			2	26307.7
796	1.05	10500		
			2	11388.4
798	0.2	2000		

TOTAL AIRSPACE: 106,968 CUBIC YARDS

4PM 4/13/94

SURFACES

NEW SUBGRADE PLAN

OLD BOTTOM OF REFUSE PLAN

TOTAL EXCAVATION VOLUME IN PHASE III:

1,178,000 CY
532,745 CY
76,500 CY
1,787,245 CY

~200,000 CY OF EXCAVATION VOLUME is ROCK. DATA ESTIMATE FROM SITE SOURCES.

SOIL VOLUME:
~ 1,587,245 CY

NET GAIN:
106,968
-30,474 From Log B
76,484 CY ✓

Jom 4/13/94
6 of 3

PIEDMONT LANDFILL ~~AIRSPACE LOSS~~ EXCAVATION LOSS

CONTOUR	AREA	AREA	DEPTH	VOLUME
756	0.20	2000		
			2	11793.6
758	1.10	11000		
			2	23484.0
760	1.25	12500		
			2	34055.4
762	2.20	22000		
			2	64741.8
764	4.40	44000		
			2	92463.5
766	4.85	48500		
			2	113098.1
768	6.50	65000		
			2	135955.9
770	7.10	71000		
			2	88348.1
772	2.20	22000		
			2	33498.7
774	1.20	12000		
			2	26460.6
776	1.45	14500		
			2	21049.8
778	0.7	7000		
			2	10861.0
780	0.4	4000		
			2	18666.7
782	1.6	16000		
			2	35925.7
784	2	20000		
			2	40896.7
786	2.09	20900		
			2	35525.0
788	1.48	14800		
			2	22003.8
790	0.76	7600		
			2	8999.1
792	0.2	2000		
			2	4966.3
794	0.3	3000		

TOTAL AIRSPACE: 30,474 CUBIC YARDS

CJM 4/13/94
7085

Wed Mar 30 10:34:20 1994 Page: 1
Triangle Volume Report
AEC Group, Inc. (c) 1991-92

Original Surface: 87221EXST.DTM 7/93 TOPOGRAPHIC MAP
Design Surface: 87221BASE.DTM SUBGRADE PLAN

Cut (cu yd)	Fill (cu yd)	Net (cu yd)
1780813.9009	44040.5815	1736773.3194

EXCAVATION VOLUME

STRUCTURAL FILL

(VERIFIES ATTACHED MANUAL CALCULATION)

FM 4/13/94
945

Mon Apr 11 08:18:45 1994 Page: 1

Triangle Volume Report

AEC Group, Inc. (c) 1991-92

Original Surface: 7221EXBS.DTM FORMER SUBGRADE PLAN (12/93)

Design Surface: 7221NWBS.DTM REVISED SUBGRADE PLAN (4/94)

Cut (cu yd)	Fill (cu yd)	Net (cu yd)
86190.6373	25790.1481	60400.4892

ADDITIONAL EXCAVATION VOLUME

APPENDIX C

Leachate Head Analysis

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 2
PROJECT NO. 87221.300

CLIENT SMCI
PROJECT PIEDMONT LRC

SUBJECT LEACHATE HEAD CALC.

Prepared By DMP Date 11/4/9
Reviewed By JTM Date 11/10/9
Approved By ____ Date ____

PURPOSE: DETERMINE THE LEACHATE HEAD BUILD-UP ON THE BASE OF THE LANDFILL AT PIEDMONT LRC THE PROPOSED DRAINAGE LAYER WILL BE A HDPE GEONET w/ TWO FEET PROTECTIVE SOIL

ASSUMPTIONS/EQUATIONS:

THE EQUATION USED WAS DERIVED BY DR. BRUCE M. McENROE (UNIV. OF KANSAS) IN THE ARTICLE ENTITLED "MAXIMUM SATURATED DEPTH OVER LANDFILL LINER, JOURNAL OF ENVIRONMENTAL ENGINEERING, (pg 262-27 MARCH/APRIL 1993. (SEE REF 1), AT FREE DRAINAGE BOUNDARY CONDITIONS:

① FOR $R < 1/4$ $\quad Y_{MAX} = (R - RS + R^2S^2)^{1/2} \left[\frac{(1-A-2R)(1+A-2RS)}{(1+A-2R)(1-A-2RS)}\right]^{1/2A}$

② FOR $R = 1/4$ $\quad Y_{max} = \left(\frac{R(1-2RS)}{(1-2R)}\right) e^{\left[\frac{2R(S-1)}{(1-2RS)(1-2R)}\right]}$

③ FOR $R > 1/4$ $\quad Y_{max} = \left[(R - RS + R^2S^2)^{1/2}\right] e^{\left[\frac{1}{B}\tan^{-1}\left(\frac{2RS-1}{B}\right) - \frac{1}{B}\tan^{-1}\left(\frac{2R-1}{B}\right)\right]}$

WHERE:

K = HYDRAULIC CONDUCTIVITY OF DRAINAGE LAYER = 17.5 cm/sec **
L = DRAINAGE DISTANCE, MEASURED HORIZONTALLY = 370'
r = VERTICAL INFLOW PER UNIT HORIZONTAL AREA = 6.1 in/mo *
$(6.1 \text{ in/mo})\left(\frac{1 \text{ mo}}{30 \text{ days}}\right)\left(\frac{1 \text{ day}}{24 \text{ hr}}\right)\left(\frac{1 \text{ hr}}{3600 \text{ sec}}\right)\left(\frac{2.54 \text{ cm}}{1 \text{ in}}\right) = 5.98 \times 10^{-6}$ cm/sec
α = LINER ANGLE, MEASURED FROM HORIZONTAL = 2%
$S \approx \tan\alpha \approx \sin\alpha$ = SLOPE OF LINER WHEN ANGLE IS SMALL = 0.
$R = r / (K \sin^2\alpha)$
$Y_{max} = y_{max} / (L \tan\alpha)$
y_{max} = MAXIMUM SATURATED DEPTH
$A = (1-4R)^{1/2}$
$B = (4R-1)^{1/2}$

(REF.

* MAX. MONTHLY FROM CURRENT RAINFALL DATA (SEE REF. 2)
** (SEE GEONET CONDUCTIVITY CALC) F.O.S OF 2 APPLIED TO K = 17.5 cm/sec

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PROJECT NO. 87221.300

CLIENT WMCI

PROJECT PIEDMONT LRC

SUBJECT LEACHATE HEAD CALC

Prepared By [illegible] Date 11/4/93 REV

Reviewed By [illegible] Date 10/20/93

Approved By ____ Date ____

REVISED BY [illegible] 4-8-94

CALCULATIONS

$$R = r/(K \sin^2\alpha) = (5.98 \times 10^{-6}\ \text{cm/sec})/[(17.5\ \text{cm/sec})(0.02)^2] =$$

$R = 0.00085429$; THUS $R < 1/4$, USE EQUATION ①

$$A = (1-4R)^{1/2} = (1-4(0.00085429))^{\frac{1}{2}} =$$

$$A = 0.99829$$

$$Y_{max} = (0.00085429-(0.00085429)(0.02)+(0.00085429)^2(0.02)^2)^{\frac{1}{2}}\left[\frac{(1-(0.99829)-2(0.00085429))}{(1+(0.99829)-2(0.00085429))}\right.$$

$$\left.\frac{(1+(0.99829)-2(0.00085429)(0.02)}{(1-(0.99829)-2(0.00085429)(0.02)}\right]^{\frac{1}{2(0.99829)}}$$

$$Y_{max} = (0.00083720)^{1/2}\left[\frac{(1.42\times10^{-6})(1.9983)}{(1.9966)(0.0016758)}\right]^{(0.50086)}$$

$$Y_{max} = (0.028934)(0.028945)$$

$$Y_{max} = 0.00083749$$

$$y_{max} = Y_{max}(L \tan\alpha) = (0.00083749)(465')(0.02)$$

$$\underline{\underline{y_{max} = .00779' = .093''}}$$

THE MAXIMUM LEACHATE HEAD BUILD-UP WILL NOT BE MORE THAN 0.093" UNDER "OPEN CELL" CONDITIONS. THIS REPRESENTS AN ASSUMED WORST CASE SCENARIO REGARDING IMPINGEMENT OF LEACHATE (OR STORM WATER) ON THE BASE LINER DRAINAGE LAYER.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF ____
PROJECT NO. 87221

CLIENT Piedmont LRC
SUBJECT Leachate Head Analysis
PROJECT Transition Application & Permit Renewal

Prepared By GJM Date 3/18/94
Reviewed By DNW Date 3/30/94
Approved By ____ Date ____

Purpose: To re-evaluate the leachate head estimate on the landfill base. The estimate was originally performed on 11/4/93 as part of the original permit application. However, based on comments by the State of North Carolina, Division of Solid Waste Management, the factor of safety applied to the geonet permeability will be increased to demonstrate continued performance of the leachate collection system.

Givens & Assumed:

- Permeability = k = 35 cm/sec for geonet under the "open cell" conditions. These conditions represent <u>worst case</u> scenario for potential leachate head build-up.

- Factor of safety to account for potential clogging and degradation of the geonet will be increased from 2 (from 11/4/94 calcs) to 4.

  i.e. $FOS = 4 = \frac{35}{X}$ $\quad X = 8.75 \text{ cm/sec} = k$

  $k = 8.75 \text{ cm/sec}$

- Thickness of geonet $\cong$ 0.2 inches

- Leachate Head Calculation, developed by Dr. Bruce M. McEnroe (of the University of Kansas) in a published article titled "Maximum Saturated Depth Over Landfill Liner", published in the <u>Journal of Environmental Engineering</u> (pp. 262-270), March/April 1993. (REF. 1)

  ① For $R < 1/4$ $\quad y_{max} = (R - RS + R^2S^2)^{1/2} \left[\frac{(1-A-2R)(1+A-2RS)}{(1+A-2R)(1-A-2RS)}\right]^{1/2A}$

  ② For $R = 1/2$ $\quad y_{max} = \left(\frac{R(1-2RS)}{1-2R}\right) e^{\left[\frac{2R(S-1)}{(1-2RS)(1-2R)}\right]}$

  ③ For $R > 1/4$ $\quad y_{max} = \left[(R - RS + R^2S^2)^{1/2}\right] \cdot e^{\left[\frac{1}{B}\tan^{-1}\left(\frac{2RS-1}{B}\right) - \frac{1}{B}\tan^{-1}\left(\frac{2R-1}{B}\right)\right]}$

Where:

- k = Hydraulic Conductivity of Geonet = 8.75 cm/sec (at F.O.S. = 4)
- L = Drainage Distance = 465'
- r = Vertical Liquid Impingement = 6.1 in/month = 5.98×10^{-6} cm/sec (REF. 2)
- S = $\tan\alpha \cong \sin\alpha = .02$ = slope of landfill base
- $R = r/(k \sin^2\alpha)$
- $Y_{max} = y_{max}/(L \tan\alpha)$ $\Rightarrow$ y_{max} = Maximum Saturated Depth
- $A = (1-4R)^{1/2}$
- $B = (4R-1)^{1/2}$

$R = \frac{Q}{(k \sin^2 \alpha)} = \frac{5.98 \times 10^{-6} \text{ cm/sec}}{(8.75 \text{ cm/sec})(.02)^2} = 1.7086 \times 10^{-3} < 1/2$, use equation ①

$A = (1-4R)^{1/2} = (1-4(1.7086 \times 10^{-3}))^{1/2} = .9966$

$$y_{max} = (R - RS + R^2S^2)^{1/2} \left[\frac{(1-A-2R)(1+A-2RS)}{(1+A-2R)(1-A-2RS)} \right]^{\frac{1}{2A}}$$

$$= (4.092 \times 10^{-2}) \left[\frac{(5.86 \times 10^{-6})(1.9965)}{(1.99316)(3.355 \times 10^{-3})} \right]^{\frac{1}{2(.9964)}}$$

$$= 1.693 \times 10^{-3}$$

$y_{max} = (1.693 \times 10^{-3})(465 \text{ ft})(.02) = 1.253 \times 10^{-2} \text{ Feet} = \underline{0.19 \text{ inches}}$

Conclusion: Despite the excessively high F.O.S. of 4, the leachate head levels remain within the 0.2" thick geonet layer. This demonstrates the inherently high liquid carrying capacity of the geonet.

A F.O.S. of 4 is more than sufficient to comfortably account for the anticipated clogging of the geonet due to solids and biomass. In addition, this scenario represents critical drainage conditions. Flow rates will greatly decrease as waste disposal is begun.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 1
PROJECT NO. 87221.466

CLIENT Piedmont LRC
PROJECT Transition Application + Permit Renewal

SUBJECT Leachate Head Analysis Under Interim Conditions.

Prepared By AJM Date 4/1/94
Reviewed By DMW Date 4/1/94
Approved By ____ Date ____

Purpose: To determine the permeability required under interim conditions to maintain leachate head levels within the geonet drainage layer thickness.

Given & Assumed:

- Under both interim & Final conditions, geonet permeability = 29.5 cm/sec
- From HELP data (attached within this Appendix, following this calculation), the peak daily leachate flow under interim conditions (60' waste) = .0288 in/day
  $\Rightarrow (.0288 \text{ in/day})\left(\frac{2.54 \text{ cm}}{1 \text{ inch}}\right)\left(\frac{1 \text{ day}}{24 \text{ hr}}\right)\left(\frac{1 \text{ hr}}{3600 \text{ sec}}\right) = 8.466 \times 10^{-7}$ cm/sec
- Geonet thickness = 0.2"

Calculation:

- Determine permeability needed to maintain head values at 0.2" under interim conditions. Use the previously defined equation by McEnroe.

① $\Rightarrow$ try k = 5 cm/sec

$$R = \frac{8.4667 \times 10^{-7}}{(5)(.02)^2} = 4.233 \times 10^{-4} < 1/4$$

$$A = (1-4R)^{1/2} = .99915$$

$$Y_{max} = (R - RS + R^2S^2)^{1/2}\left[\frac{(1-A-2R)(1+A-2RS)}{(1+A-2R)(1-A-2RS)}\right]^{1/2A}$$

$$= (2.037 \times 10^{-2})(2.072 \times 10^{-2})$$

$$= 4.221 \times 10^{-4}$$

$$y_{max} = 4.221 \times 10^{-4}(L)(\tan\alpha) = 3.93 \times 10^{-3} \text{ ft} = .047 \text{ inches}$$

② $\Rightarrow$ try k = 1.25 cm/sec

$$R = \frac{8.4667 \times 10^{-7}}{(1.25)(.02)^2} = 1.693 \times 10^{-3}$$

$$A = (1-4R)^{1/2} = .99661$$

$$Y_{max} = 1.0474)[4.119 \times 10^{-2}] = 1.6779 \times 10^{-3}$$

$$y_{max} = Y_{max}(L)(\tan\alpha) = .19 \text{ inches}$$

$\therefore$ At a permeability of 1.25 cm/sec, leachate head = .19 inches $\doteq$ 0.2 inches $\Rightarrow$ F.O.S. = $\frac{29.5}{1.25} = 23.$

REFERENCE 1

Maximum Saturated Depth over Landfill Liner

By Bruce M. McEnroe,[1] Member, ASCE

Abstract: Explicit formulas for the steady-state maximum saturated depth over a landfill liner are derived analytically from basic principles of unconfined seepage. The lateral drainage over the liner is described by an extended form of the Dupuit discharge formula. The liner is modeled as an impervious sloping plane. The factors that determine the maximum saturated depth over the liner are the rate of vertical inflow to the drainage layer, the hydraulic conductivity of the drainage layer, the slope of the liner, the maximum distance of flow, and the hydraulic condition at the downstream end of the drainage layer. Practical applications of these formulas include the analysis and design of drainage systems for landfill covers and bottom liners. An example illustrates the use of these formulas in the design of a cover drain for a hazardous-waste landfill. A formula that appears in several technical guidance documents of the U.S. Environmental Protection Agency is shown to overestimate the maximum saturated depth over a landfill liner.

Introduction

Flexible membrane liners of various geosynthetic materials are commonly used to prevent migration of leachate from landfills. Final covers for landfills and hazardous-waste sites also incorporate flexible membrane barriers to keep water from percolating into the waste. The barrier is normally placed on a slope, with a drainage layer of coarse granular material over it. Liquids percolate downward into a saturated zone directly over the barrier and flow laterally to a collection system, which is typically a perforated pipe. The drainage system should be designed to prevent saturation within the overlying waste or cover soil.

Several different methods for estimating the steady-state maximum saturated depth over a landfill liner have been proposed. An earlier paper by the writer (McEnroe 1989) presents an analytical solution for the saturated depth at any point along the liner. However, the determination of the maximum saturated depth from this general solution can be tedious. Furthermore, because this solution is based on the standard Dupuit assumptions for unconfined seepage, it underestimates saturated depths for liner slopes greater than about 10% (6°).

An explicit formula for the maximum saturated depth over a sloping liner appears in several technical guidance documents of the U.S. Environmental Protection Agency (USEPA) (Moore 1983; "Requirements" 1989). This formula is

$$y_{max} = L\left(\frac{r}{K}\right)^{1/2}\left[\frac{KS^2}{r} + 1 - \frac{KS}{r}\left(S^2 + \frac{r}{K}\right)^{1/2}\right] \quad \ldots\ldots\ldots\ldots (1)$$

in which y_{max} = maximum saturated depth over the liner; L = maximum distance of flow; r = rate of vertical inflow to the drainage layer per unit horizontal area; K = hydraulic conductivity of the drainage layer; and S = dimensionless slope of the liner. This formula was first presented by Moore (1983) without derivation or explanation of its origin or limitations.

Other investigators have published solutions for closely related problems with different boundary conditions. Yates et al. (1985) presented a general analytical solution for steady drainage over an impervious sloping barrier with recharge, based on the standard Dupuit assumptions. Werner (1957) presented an approximate solution for this problem. McEnroe and Schroeder (1988) used numerical methods to investigate steady drainage over a slightly pervious sloping barrier. Childs (1971), Towner (1975), and McBean et al. (1980) all presented solutions for steady drainage over a sloping barrier with drains at both the upper and lower ends. The latter drain configuration is not typical for landfill drainage systems.

This paper presents a set of explicit formulas for the steady-state maximum saturated depth over an impervious sloping barrier. These formulas are derived analytically from the equation of continuity and an extended form of the Dupuit discharge formula for unconfined seepage.

Problem Formulation

Fig. 1 shows the typical components of a landfill drainage system, and defines the coordinate system and some of the variables used in the analysis. Because the upstream boundary is either a drainage divide or an impervious sidewall, no flow can cross this boundary. Under steady-state conditions, continuity requires that the discharge at any location equals the vertical inflow upstream of this location, i.e.

$$Q = rx \quad \ldots\ldots\ldots\ldots (2)$$

in which x = horizontal distance from the upstream boundary; Q = discharge per unit width in the saturated zone; and r = vertical discharge into the saturated zone per unit horizontal area.

The standard Dupuit assumptions for unconfined ground water flow are that the streamlines are essentially horizontal and the equipotentials are essentially vertical, which means that the hydraulic gradient equals the slope of the phreatic surface and does not vary with depth. These approximations lead to the discharge formula

$$Q = -Ky\frac{dh}{dx} \quad \ldots\ldots\ldots\ldots (3)$$

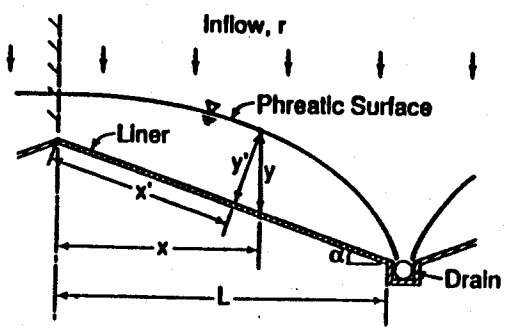


FIG. 1. Landfill Drainage System

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in w... y = saturated depth, measured vertically; and h = elevation of the phreatic surface. The elevation of the phreatic surface can be expressed in terms of the saturated depth and the slope of the liner

$$h = y + x \tan \alpha \quad \ldots\ldots (4)$$

The standard Dupuit assumptions are generally satisfactory for slopes up to about 10% (6°). For steeper slopes, the extended Dupuit assumptions are more realistic. These assumptions are that the streamlines are essentially parallel to the liner, and that the equipotentials are essentially perpendicular to the liner. The extended Dupuit assumptions lead to the discharge formula

$$Q = -Ky' \frac{dh}{dx'} \quad \ldots\ldots (5)$$

in which x' = distance from the upstream boundary measured parallel to the liner; and y' = saturated depth measured perpendicular to the liner. This extended Dupuit discharge formula can be written in terms of the horizontal and vertical coordinates as (Childs 1971)

$$Q = \frac{-Ky \dfrac{dh}{dx}}{1 - \dfrac{dh}{dx} \sin \alpha \cos \alpha} \quad \ldots\ldots (6)$$

Chapman (1980) showed that (6) also can be expressed in the form

$$Q = \frac{-Ky \dfrac{dh}{dx} \cos^2 \alpha}{1 - \dfrac{dy}{dx} \sin \alpha \cos \alpha} \quad \ldots\ldots (7)$$

Because the assumptions used to derive (7) require that $dy/dx \ll 1$, this equation is closely approximated by the simpler discharge equation

$$Q = -Ky \frac{dh}{dx} \cos^2 \alpha \quad \ldots\ldots (8)$$

Chapman showed that (8) is a major improvement on the standard Dupuit discharge formula, and a fair approximation of the extended Dupuit discharge formula for unconfined seepage on steep slopes.

Eqs. (2), (4), and (8) can be combined to form the governing equation in terms of saturated depth

$$Ky \left(\frac{dy}{dx} - \tan \alpha \right) \cos^2 \alpha + rx = 0 \quad \ldots\ldots (9)$$

The specification of the saturated depth at the downstream boundary, y_L, completes the problem statement. The saturated-depth profile is determined by the values of K, r, α, and y_L.

The analysis can be simplified by restating the governing equation and boundary condition in terms of the dimensionless variables $X = x/L$; $R = r/(K \sin^2 \alpha)$; $Y = y/(L \tan \alpha)$; and $Y_L = y_L/(L \tan \alpha)$. The dimensionless form of the governing equation is

$$Y \left(\frac{dY}{dX} - 1 \right) + RX = 0 \quad \ldots\ldots (10)$$

The boundary condition for (10) is $Y = Y_L$ at $X = 1$.

Saturated-Depth Profile

Eq. (10) is a nonlinear differential equation of homogeneous type. This equation becomes separable upon the change of variable $u = Y/X$, and can be integrated directly. In terms of this new variable, the governing equation is

$$-\frac{dX}{X} = \frac{u \, du}{R - u + u^2} \quad \ldots\ldots (11)$$

Integration of (11) yields

$$-\ln X = \frac{1}{2} \ln (R - u + u^2) + \frac{1}{2A} \ln \frac{2u - 1 - A}{2u - 1 + A} + C$$

for $R < 1/4$ $\ldots\ldots (12)$

$$-\ln X = \ln \left(\frac{1}{2} - u \right) + \frac{1}{1 - 2u} + C \quad \text{for } R = 1/4 \quad \ldots\ldots (13)$$

and

$$-\ln X = \frac{1}{2} \ln (R - u + u^2) + \frac{1}{B} \tan^{-1} \left(\frac{2u - 1}{B} \right) + C$$

for $R > 1/4$ $\ldots\ldots (14)$

in which

$$A = (1 - 4R)^{1/2} \quad \ldots\ldots (15)$$

$$B = (4R - 1)^{1/2} \quad \ldots\ldots (16)$$

and C = a constant of integration. Application of the boundary condition $u = Y_L$ at $x = L$ leads to the formulas

$$X = \left(\frac{R - Y_L + Y_L^2}{R - u + u^2} \right)^{1/2} \left[\frac{(1 - A - 2u)(1 + A - 2Y_L)}{(1 + A - 2u)(1 - A - 2Y_L)} \right]^{1/2A}$$

for $R < 1/4$ $\ldots\ldots (17)$

$$X = \frac{1 - 2Y_L}{1 - 2u} \exp \left[\frac{2(Y_L - u)}{(1 - 2Y_L)(1 - 2u)} \right] \quad \text{for } R = 1/4 \quad \ldots\ldots (18)$$

and

$$X = \left(\frac{R - Y_L + Y_L^2}{R - u + u^2} \right)^{1/2} \exp \left[\frac{1}{B} \tan^{-1} \left(\frac{2Y_L - 1}{B} \right) \right.$$

$$- \frac{1}{B} \tan^{-1}\left(\frac{2u-1}{B}\right)\Bigg] \quad \text{for } R > 1/4 \quad \ldots\ldots (19)$$

The values of R and Y_L completely determine the saturated-depth profile. To compute the coordinates of a point on the profile, one selects a feasible value of u and solves (17), (18), or (19) for X. The value of Y at this location is obtained from the relationship $Y = uX$. An earlier paper by the writer (McEnroe 1989) shows the possible shapes of the phreatic-surface profile.

Maximum Saturated Depth

General Case

The saturated depth is a maximum in which $dY/dX = 0$. From (10), the maximum saturated depth occurs where $Y = RX$ (i.e., where $u = R$). The substitution of R for u in (17)–(19) leads to the following explicit formulas for Y_{max}:

$$Y_{max} = (R - Y_L + Y_L^2)^{1/2}\left[\frac{(1 - A - 2R)(1 + A - 2Y_L)}{(1 + A - 2R)(1 - A - 2Y_L)}\right]^{1/2A}$$

for $R < 1/4$ ..... (20)

$$Y_{max} = \frac{R(1 - 2Y_L)}{1 - 2R}\exp\left[\frac{2(Y_L - R)}{(1 - 2Y_L)(1 - 2R)}\right] \quad \text{for } R = 1/4 \quad \ldots (21)$$

and

$$Y_{max} = (R - Y_L + Y_L^2)^{1/2}\exp\left[\frac{1}{B}\tan^{-1}\left(\frac{2Y_L - 1}{B}\right)\right.$$

$$\left. - \frac{1}{B}\tan^{-1}\left(\frac{2R - 1}{B}\right)\right] \quad \text{for } R > 1/4 \quad \ldots\ldots (22)$$

Free-Drainage Boundary Condition

Drainage is normally collected in a perforated pipe bedded in a drain trough. If the drainage system is functioning properly, the water level in the collection trough is below the top of the trough, and therefore has no effect on the saturated-depth profile over the liner. This desired operating condition is termed free drainage. If the collection system is undersized or clogged, water may pond over the collection trough and create a backwater effect over the liner.

A solution for the phreatic-surface profile in the vicinity of a bottom drain by Numerov (Harr 1962) provides guidance for estimating Y_L for the free-drainage boundary condition. For the boundary geometry in Fig. 2, Numerov's solution is

$$x = K\frac{h_0^2 - h^2}{2Q} + \frac{Q}{K}F(u) \quad \ldots\ldots (23)$$

in which

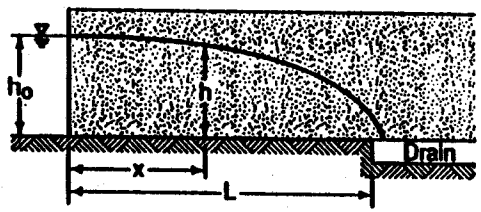

FIG. 2. Simplified Analysis of Free-Drainage Boundary Condition

$$Q = \frac{Kh_0^2}{L + \left(L^2 + \frac{h_0^2}{3}\right)^{1/2}} \quad \ldots\ldots (24)$$

$$u = \tanh\left(K\pi\frac{h_0 - h}{2Q}\right) \quad \ldots\ldots (25)$$

$$F(u) = \frac{2}{\pi}\int_0^1 \tan^{-1}\left(u\tan\frac{\pi t}{2}\right)\left[1 - \frac{1}{\pi}\tan^{-1}\left(u\tan\frac{\pi t}{2}\right)\right]dt \quad \ldots\ldots (26)$$

This solution was derived by conformal mapping without invoking the Dupuit approximations. The evaluation of this solution for specific values of L and y_0 indicates that the hydraulic gradient $dh/dx \approx -1$ at $x = L$ provided $L > y_0$. Although this result is based on a horizontal linear and no vertical inflow, the effects of the slope of the liner and the rate of vertical inflow on the conditions at the brink of the trough should be minor. The situation is roughly analogous to an abrupt overfall in open-channel flow with subcritical flow upstream of the overfall. In the latter case, the depth at the brink of the overfall depends only on the discharge and channel geometry at the overfall, and not on the slope of the channel or the rate of lateral inflow upstream of the overfall. Therefore, the approximation $dh/dx = -1$ at $x = L$ is recommended for free drainage, regardless of the slope of the liner or the rate of inflow. The corresponding value of y_L follows from the substitution of rL for Q and -1 for dh/dx in (8). The result is $y_L = rL/(K\cos^2\alpha)$, or $Y_L = RS$, in which $S = \tan\alpha$. The substitution of RS for Y_L in (20)–(22) leads to the following explicit formulas for the maximum saturated depth over a landfill liner that is draining freely with no backwater effect from the collection trough:

$$Y_{max} = (R - RS + R^2S^2)^{1/2}\left[\frac{(1 - A - 2R)(1 + A - 2RS)}{(1 + A - 2R)(1 - A - 2RS)}\right]^{1/2A}$$

for $R < 1/4$ ..... (27)

$$Y_{max} = \frac{R(1 - 2RS)}{1 - 2R}\exp\left[\frac{2R(S - 1)}{(1 - 2RS)(1 - 2R)}\right] \quad \text{for } R = 1/4 \quad \ldots (28)$$

and

$$Y_{max} = (R - RS + R^2S^2)^{1/2}\exp\left[\frac{1}{B}\tan^{-1}\left(\frac{2RS - 1}{B}\right)\right.$$

$$- \frac{1}{B} \quad \cdot 1 \left(\frac{2R - 1}{B}\right)\Bigg] \quad \text{for } R > 1/4 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Comparison with Formula of USEPA

Eq. (1), which appears in several publications of the U.S. Environmental Protection Agency, can be written in the dimensionless form

$$Y_{max} = R^{1/2}\left[1 - \frac{(1 + R)^{1/2} - 1}{R}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

provided the slope of the liner is small enough that the approximation $\sin\alpha = \tan\alpha$ is good. Fig. 3 compares (30) with (27)–(29). This comparison shows that (30) overestimates the maximum saturated depth significantly for $R < 1$, which includes much of the range of practical interest. For a drainage layer with a hydraulic conductivity of 10^{-2} cm/s on a slope of 3%, R would exceed unity only for a vertical inflow rate greater than 280 cm/yr.

Practical Applications

Eqs. (27)–(29) are useful for the analysis and design of landfill covers and leachate-collection systems. These equations yield an estimate of the maximum saturated depth over the liner for a constant rate of inflow. This maximum saturated depth is a spatial maximum for steady-state conditions. If the rate of inflow is an estimate of the long-term average inflow rate, then the resulting maximum saturated depth should be considered a long-term average. Over the short term, both the rate of inflow and the maximum saturated depth may differ considerably from their long-term averages.

The U.S. Environmental Protection Agency currently recommends that granular drainage layers for final covers on hazardous-waste landfills have a minimum thickness of 30 cm, a minimum hydraulic conductivity of 0.01 cm/s, and a minimum slope of 3% at the bottom of the layer ("Technical"

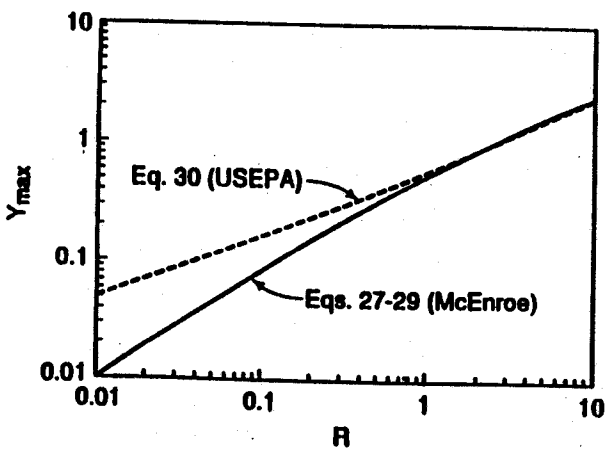

FIG. 3. Comparison of (27)–(29) and (30) for Maximum Saturated Depth over Landfill Liner

1989). The maximum saturated depth over the liner should not ed the thickness of the drainage layer, so that saturated conditions do not extend into the overlying soil cover. The following example illustrates the application of (27)–(29) to the design of a cover drain for a hazardous-waste landfill.

Problem

A final cover for a hazardous-waste landfill is to be constructed on a slope of 3% with a maximum drainage distance of 200 m. The granular drainage layer is to be 30 cm thick. The amount of rainfall percolating though the cover should average about 30 cm/yr. Estimate the minimum acceptable hydraulic conductivity for the drainage material.

Solution

The maximum saturated depth over the cover liner should not exceed 30 cm, which is the thickness of the drainage layer. The hydraulic conductivity needed to meet this constraint for a constant inflow rate of 30 cm/yr is determined by trial. For a hydraulic conductivity of 0.01 cm/s, the dimensionless inflow rate $R = 0.11$, (27) yields $Y_{max} = 0.088$, and the maximum saturated depth $y_{max} = 53$ cm. A hydraulic conductivity of 0.02 cm/s yields a maximum saturated depth of 28 cm, slightly less than the drainage-layer thickness, which would be acceptable if the rate of inflow were truly constant at 30 cm/yr. Because the rate of inflow to the drainage layer would vary with time, a larger hydraulic conductivity should be specified to prevent saturation of the cover soil during prolonged wet periods. A hydraulic conductivity of 0.1 cm/s should be satisfactory. This hydraulic conductivity would yield a maximum saturated depth of 6 cm for a constant inflow rate of 30 cm/yr.

The approximate formula of the USEPA greatly overestimates y_{max} for the conditions in this example. Eq. (1) yields $y_{max} = 100$ cm for $K = 0.01$ cm/s, $y_{max} = 70$ cm for $K = 0.02$ cm/s, and $y_{max} = 31$ cm for $K = 0.1$ cm/s. These values of y_{max} exceed the values from (27)–(29) by 90%, 150%, and 420%.

Conclusions

Eqs. (20)–(22) comprise a general solution for the steady-state maximum saturated depth over an impervious liner. The maximum saturated depth depends upon five factors: the rate of vertical inflow to the drainage layer, the hydraulic conductivity of the drainage layer, the maximum distance of flow, the slope of the liner, and the saturated depth at the downstream boundary. In landfill systems, the normal condition at the downstream boundary is a free overfall to a collection trough. Eqs. (27)–(29) yield the maximum saturated depth for this case. These formulas are recommended for the analysis and design of drainage systems for landfill covers and bottom liners. Eq. (1), which appears in several publications of the U.S. Environmental Protection Agency, greatly overestimates maximum saturated depths.

Acknowledgments

This work was supported by University of Kansas General Research Allocation No. 3139.

an, T. C. (1980). "Modelling groundwater flow over sloping beds." *Water Res.*, 16(6), 1114–1118.
C..., E. C. (1971). "Drainage of groundwater resting on a sloping bed." *Water Resour. Res.*, 7(5), 1256–1263.
Harr, M. E. (1962). *Groundwater and seepage*. McGraw-Hill Book Co., New York, N.Y., 210–226.
McEnroe, B. M. (1980). "Steady drainage of landfill covers and bottom liners." *J. Envir. Engrg.*, ASCE, 115(6), 1114–1122.
McEnroe, B. M., and Schroeder, P. R. (1988). "Leachate collection in landfills: steady case." *J. Environ. Engrg.*, ASCE, 114(5), 1052–1062.
McBean, E. A., Poland, R., Rovers, F. A., and Crutcher, A. J. (1982). "Leachate collection design for containment landfills." *J. Environ. Engrg.*, ASCE, 108(1), 204–209.
Moore, C. A. (1983). "Landfill and surface impoundments evaluation." *EPA/530/SW-869-C*, U.S. Envir. Protection Agency, Washington, D.C.
"Requirements for hazardous waste landfill design, construction, and closure." (1989). *EPA/625/4-89/022*, U.S. Envir. Protection Agency, Washington, D.C., 57.
"Technical guidance document: final covers on hazardous waste landfills and surface impoundments." (1989). *EPA/530-SW-89-047*, U.S. Envir. Protection Agency, Washington, D.C., 61–21.
Towner, G. D. (1975). "Drainage of groundwater resting on a sloping bed with uniform rainfall." *Water Resour. Res.*, 11(1), 144–147.
Werner, P. W. (1957). "Some problems in non-artesian groundwater flow." *Trans. Am. Geophysical Union*, 38(4), 511–518.
Yates, S. R., Warrick, A. W., and Lomen, D. O. (1985). "Hillside seepage: an analytical solution to the nonlinear Dupuit-Forchheimer problem." *Water Resour. Res.*, 21(3), 331–336.

Appendix II. Notation

The following symbols are used in this paper:

h = elevation of phreatic surface (L);
K = hydraulic conductivity of drainage layer (L/T);
L = drainage distance, measured horizontal (L);
Q = lateral discharge per unit width (L^3/T);
R = $r/(K \sin^2\alpha)$;
r = vertical inflow per unit horizontal area ($L^3/T/L^2$);
S = slope of liner (= $\tan\alpha$);
u = Y/X;
X = x/L;
x = distance coordinate, measured horizontally (L);
x' = distance coordinate, measured parallel to barrier (L);
Y = $y/(L \tan\alpha)$;
Y_L = $y_L/(L \tan\alpha)$;
Y_{max} = $y_{max}/(L \tan\alpha)$;
y = saturated depth, measured vertically (L);
y' = saturated depth, measured perpendicular to barrier (L);
y_L = saturated depth at brink of drain trough, measured vertically (L);
y_{max} = maximum saturated depth (L); and
α = liner angle, measured from horizontal.

Minimization of Multistage RBC Active Disc Area

By Roland Leduc[1] and Ian Buchanan[2]

Abstract: An analytical approach to the minimization of total active disc area (A_T), required for organic carbon removal [i.e., biochemical oxygen demand (BOD)] by a multistage rotating biological contactor (RBC), is presented. RBC operation is simulated using semiempirical models based on a saturation-type (Monod) removal rate, and on a first-order kinetics removal rate. The parameters used in the predictive equations are considered to be different from one RBC stage to the next. The minimizations of A_T, based in turn on each of the two semiempirical models, lead to a relationship between the optimal soluble substrate concentration of any stage and those of the immediately preceding and succeeding stages. When used in design, these relationships allow the minimum A_T, as well as the optimal number of stages required for carbon removal, to be obtained. For the saturation model, two parametric ratios (Φ_P and Φ_{K_S}) are introduced, which define the change in the model parameters with stage. Application of the relationships to the determination of the optimal number of stages and the minimum A_T is included.

Introduction

The rotating biological contactor (RBC) is a biological process for the treatment of wastewaters. It provides an environment in which biodegradable organic material contained in wastewater is metabolized by attached-growth microorganisms, thereby reducing the concentration of soluble organic material in the wastewater [often expressed as the soluble biochemical oxygen demand (SBOD)]. Rotating biological contactors are most often designed as a series of stages (see Fig. 1 for an illustration of a four-stage RBC). The series arrangement results in a high degree of organic removal for the following reasons (Antonie 1976):

- Microorganisms that grow in a particular stage are adapted to the wastewater characteristics of that stage.
- The residence time distribution within a multistage RBC approaches plug-flow, which increases the biochemical oxygen demand (BOD) removal rate.

An RBC stage consists of a tank through which wastewater flows and an array of plastic discs (the media) mounted on a horizontal shaft, which is slowly rotated. This rotation causes an area of the discs to be alternatively submerged in the wastewater flow and exposed to the air. Microorganisms grow to form an attached biofilm on the portion of the discs that is periodically submerged. This area is hereinafter termed the active disc area.

For the attached microorganisms to grow, they require a steady supply

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REFERENCE 2

LEACHATE DISPOSAL QUANTITIES FOR THE PIEDMONT LANDFILL & RECYC. CTR

Month/Year	Quantity Removed	Average Quantity Gal/day	Monthly Rainfall, Inches	Sq. Ft Open to Rainfall	Gal/acre/in Rainfall	Notes
Jul-92	204,795	7,877	→ 6.1	740,520	1,968	
Aug-92	239,665	9,218	5.9	740,520	2,389	
Sep-92	100,809	3,877	2.3	740,520	2,578	
Oct-92	162,151	6,237	3.8	740,520	2,510	
Nov-92	489,994	18,846	5.4	740,520	5,338	
Dec-92	313,578	12,061	4.2	740,520	4,392	
Jan-93	407,836	15,686	5.6	740,520	4,284	
Feb-93	361,356	13,898	3.5	958,320	4,761	See Note A
Mar-93	608,562	23,406	5.2	958,320	5,320	
Apr-93	337,597	12,985	5.2	958,320	2,951	
May-93	152,200	5,854	2.5	958,320	2,767	
Jun-93	80,402	3,092	3.1	958,320	1,179	
Jul-93	106,434	4,094		958,320	ERR	
Aug-93	85,615	3,293	3.3	958,320	1,197	
Total:	3,650,994		56			
Average:		10,277			ERR	

ALSO, THE AVERAGE MONTHLY RAINFALL DATA WAS CALCULATED USING 20 YEARS OF SYNTHETICALLY GENERATED RAINFALL FOR THE NEAREST MAJOR CITY, GREENSBORO, NC (UIA HELP MODEL). AS YOU CAN SEE, THE 6.1" USED IN OUR CALCULATIONS EXCEEDS THE PEAK MONTHLY AVERAGE BY NEARLY 1.8". SO 6.1" IS A RATHER CONSERVATIVE NUMBER.

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

PRECIPITATION	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
TOTALS	3.15	3.42	3.54	3.51	2.75	4.33
	3.63	4.29	2.99	3.99	2.40	3.09

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE ____ OF ____

PROJECT NO. ____

CLIENT ____ SUBJECT ____ Prepared By ____ Date ____

PROJECT ____ Reviewed By ____ Date ____

Approved By ____ Date ____

REFERENCE 3

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 1
PROJECT NO. 87221

CLIENT Piedmont LRC
PROJECT Permit Renewal
SUBJECT Geonet Conductivity Calculations
Prepared By AJM Date 11/4/9
Reviewed By JMC Date 11/4/9
Approved By ____ Date ____

Purpose: Calculate the hydraulic conductivity of the geonet drainage layer component of the landfill base liner under "open cell" conditions. The manufacturer's transmissivity chart will be referenced for this purpose.

Given:
- Material = PN 8000
- Underlying Materials: HDPE and 2' Compacted Soil
- Overlying Materials: Geotextile and 2' Protective Soil
- Unit Weight of Protective Soils = 135 #/ft³
- Slope = Gradient = 2% min, 5.7% max
- Thickness of geonet = 0.2"

Calculation: ⇒ From Designing with Geosynthetics, 2nd Ed., by Robert Koerner, pps 90-91

$$\theta = k_p t \; ; \; k_p = \theta/t$$

Where θ = transmissivity
k_p = permeability coefficient in the plane of the geonet
t = thickness

From the attached chart, @
Gradient = 0.10 (conservative to match chart, avoids having to extrapolate)
Overburden = (135 lb/ft³)(2 ft) = 270 psf ⇒ Say 2000 psf to match the chart (conservative, avoids having to extrapolate)
⇒ θ = transmissivity = 1.8×10^{-3} m²/sec

⇒ $\theta = k_p t \; ; \; k_p = \theta/t$

$$k_p = \frac{(1.8 \times 10^{-3} \text{ m}^2/\text{sec})}{(0.20 \text{ in})}\left[\left(\frac{\text{inch}}{2.5 \text{ cm}}\right)\left(\frac{100 \text{ cm}}{1 \text{ m}}\right)^2\right] = 35.43 \text{ cm/sec}$$

Conclusion:

∴ hydraulic conductivity = k_p = 35 cm/sec

⇒ Under the light overburden of "open cell" conditions, the geonet hydraulic conductivity

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 2
PROJECT NO. 82221

CLIENT Piedmont Lec
PROJECT Transition Application & Permit Renewal

SUBJECT Geonet Permeability Calculations

Prepared By GJM Date 3/18/
Reviewed By DMW Date 3/30/9
Approved By ____ Date ____

Purpose: Calculate the hydraulic conductivity of the geonet drainage layer component of the landfill base liner under interim and final closure conditions. The manufacturer's transmissivity chart will be referenced for this purpose.

Given & Assumed:

- Material: PN 3000 texnet (i.e. geotextile above the geonet)
- Assumed Interim Fill Conditions = 63' of waste and soils overly geonet.
  - 1' of intermediate cover @ γ = 120 pcf
  - 60' of waste @ γ = 75 pcf
  - 2' of protective soils @ γ = 135 pcf

  ⇒ Overburden = $(1'\times 120\ lb/ft^3) + (60')(75\ lb/ft^3) + (2'\times 135\ lb/ft^3)$ = 4890 psf

- Assumed Fill Closure Fill Conditions = 125.5' of waste and soils overly geonet
  - 3.5' of final cover @ γ = 120 pcf
  - 120' of waste @ γ = 75 pcf (This represents a worst case scenario)
  - 2' of protective soils @ γ = 135 pcf

  ⇒ Overburden = $(3.5'\times 120\ lb/ft^3) + (120')(75\ lb/ft^3) + (2')(135\ lb/ft^3)$ = 9690 psf

- Slope = Gradient = 2% min → 5.7% max
- Thickness of geonet = 0.2"
- From Designing with Geosynthetics; 2nd Edition, by Robert Koerner, pps 90-9

$$\theta = k_p t \quad \Rightarrow \quad k_p = \theta/t$$

Where: θ = hydraulic transmissivity
k_p = permeability coefficient in the plane of the geonet
t = thickness of geonet

Calculations:

① INTERIM FILL CONDITIONS

- From Attached Chart; @ - Gradient = 0.1 (Conservative)
  - Overburden = 5000psf (conservative)

$\Theta = 1.5 \times 10^{-3} \ m^2/sec$

$\Theta = k_p t \qquad k_p = \Theta / t$

$k_p = (1.5 \times 10^{-3} \ m^2/sec)\left(\frac{100cm}{1m}\right)^2 \left(\frac{1}{0.2in}\right)\left(\frac{1in}{2.54cm}\right) = 29.5 \ cm/sec$

$k_p = 29.5 \ cm/sec$ (at interim conditions analyzed)

② FINAL CLOSURE CONFIGURATION

- From Attached Chart; @ - Gradient = 0.1 (conservative)
  - Overburden = 10,000 psf (conservative)

$\Theta = 1.5 \times 10^{-3} \ m^2/sec$

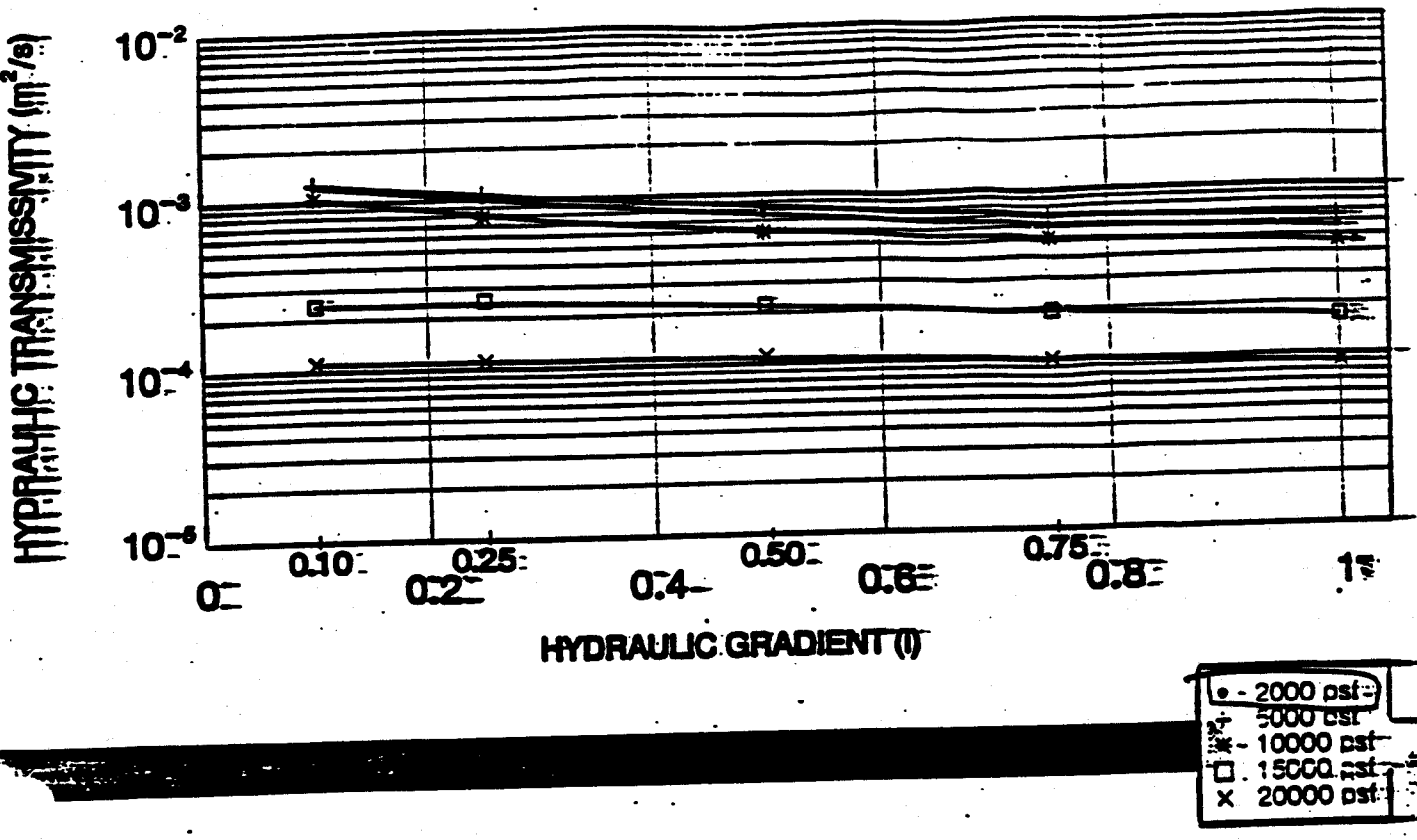
$\Theta = k_p t \qquad k_p = \Theta / t$

$k_p = (1.5 \times 10^{-3} \ m^2/sec)\left(\frac{100cm}{m}\right)^2 \left(\frac{1}{0.2in}\right)\left(\frac{1in}{2.54cm}\right) = 29.5 \ cm/sec$

$k_p = 29.5 \ cm/sec$ (at worst case closure conditions)

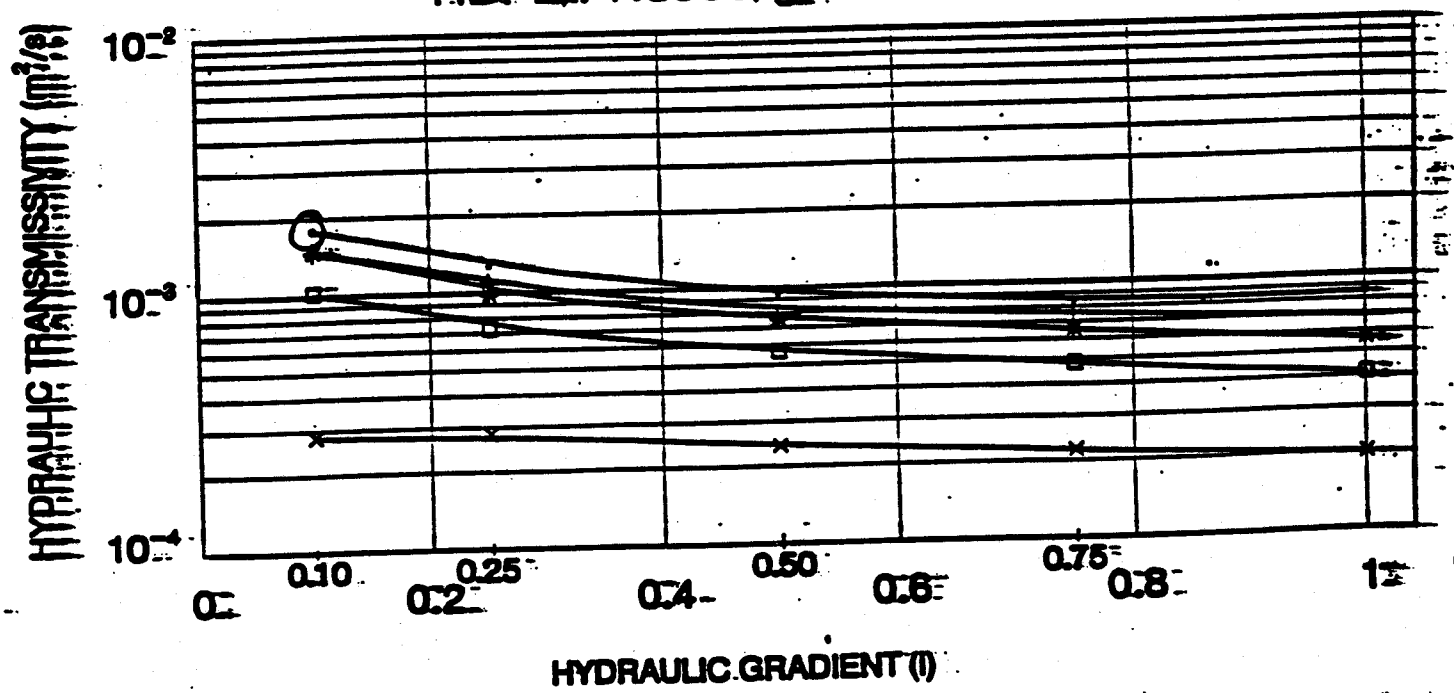
POLY-NET PN2000

HDPE/PN2000/geotextile/soil



POLY-NET PN3000

HDPE/PN3000/geotextile/soil



RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 3
PROJECT NO. 87221

CLIENT Piedmont LEC
PROJECT Transition Application and Permit Renewal

SUBJECT Additional Leachate Generation and Permeability Analysis

Prepared By AJM Date 3/18/9
Reviewed By [illegible] Date 3/30/9
Approved By ____ Date ____

Purpose: To determine an estimate for leachate collected within the Piedmont LEC geonet drainage layer under "interim fill" conditions. This estimate, along with the estimate for geonet transmissivity under Full landfill loading, will be compared to the respective values under "open cell" conditions. Based on the resulting information, it can be determined whether the most critical and conservative conditions were analyzed in the 11/4/93 leachate head calcs (Appendix C- Engineering Report).

Givens & Assumed:

- Use the Hydrologic Evaluation of Landfill Performance (HELP) model, Version 2, to estimate leachate generation quantities.
- Analyze under interim conditions (i.e. 60 feet of waste).
- Assume a time frame of 20 years prior to Final cover application. This is a conservatively long estimate.
- Permeability of geonet = k = 29.5 cm/sec. It should be noted that the higher the k value of the drainage layer, the higher the leachate drainage estimate. The selected value was calculated, and is attached.
- The initial moisture contents of the layers of the landfill cross-section will be initiated by the model.
- Slope of drainage in the geonet drainage layer will be selected as 5.7%. It should be noted that higher drainage slopes will result in higher leachate drainage quantities. (Phase 3-Cell 1-Subcell 2)
- The length of drainage is selected to be 250'. It should be noted that the shorter the drainage length, the resulting quantities of drainage will increase. (Phase 3-Cell 1-Subcell 2)
- Synthetic rainfall data from Greensboro, N. Carolina.
- Refer to the landfill cross-section on the following page.

① Intermediate Cover
- Soil Texture # 10
- Porosity = .398
- Field Capacity = .244
- Wilting Point = .136

② Waste
- Soil Texture # 18
- Porosity = .650
- Field Capacity = .294
- Wilting Point = .128

③ Protective Soils
- Soil Texture # 10
- Porosity = .398
- Field Capacity = .244
- Wilting Point = .136

④ Geonet
- Soil Texture # 19
- Porosity = 0.80 (manufacturer's Data)
- Field Capacity = .030
- Wilting Point = .020

⑤ Base Liner
- Soil Texture # 16
- Porosity = .430
- Field Capacity = .366
- Wilting Point = .280

① 12" Intermediate Cover

② 720' Waste

③ 24" Protective Soils

.2" Geonet ④

⑤ 24" ∫ Compacted Clay Overlain by Geonet

- Based on the results of the HELP model, the peak daily drainage through the geonet = 0.028 inches/day.

⇒ Compare this value to the value of impingement used in evaluating the leachate heads under "open cell" conditions (performed 11/4/94)

"Open Cell" = 6.1 in/month (This was the peak rainfall value measured at the facility over a 13 month period. The 25-year, 24-hour storm ≅ 6 in/month)

Interim Fill = .028 in/day = .893 in/month (at 31 day month)

⇒ Decrease in leachate drainage by going from "open" to interim conditions is as follows: $\left[\frac{6.1 - .893}{6.1}\right] 100\% = \underline{85.4\%}$

⇒ Compare the geonet permeabilities given the overburden conditions...

"Open Cell" = 35 cm/sec

Interim Fill (or Max Final Fill) = 29.5 cm/sec

⇒ Decrease in permeability by going from "open" to interim conditions is as follows: $\left[\frac{35 - 29.5}{35}\right] 100\% = \underline{15.7\%}$

Conclusion: Although the interim and final fill conditions decrease the geonet permeability by 15.7%, the Filling to interim conditions decreases the peak anticipated leachate drainage in the geonet by 85.4%. Under final conditions, with final cover application, the leachate quantity will decrease further. The huge benefits of decrease leachate drainage by filling with waste greatly exceed the loss in permeability resulting from filling to maximum elevations. Therefore, analyzing head buildups in the drainage layer under "open cell" conditions, as was done on 11/4/93 (Appendix C) is the most conservative analysis.

TABLE 4. DEFAULT UNVEGETATED, UNCOMPACTED SOIL CHARACTERISTICS

SOIL TEXTURE			POROSITY	FIELD CAPACITY	WILTING POINT	SAT. HYD. CONDUCTIVITY
HELP	USDA	USCS	(VOL/VOL)	(VOL/VOL)	(VOL/VOL)	(CM/SEC)
1	CoS	GS	0.417	0.045	0.018	1.0E-02
2	S	SW	0.437	0.062	0.024	5.8E-03
3	FS	SM	0.457	0.083	0.033	3.1E-03
4	LS	SM	0.437	0.105	0.047	1.7E-03
5	LFS	SM	0.457	0.131	0.058	1.0E-03
6	SL	SM	0.453	0.190	0.085	7.2E-04
7	FSL	SM	0.473	0.222	0.104	5.2E-04
8	L	ML	0.463	0.232	0.116	3.7E-04
9	SiL	ML	0.501	0.284	0.135	1.9E-04
10	SCL	SC	0.398	0.244	0.136	1.2E-04
11	CL	CL	0.464	0.310	0.187	6.4E-05
12	SiCL	CL	0.471	0.342	0.210	4.2E-05
13	SC	CH	0.430	0.321	0.221	3.3E-05
14	SiC	CH	0.479	0.371	0.251	2.5E-05
15	C	CH	0.475	0.378	0.265	1.7E-05
16	Liner Soil		0.430	0.366	0.280	1.0E-07
17	Liner Soil		0.400	0.356	0.290	1.0E-08
18	Mun. Waste		0.520	0.294	0.140	2.0E-04
19	USER SPECIFIED SOIL CHARACTERISTICS					
20	USER SPECIFIED SOIL CHARACTERISTICS					

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PIEDMONT LRC
LEACHATE GENERATION ESTIMATES
INTERIM FILL CONDITIONS

Years 0-20

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POOR GRASS

LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.3980 VOL/VOL
FIELD CAPACITY	=	0.2449 VOL/VOL
WILTING POINT	=	0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3016 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.0001199999970 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	720.00 INCHES
POROSITY	=	0.6500 VOL/VOL
FIELD CAPACITY	=	0.2950 VOL/VOL
WILTING POINT	=	0.1280 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3035 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.0001999999949 CM/SEC

LAYER 3

VERTICAL PERCOLATION LAYER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.3980 VOL/VOL
FIELD CAPACITY	=	0.2449 VOL/VOL

WILTING POINT	=	0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2337 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.0001199999970 CM/SEC

LAYER 4

LATERAL DRAINAGE LAYER

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8000 VOL/VOL
FIELD CAPACITY	=	0.0300 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0310 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	29.5000000000000 CM/SEC
SLOPE	=	5.70 PERCENT
DRAINAGE LENGTH	=	250.0 FEET

LAYER 5

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.4300 VOL/VOL
FIELD CAPACITY	=	0.3667 VOL/VOL
WILTING POINT	=	0.2800 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4300 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.0000001000000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00010000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	89.91
TOTAL AREA OF COVER	=	1. SQ FT
EVAPORATIVE ZONE DEPTH	=	9.00 INCHES
UPPER LIMIT VEG. STORAGE	=	3.5820 INCHES
INITIAL VEG. STORAGE	=	2.6759 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR GREENSBORO NORTH CAROLINA

MAXIMUM LEAF AREA INDEX	=	1.00
START OF GROWING SEASON (JULIAN DATE)	=	100

END OF GROWING SEASON (JULIAN DATE) = 302

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
37.50	39.90	48.00	58.30	66.50	73.50
77.20	76.30	69.90	58.40	48.50	40.20

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.15	3.42	3.54	3.51	2.75	4.33
	3.63	4.29	2.99	3.99	2.40	3.09
STD. DEVIATIONS	1.46	2.28	1.77	1.53	1.73	1.54
	1.56	2.03	2.06	2.88	1.66	1.86
RUNOFF						
TOTALS	0.268	0.394	0.400	0.441	0.214	0.541
	0.301	0.519	0.484	0.868	0.341	0.517
STD. DEVIATIONS	0.250	0.854	0.595	0.456	0.318	0.758
	0.311	0.720	0.637	1.218	0.402	0.747
EVAPOTRANSPIRATION						
TOTALS	1.508	1.767	2.378	2.449	2.484	3.481
	3.362	3.243	2.227	1.781	1.406	1.292
STD. DEVIATIONS	0.251	0.379	0.597	0.665	1.196	0.880
	1.163	1.112	1.100	0.661	0.593	0.253
LATERAL DRAINAGE FROM LAYER 4						
TOTALS	0.4387	0.4108	0.4636	0.4569	0.4747	0.4585
	0.4702	0.4643	0.4446	0.4561	0.4444	0.4639
STD. DEVIATIONS	0.2819	0.2618	0.2897	0.2831	0.2905	0.2770
	0.2816	0.2755	0.2622	0.2663	0.2593	0.2693
PERCOLATION FROM LAYER 5						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	41.09 (6.492)	3.	100.00
RUNOFF	5.290 (2.447)	0.	12.87
EVAPOTRANSPIRATION	27.378 (2.545)	2.	66.63
LATERAL DRAINAGE FROM LAYER 4	5.4468 (3.2876)	0.	13.26
PERCOLATION FROM LAYER 5	0.0000 (0.0000)	0.	0.00
CHANGE IN WATER STORAGE	2.973 (3.329)	0.	7.24

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	4.44	0.4
RUNOFF	3.095	0.3
LATERAL DRAINAGE FROM LAYER 4	0.0288	0.0
PERCOLATION FROM LAYER 5	0.0000	0.0
HEAD ON LAYER 5	0.0	
SNOW WATER	2.71	0.2

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3680

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1317

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	3.20	0.2666
2	277.25	0.3851
3	6.67	0.2780
4	0.01	0.0439
5	10.32	0.4300
SNOW WATER	0.00	

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APPENDIX D

Leachate Containment Calculation

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 6
PROJECT NO. 87221

CLIENT WMCT
PROJECT PIEDMONT LRC

SUBJECT CELL STORAGE QUANTITY/PUMP CAPACITY

Prepared By RMC Date 10/15
Reviewed By RJM Date 10/15
Approved By ____ Date ____

PURPOSE: DETERMINE IF NEWLY CONSTRUCTED CELLS (WORSE CASE SCENARIO) CAN CONTAIN THE 24 HR - 25 YR STORM; IF THE CELL DOES OVERTOP, DETERMINE WHAT SIZE PUMP IS NEEDED TO KEEP THE CELL FROM OVERTOPPING.

ASSUMPTIONS:

- 25 YR-24 HR STORM = 6.0" (ACTUAL VALUE IS IN THE NEIGHBORHOOD OF 5.5"; HOWEVER 6.0" WAS CHOSEN AS CONSER
- RUNOFF FACTOR = 0
- EVAPORATION = 0 (THIS IS A CONSERVATIVE ASSUMPTION)
- POROSITY OF PROTECTIVE & DRAINAGE LAYER = 0.40 (THICKNESS = 2.0')

- SOFTWARE USED: "QTR-55" AND "POND2" (PART OF HAESTAD METHODS PACKAGE, 1989); THESE PROGRAMS WERE USED IN DETERMINING THE TIME OF CONCENTRATION WITHIN THE CELLS AND, IF NECESSARY, THE PUMPING RATE NEEDED TO KEEP THE CELL FROM OVERTOPPING, RESPECTIVELY.

"AREA" = SIZE CELL UP TO TOP OF BERMS
"ELEV" = ELEVATION OF TOP OF HDPE
"POND AREA" = AREA WHERE RAINFALL PONDS
"AIRSPACE VOLUME" = VOLUME AVAILABLE ABOVE THE PROTECTIVE/DRAINAGE LAYER
"VOID VOLUME" = VOLUME AVAILABLE WITHIN PROTECTIVE/DRAINAGE LAYER
"TOTAL VOLUME" = AIRSPACE + VOID VOLUME
"STORAGE NEEDED" = STORAGE NECESSARY TO CONTAIN 24HR-25YR STOR

AREAS DETERMINED USING A PLANIMETER AND INTERPOLATION.

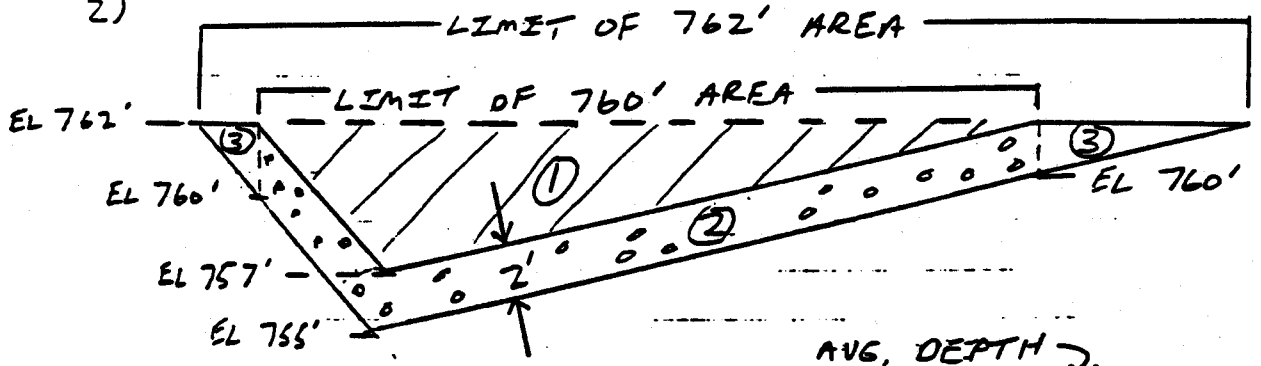
ALL BERMS AT LEAST 5' HIGH (TOP OF HDPE TO TOP OF BERM)

CALCULATIONS:

EXAMPLE OF HOW VOLUMES WERE ESTIMATED USING THE INFO FROM PHASE 3 - CELL 2 - SUBCELL 1 AT ELEVATION 762'.

1) THE AREA FOR ALL OF THE GIVEN ELEVATIONS WAS EITHER PLANIMETERED OR INTERPOLATED.

2) CROSS-SECTION OF POND-AREA



N.T.S.

AVG. DEPTH

① AIRSPACE VOLUME = (760' POND AREA) $\left(\frac{762' - 757'}{2}\right)$

$V_{①}$ = (1.27 AC)(2.5') = 3.18 AC-FT

② VOID VOLUME = (760' POND AREA)(DRAINAGE LAYER THICKNESS)(POROSITY)

$V_{②}$ = (1.27 AC)(2')(0.4) = 1.02 AC-FT

AVG D

③ VOID VOLUME = [(762' POND AREA) − (760' POND AREA)] $\left(\frac{762' - 760'}{2}\right)$ (POR

$V_{③}$ = (2.25 AC − 1.27 AC)(1')(0.4) = 0.39 AC-FT

VOID VOLUME = $V_{②} + V_{③}$ = 1.02 AC-FT + 0.39 AC-FT = 1.41 AC-F

TOTAL VOLUME = AIRSPACE VOLUME + VOID VOLUME

$V_{TOT} = V_{①} + (V_{②} + V_{③})$

V_{TOT} = 4.59 AC-FT

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 3 OF 6
PROJECT NO. 87221.300

CLIENT WMCI
PROJECT PIEDMONT LRC

SUBJECT CELL STORAGE QUANTITY / PUMP CAPACITY

Prepared By JDMC Date 10/15/93
Reviewed By RJM Date 10/18/93
Approved By ____ Date ____

PHASE 3 CELL 2 SUBCELL 1 (LOW POINT)

AREA = 9.94 AC STORAGE NEEDED = 4.97 AC-FT

ELEV. (FT)	POND AREA (Acres)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
755	0.01	0	0	0
755.5	0.05	0	0.005	0.005
756	0.10	0	0.02	0.02
757	0.30	0	0.12	0.12
758	0.58	0.05	0.27	0.32
759	0.93	0.30	0.49	0.79
760	1.27	0.87	0.74	1.61
761	1.76	1.86	1.08	2.94
762	2.25	3.18	1.41	4.59

IF THE RAINFALL WAS PUMPED OUT AT 0.12 CFS (54GPM) BEGINNING WHEN THERE IS A DEPTH OF 755.5', THE CELL WILL CREST AT 762' AND THEN RECEDE.

PHASE 3 CELL 2 SUBCELL 2 (LOW POINT)

AREA = 9.27 AC STORAGE NEEDED = 4.64 AC-FT

ELEV. (FT)	POND AREA (Acres)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
771	0.01	0	0	0
771.5	0.03	0	0.006	0.006
772	0.06	0	0.012	0.012
773	0.16	0	0.06	0.06
774	0.43	0.03	0.20	0.23
775	0.75	0.16	0.36	0.52
776	1.22	0.65	0.66	1.31
777	1.77	1.50	1.01	2.51
778	2.31	3.05	1.41	4.46

CELL WILL NOT OVERTOP IF THE BERMS ARE A MINIMUM OF 778.0'; CELL WILL CREST AT 778.0'.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 2

PROJECT NO. 87221.380

CLIENT WMCI

SUBJECT CELL STORAGE QUANTITY / PUMP CAPACITY

PROJECT PIEDMONT LRC

Prepared By JME Date 10/15/

Reviewed By CJM Date 10/16/9

Approved By ____ Date ____

PHASE 3 CELL 1 SUBCELL 1 — AREA = 6.01 AC — STORAGE NEEDED = 3.01 AC-FT

	ELEV. (FT)	POND AREA (ACRES)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
(LOW POINT)	765.8	0.01	0	0	0
	766.3	0.06	0	0.01	0.01
	766.8	0.13	0	0.03	0.03
	767.8	0.35	0	0.14	0.14
	768.8	0.74	0.07	0.35	0.42
	769.8	1.21	0.35	0.62	0.97
	770.8	1.87	1.11	1.04	2.15
	771.8	2.57	2.42	1.52	3.94

CELL WILL NOT OVERTOP AS LONG AS THE BERMS ARE A MINIMUM OF 771.8'; CELL WILL CREST AT 771.3'.

PHASE 3 CELL 1 SUBCELL 2 — AREA = 6.73 AC — STORAGE NEEDED = 3.37 AC-FT

	ELEV. (FT)	POND AREA (ACRES)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
(LOW POINT)	765	0.01	0	0	0
	766	0.22	0	0.04	0.04
	767	0.42	0	0.17	0.17
	768	0.68	0.11	0.36	0.47
	769	1.00	0.42	0.57	0.99
	770	1.39	1.02	0.83	1.85
	771	1.70	2.00	1.08	3.08
	772	2.10	3.48	1.40	4.88

CELL WILL NOT OVERTOP AS LONG AS THE BERMS ARE A MINIMUM OF 772'; CELL WILL CREST AT 771.1'.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 5 OF 6
PROJECT NO. 87721.300

CLIENT WMCI
PROJECT PIEDMONT LRC

SUBJECT CELL STORAGE QUANTITY / PUMP CAPACITY

Prepared By JMC Date 10/15/
Reviewed By [illegible] Date 10/18/93
Approved By ____ Date ____

PHASE 3 CELL 1 SUBCELL 3 (LOW POINT)

AREA = 4.48 AC STORAGE NEEDED = 2.24 AC-FT

ELEV. (FT)	POND AREA (ACRES)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
768.5	0.01	0	0	0
769	0.28	0	0.03	0.03
769.5	0.47	0	0.09	0.09
770.5	0.82	0	0.33	0.33
771.5	1.29	0.24	0.70	0.94
772.5	1.80	0.82	1.05	1.87
773.5	2.23	1.94	1.41	3.35

CELL WILL NOT OVERTOP AS LONG AS THE BERMS ARE A MINIMUM OF 773.5'; CELL WILL CREST AT 772.8'. SOUTHERN PORTION OF THIS CELL IS SEPARATED BY A RAINFLAP (TEMP.). THIS AREA WAS NOT INCLUDED IN THE EVALUATION BECAUSE THE COMBINATION OF RAINFLAP / STORMWATER PUMPING WILL KEEP WATER OUT OF THE NORTHERN PORTIONS. WHEN FILLI ADVANCES TO THIS AREA, AND THE RAINFLAP IS REMOVED, THE PERIMETER BERMS ARE HIGH ENOUGH TO CONTAIN LEACHATE. (SEE B

PHASE 3 CELL 1 SUBCELL 3 (SOUTHERN PORTION)

AREA = 2.82 Ac STORAGE NEEDED = 1.41 AC-FT

ELEV. (FT)	POND AREA (ACRES)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
774	0.01	0	0	0
776	0.12	0	0.05	0.05
778	0.45	0.12	0.23	0.35
780	0.66	0.90	0.44	1.34
782	0.89	1.98	0.62	2.60

CELL WILL NOT OVERTOP AS LONG AS THE BERMS ARE A MINIMUM OF 780'; CELL WILL CREST AT 779.9'.

GRAPHICAL PEAK DISCHARGE METHOD (TR-55)

CURVE NUMBER (CN) = 80
RAINFALL DISTRIBUTION TYPE = II
MANNING'S NUMBER FOR SHEET FLOW (n) = 0.20
WATERSHED =

DESIGN STORM: 25-yr, 24-hr (P) = 6.0 IN.
RUNOFF (Q) = 0 IN.
2-yr, 24-hr (P_2) = 3.5 IN.
PEAK DISCHARGE AT OUTFALL NODE =

RUST ENVIRONMENT & INFRASTRUCTURE
CALCULATION SHEET
CLIENT WMCT
PROJECT PIEDMONT CRC
SUBJECT CELL STORAGE QUANTITY / FINAL QUALITY
PROJECT NO. 87221.300
Prepared By [illegible] Date 10/15/13
Reviewed By [illegible] Date 10/15/13
Approved By ____ Date ____

	SUB AREA	AREA (ACRES)	CONCENTRATION TIME TO SUBAREA OUTFALL (t_c)							TRAVELING TIME DOWNSTREAM FROM SUBAREA OUTFALL				T_c (HRS)	T_t (HRS)
			SHEET FLOW			SURFACE/DIVERSION BERM FLOW				DRAINAGE DITCH FLOW					
			LENGTH L_1 (FT)	SLOPE S_1 (FT/FT)	t_1 (HRS)	LENGTH L_2 (FT)	SLOPE S_2 (FT/FT)	VELOCITY V_2 (FPS)	t_2 (HRS)	LENGTH L_3 (FT)	SLOPE S_3 (FT/FT)	VELOCITY V_3 (FPS)	t_3 (HRS)		
1	PHASE 3 C-2, SC-1	9.94	300	0.026	0.426	250	0.026	1.15	0.060					0.486	
2	PHASE 3 C-2, SC-2	9.27	300	0.044	0.345	140	0.044	1.65	0.024					0.411	
						190	0.038	1.25	0.042						
4	PHASE 3 C1, SC-1	6.01	300	0.021	0.464	170	0.021	1.10	0.043					0.507	
3	PHASE 3 C1, SC-2	6.73	300	0.055	0.316	80	0.029	1.20	0.019					0.335	
5	PHASE 3 C1, SC-3	4.48	215	0.052	0.247	120	0.022	1.10	0.030					0.443	
			85	0.022	0.166										
6	PHASE 3 C1, SC-3 SOUTH	2.82	300	0.051	0.326	120	0.051	1.80	0.019					0.345	

$L_1 \leq 300$ FT
V_2 FOR SURFACE FLOW SEE
$T_c = t_1 + t_2$ (TO SUB AREA OUTFALL)
$T_c = t_3$ (FROM SUBAREA OUTFALL TO WATERSHED OUTFALL)

$$t_1 = \frac{0.007 (n L_1)^{0.8}}{P_2^{0.5} (s_i)^{0.4}}$$

$t_2 = L_2 / 3600 V_2$
$t_3 = L_3 / 3600 V_3$

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:40:57
Watershed file: --> A:PIED321 .WSD
Hydrograph file: --> A:PIED321 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL1

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
321	9.94	80.0	0.50	0.00	8.40	6.00	.06 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 9.94 acres or 0.01553 sq.mi
Peak discharge = 49 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
321	0.49	0.00	0.50	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:40:57
Watershed file: --> A:PIED321 .WSD
Hydrograph file: --> A:PIED321 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL1

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
321	49	12.4
Composite Watershed	49	12.4

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:40:57
Watershed file: --> A:PIED321 .WSD
Hydrograph file: --> A:PIED321 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL1

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
321	2	2	3	5	9	16	29	44	49
Total (cfs)	2	2	3	5	9	16	29	44	49

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
321	47	37	28	21	13	9	7	6	5
Total (cfs)	47	37	28	21	13	9	7	6	5

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
321	4	4	3	3	3	2	2	2	2
Total (cfs)	4	4	3	3	3	2	2	2	2

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
321	2	1	1	1	0
Total (cfs)	2	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:40:57
Watershed file: --> A:PIED321 .WSD
Hydrograph file: --> A:PIED321 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL1

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	2	14.8	3
11.1	2	14.9	3
11.2	2	15.0	3
11.3	2	15.1	3
11.4	2	15.2	3
11.5	3	15.3	3
11.6	3	15.4	3
11.7	4	15.5	3
11.8	4	15.6	3
11.9	5	15.7	3
12.0	9	15.8	2
12.1	16	15.9	2
12.2	29	16.0	2
12.3	44	16.1	2
12.4	49	16.2	2
12.5	47	16.3	2
12.6	37	16.4	2
12.7	28	16.5	2
12.8	21	16.6	2
12.9	17	16.7	2
13.0	13	16.8	2
13.1	11	16.9	2
13.2	9	17.0	2
13.3	8	17.1	2
13.4	7	17.2	2
13.5	7	17.3	2
13.6	6	17.4	2
13.7	5	17.5	2
13.8	5	17.6	2
13.9	4	17.7	2
14.0	4	17.8	2
14.1	4	17.9	2
14.2	4	18.0	2
14.3	4	18.1	2
14.4	4	18.2	2
14.5	3	18.3	2
14.6	3	18.4	2

14.7	3	18.5	2

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:40:57
Watershed file: --> A:PIED321 .WSD
Hydrograph file: --> A:PIED321 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL1

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* PIEDMONT LANDFILL AND RECYCLING CENTER *
* KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA *
* PHASE 3 - CELL 2 - SUBCELL 1 *
* 24 HR-25 YR STORM STORAGE *
*
****

Inflow Hydrograph: A:PIED1 .HYD
Rating Table file: A:POND1 .PND

----INITIAL CONDITIONS----
Elevation = 755.00 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
755.00	0.0	0.000
755.50	0.1	0.005
756.00	0.1	0.020
757.00	0.1	0.120
758.00	0.1	0.320
759.00	0.1	0.790
760.00	0.1	1.610
761.00	0.1	2.940
762.00	0.1	4.590

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
1.2	1.3
4.8	5.0
29.0	29.2
77.4	77.6
191.2	191.3
389.6	389.7
711.5	711.6
1110.8	1110.9

Time increment (t) = 0.100 hrs.

Pond File: A:POND1 .PND
Inflow Hydrograph: A:PIED1 .HYD
Outflow Hydrograph: A:PNDOUT1 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	2.00	-----	0.0	0.0	0.00	755.00
11.100	2.00	4.0	3.8	4.0	0.10	755.87
11.200	2.00	4.0	7.6	7.8	0.12	756.12
11.300	2.00	4.0	11.3	11.6	0.12	756.27
11.400	2.00	4.0	15.1	15.3	0.12	756.43
11.500	3.00	5.0	19.8	20.1	0.12	756.62
11.600	3.00	6.0	25.6	25.8	0.12	756.86
11.700	4.00	7.0	32.4	32.6	0.12	757.07
11.800	4.00	8.0	40.1	40.4	0.12	757.23
11.900	5.00	9.0	48.9	49.1	0.12	757.41
12.000	9.00	14.0	62.6	62.9	0.12	757.70
12.100	16.00	25.0	87.4	87.6	0.12	758.09
12.200	29.00	45.0	132.2	132.4	0.12	758.48
12.300	44.00	73.0	204.9	205.2	0.12	759.07
12.400	49.00	93.0	297.7	297.9	0.12	759.54
12.500	47.00	96.0	393.4	393.7	0.12	760.01
12.600	37.00	84.0	477.2	477.4	0.12	760.27
12.700	28.00	65.0	542.0	542.2	0.12	760.47
12.800	21.00	49.0	590.7	591.0	0.12	760.63
12.900	17.00	38.0	628.5	628.7	0.12	760.74
13.000	13.00	30.0	658.2	658.5	0.12	760.83
13.100	11.00	24.0	682.0	682.2	0.12	760.91
13.200	9.00	20.0	701.8	702.0	0.12	760.97
13.300	8.00	17.0	718.5	718.8	0.12	761.02
13.400	7.00	15.0	733.3	733.5	0.12	761.05
13.500	7.00	14.0	747.0	747.3	0.12	761.09
13.600	6.00	13.0	759.8	760.0	0.12	761.12
13.700	6.00	12.0	771.6	771.8	0.12	761.15
13.800	5.00	11.0	782.3	782.6	0.12	761.18
13.900	4.00	9.0	791.1	791.3	0.12	761.20
14.000	4.00	8.0	798.8	799.1	0.12	761.22
14.100	4.00	8.0	806.6	806.8	0.12	761.24
14.200	4.00	8.0	814.4	814.6	0.12	761.26
14.300	4.00	8.0	822.1	822.4	0.12	761.28
14.400	4.00	8.0	829.9	830.1	0.12	761.30
14.500	3.00	7.0	836.6	836.9	0.12	761.31
14.600	3.00	6.0	842.4	842.6	0.12	761.33
14.700	3.00	6.0	848.2	848.4	0.12	761.34
14.800	3.00	6.0	853.9	854.2	0.12	761.36
14.900	3.00	6.0	859.7	859.9	0.12	761.37
15.000	3.00	6.0	865.4	865.7	0.12	761.39
15.100	3.00	6.0	871.2	871.4	0.12	761.40
15.200	3.00	6.0	877.0	877.2	0.12	761.41
15.300	3.00	6.0	882.7	883.0	0.12	761.43
15.400	3.00	6.0	888.5	888.7	0.12	761.44

nd File: A:POND1 .PND
flow Hydrograph: A:PIED1 .HYD
Outflow Hydrograph: A:PNDOUT1 .HYD

INFLOW HYDROGRAPH

ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	3.00	6.0	894.2	894.5	0.12	761.46
15.600	3.00	6.0	900.0	900.2	0.12	761.47
15.700	3.00	6.0	905.8	906.0	0.12	761.49
15.800	2.00	5.0	910.5	910.8	0.12	761.50
15.900	2.00	4.0	914.3	914.5	0.12	761.51
16.000	2.00	4.0	918.0	918.3	0.12	761.52
16.100	2.00	4.0	921.8	922.0	0.12	761.53
16.200	2.00	4.0	925.6	925.8	0.12	761.54
16.300	2.00	4.0	929.3	929.6	0.12	761.55
16.400	2.00	4.0	933.1	933.3	0.12	761.56
16.500	2.00	4.0	936.8	937.1	0.12	761.56
16.600	2.00	4.0	940.6	940.8	0.12	761.57
16.700	2.00	4.0	944.4	944.6	0.12	761.58
16.800	2.00	4.0	948.1	948.4	0.12	761.59
16.900	2.00	4.0	951.9	952.1	0.12	761.60
17.000	2.00	4.0	955.6	955.9	0.12	761.61
17.100	2.00	4.0	959.4	959.6	0.12	761.62
7.200	2.00	4.0	963.2	963.4	0.12	761.63
7.300	2.00	4.0	966.9	967.2	0.12	761.64
17.400	2.00	4.0	970.7	970.9	0.12	761.65
17.500	2.00	4.0	974.4	974.7	0.12	761.66
17.600	2.00	4.0	978.2	978.4	0.12	761.67
17.700	2.00	4.0	982.0	982.2	0.12	761.68
17.800	2.00	4.0	985.7	986.0	0.12	761.69
17.900	2.00	4.0	989.5	989.7	0.12	761.70
18.000	2.00	4.0	993.2	993.5	0.12	761.71
18.100	2.00	4.0	997.0	997.2	0.12	761.72
18.200	2.00	4.0	1000.8	1001.0	0.12	761.72
18.300	2.00	4.0	1004.5	1004.8	0.12	761.73
18.400	2.00	4.0	1008.3	1008.5	0.12	761.74
18.500	2.00	4.0	1012.0	1012.3	0.12	761.75
18.600	1.00	3.0	1014.8	1015.0	0.12	761.76
18.700	1.00	2.0	1016.6	1016.8	0.12	761.76
18.800	1.00	2.0	1018.3	1018.6	0.12	761.77
18.900	1.00	2.0	1020.1	1020.3	0.12	761.77
19.000	1.00	2.0	1021.8	1022.1	0.12	761.78
19.100	1.00	2.0	1023.6	1023.8	0.12	761.78
19.200	1.00	2.0	1025.4	1025.6	0.12	761.79
19.300	1.00	2.0	1027.1	1027.4	0.12	761.79
19.400	1.00	2.0	1028.9	1029.1	0.12	761.80
19.500	1.00	2.0	1030.6	1030.9	0.12	761.80
19.600	1.00	2.0	1032.4	1032.6	0.12	761.80
9.700	1.00	2.0	1034.2	1034.4	0.12	761.81
9.800	1.00	2.0	1035.9	1036.2	0.12	761.81
19.900	1.00	2.0	1037.7	1037.9	0.12	761.82
20.000	1.00	2.0	1039.4	1039.7	0.12	761.82

nd File: A:POND1 .PND
.nflow Hydrograph: A:PIED1 .HYD
Outflow Hydrograph: A:PNDOUT1 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
20.100	1.00
20.200	1.00
20.300	1.00
20.400	1.00
20.500	1.00
20.600	1.00
20.700	1.00
20.800	1.00
20.900	1.00
21.000	1.00
21.100	1.00
21.200	1.00
21.300	1.00
21.400	1.00
21.500	1.00
21.600	1.00
21.700	1.00
21.800	1.00
21.900	1.00
22.000	1.00
22.100	1.00
22.200	1.00
22.300	1.00
22.400	1.00
22.500	1.00
22.600	1.00
22.700	1.00
22.800	1.00
22.900	1.00
23.000	1.00
23.100	1.00
23.200	1.00
23.300	1.00
23.400	1.00
23.500	1.00
23.600	1.00
23.700	1.00
23.800	1.00
23.900	1.00
24.000	0.00
24.100	0.00
24.200	0.00
24.300	0.00
24.400	0.00
24.500	0.00
24.600	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
2.0	1041.2	1041.4	0.12	761.83
2.0	1043.0	1043.2	0.12	761.83
2.0	1044.7	1045.0	0.12	761.83
2.0	1046.5	1046.7	0.12	761.84
2.0	1048.2	1048.5	0.12	761.84
2.0	1050.0	1050.2	0.12	761.85
2.0	1051.8	1052.0	0.12	761.85
2.0	1053.5	1053.8	0.12	761.86
2.0	1055.3	1055.5	0.12	761.86
2.0	1057.0	1057.3	0.12	761.87
2.0	1058.8	1059.0	0.12	761.87
2.0	1060.6	1060.8	0.12	761.87
2.0	1062.3	1062.6	0.12	761.88
2.0	1064.1	1064.3	0.12	761.88
2.0	1065.8	1066.1	0.12	761.89
2.0	1067.6	1067.8	0.12	761.89
2.0	1069.4	1069.6	0.12	761.90
2.0	1071.1	1071.4	0.12	761.90
2.0	1072.9	1073.1	0.12	761.91
2.0	1074.6	1074.9	0.12	761.91
2.0	1076.4	1076.6	0.12	761.91
2.0	1078.2	1078.4	0.12	761.92
2.0	1079.9	1080.2	0.12	761.92
2.0	1081.7	1081.9	0.12	761.93
2.0	1083.4	1083.7	0.12	761.93
2.0	1085.2	1085.4	0.12	761.94
2.0	1087.0	1087.2	0.12	761.94
2.0	1088.7	1089.0	0.12	761.95
2.0	1090.5	1090.7	0.12	761.95
2.0	1092.2	1092.5	0.12	761.95
2.0	1094.0	1094.2	0.12	761.96
2.0	1095.8	1096.0	0.12	761.96
2.0	1097.5	1097.8	0.12	761.97
2.0	1099.3	1099.5	0.12	761.97
2.0	1101.0	1101.3	0.12	761.98
2.0	1102.8	1103.0	0.12	761.98
2.0	1104.6	1104.8	0.12	761.98
2.0	1106.3	1106.6	0.12	761.99
2.0	1108.1	1108.3	0.12	761.99
1.0	1108.8	1109.1	0.12	762.00
0.0	1108.6	1108.8	0.12	761.99
0.0	1108.4	1108.6	0.12	761.99
0.0	1108.1	1108.4	0.12	761.99
0.0	1107.9	1108.1	0.12	761.99
0.0	1107.6	1107.9	0.12	761.99
0.0	1107.4	1107.6	0.12	761.99

nd File: A:POND1 .PND
flow Hydrograph: A:PIED1 .HYD
outflow Hydrograph: A:PNDOUT1 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
24.700	0.00
24.800	0.00
24.900	0.00
25.000	0.00
25.100	0.00
25.200	0.00
25.300	0.00
25.400	0.00
25.500	0.00
25.600	0.00
25.700	0.00
25.800	0.00
25.900	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
0.0	1107.2	1107.4	0.12	761.99
0.0	1106.9	1107.2	0.12	761.99
0.0	1106.7	1106.9	0.12	761.99
0.0	1106.4	1106.7	0.12	761.99
0.0	1106.2	1106.4	0.12	761.99
0.0	1106.0	1106.2	0.12	761.99
0.0	1105.7	1106.0	0.12	761.99
0.0	1105.5	1105.7	0.12	761.99
0.0	1105.2	1105.5	0.12	761.99
0.0	1105.0	1105.2	0.12	761.99
0.0	1104.8	1105.0	0.12	761.99
0.0	1104.5	1104.8	0.12	761.98
0.0	1104.3	1104.5	0.12	761.98

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: A:POND1 .PND
Inflow Hydrograph: A:PIED1 .HYD
Outflow Hydrograph: A:PNDOUT1 .HYD

Starting Pond W.S. Elevation = 755.00 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	49.00 cfs
Peak Outflow	=	0.12 cfs
Peak Elevation	=	762.00 ft

***** Summary of Approximate Peak Storage *****

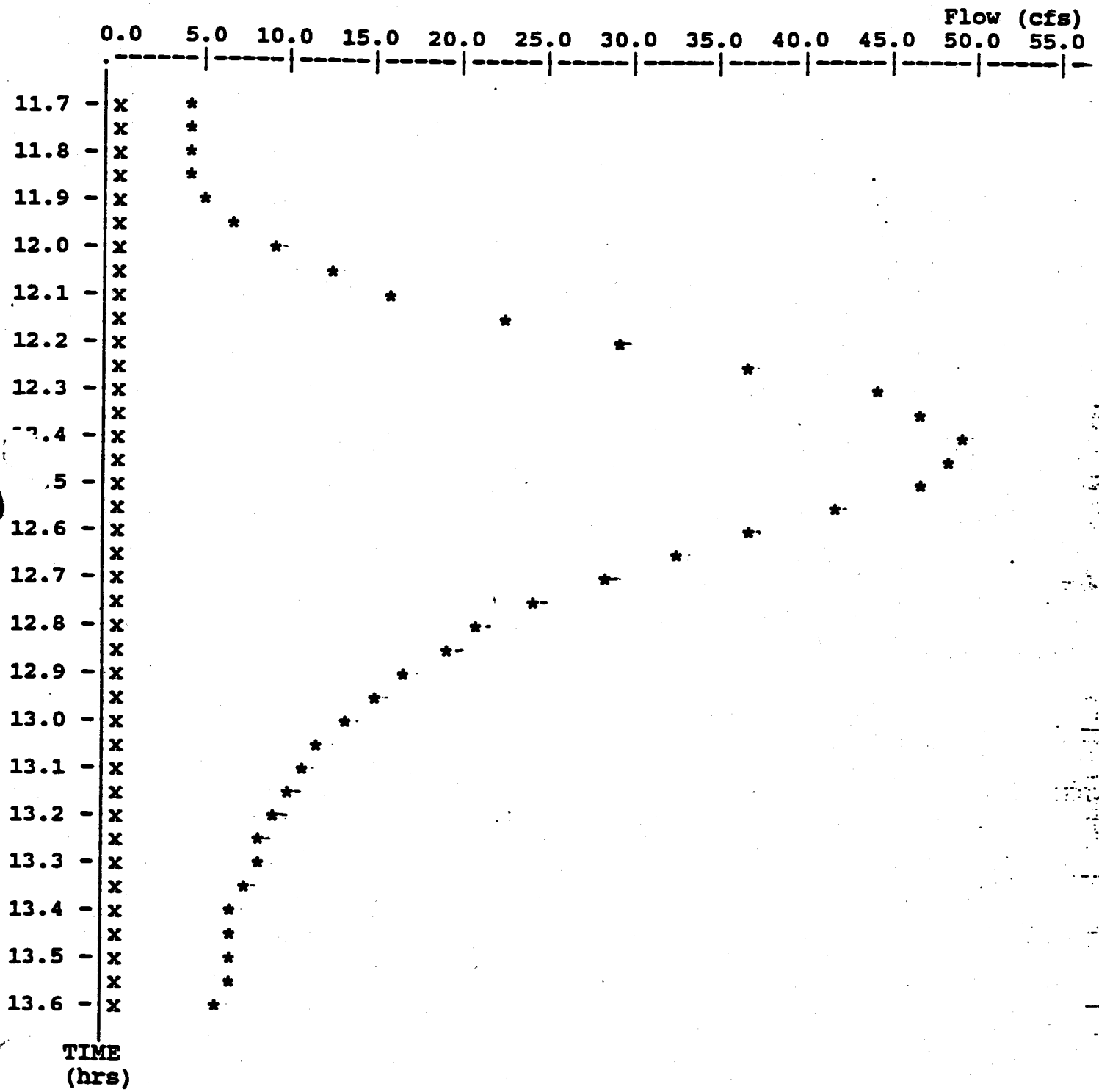
Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	4.58 ac-ft
Total Storage in Pond	=	4.58 ac-ft

Warning: Inflow hydrograph truncated on left side.

Pond File: A:POND1 .PND
Inflow Hydrograph: A:PIED1 .HYD
Outflow Hydrograph: A:PNDOUT1 .HYD

EXECUTED: 10-14-1993
09:54:33

Peak Inflow = 49.00 cfs
Peak Outflow = 0.12 cfs
Peak Elevation = 762.00 ft



* File: A:PIED1 .HYD Qmax = 49.0 cfs
x File: A:PNDOUT1 .HYD Qmax = 0.1 cfs

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:44:03
Watershed file: --> A:PIED322 .WSD
Hydrograph file: --> A:PIED322 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL2

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
322	9.27	80.0	0.40	0.00	8.40	6.00	.06 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 9.27 acres or 0.01448 sq.mi
Peak discharge = 51 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
322	0.41	0.00	0.40	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:44:03
Watershed file: --> A:PIED322 .WSD
Hydrograph file: --> A:PIED322 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL2

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
322	51	12.3
Composite Watershed	51	12.3

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:44:03
Watershed file: --> A:PIED322 .WSD
Hydrograph file: --> A:PIED322 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL2

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
322	2	2	3	7	12	24	41	51	50
Total (cfs)	2	2	3	7	12	24	41	51	50

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
322	37	26	19	14	9	7	5	5	4
Total (cfs)	37	26	19	14	9	7	5	5	4

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
322	4	3	3	3	2	2	2	2	2
Total (cfs)	4	3	3	3	2	2	2	2	2

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
322	2	1	1	1	0
Total (cfs)	2	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:44:03
Watershed file: --> A:PIED322 .WSD
Hydrograph file: --> A:PIED322 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL2

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	2	14.8	3
11.1	2	14.9	3
11.2	2	15.0	3
11.3	2	15.1	3
11.4	2	15.2	3
11.5	3	15.3	2
11.6	3	15.4	2
11.7	4	15.5	2
11.8	6	15.6	2
11.9	7	15.7	2
12.0	12	15.8	2
12.1	24	15.9	2
12.2	41	16.0	2
12.3	51	16.1	2
12.4	50	16.2	2
12.5	37	16.3	2
12.6	26	16.4	2
12.7	19	16.5	2
12.8	14	16.6	2
12.9	11	16.7	2
13.0	9	16.8	2
13.1	8	16.9	2
13.2	7	17.0	2
13.3	6	17.1	2
13.4	5	17.2	2
13.5	5	17.3	2
13.6	5	17.4	2
13.7	4	17.5	2
13.8	4	17.6	2
13.9	4	17.7	2
14.0	4	17.8	2
14.1	4	17.9	2
14.2	3	18.0	2
14.3	3	18.1	2
14.4	3	18.2	2
14.5	3	18.3	2
14.6	3	18.4	2

14.7	3	18.5	2

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:44:03
Watershed file: --> A:PIED322 .WSD
Hydrograph file: --> A:PIED322 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL2-SUBCELL2

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
PHASE 3 - CELL 2 - SUBCELL 2
24 HR-25 YR STORM STORAGE

Inflow Hydrograph: A:PIED2 .HYD
Rating Table file: A:POND2 .PND

----INITIAL CONDITIONS----
Elevation = 771.00 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA			INTERMEDIATE ROUTING COMPUTATIONS	
ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)	2S/t (cfs)	2S/t + O (cfs)
771.00	0.0	0.000	0.0	0.0
771.50	0.0	0.006	1.5	1.5
772.00	0.0	0.012	2.9	2.9
773.00	0.0	0.060	14.5	14.5
774.00	0.0	0.230	55.7	55.7
775.00	0.0	0.520	125.8	125.8
776.00	0.0	1.310	317.0	317.0
777.00	0.0	2.510	607.4	607.4
778.00	0.0	4.460	1079.3	1079.3
778.50	0.0	5.720	1384.2	1384.2

Time increment (t) = 0.100 hrs.

File: A:POND2 .PND
ow Hydrograph: A:PIED2 .HYD
utflow Hydrograph: A:PNDOUT2 .HYD

INFLOW HYDROGRAPH | ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	2.00	-----	0.0	0.0	0.00	771.00
11.100	2.00	4.0	4.0	4.0	0.00	772.09
11.200	2.00	4.0	8.0	8.0	0.00	772.44
11.300	2.00	4.0	12.0	12.0	0.00	772.78
11.400	2.00	4.0	16.0	16.0	0.00	773.04
11.500	3.00	5.0	21.0	21.0	0.00	773.16
11.600	3.00	6.0	27.0	27.0	0.00	773.30
11.700	4.00	7.0	34.0	34.0	0.00	773.47
11.800	6.00	10.0	44.0	44.0	0.00	773.72
11.900	7.00	13.0	57.0	57.0	0.00	774.02
12.000	12.00	19.0	76.0	76.0	0.00	774.29
12.100	24.00	36.0	112.0	112.0	0.00	774.80
12.200	41.00	65.0	177.0	177.0	0.00	775.27
12.300	51.00	92.0	269.0	269.0	0.00	775.75
12.400	50.00	101.0	370.0	370.0	0.00	776.18
12.500	37.00	87.0	457.0	457.0	0.00	776.48
.600	26.00	63.0	520.0	520.0	0.00	776.70
'00	19.00	45.0	565.0	565.0	0.00	776.85
.800	14.00	33.0	598.0	598.0	0.00	776.97
12.900	12.00	26.0	624.0	624.0	0.00	777.04
13.000	9.00	21.0	645.0	645.0	0.00	777.08
13.100	8.00	17.0	662.0	662.0	0.00	777.12
13.200	7.00	15.0	677.0	677.0	0.00	777.15
13.300	6.00	13.0	690.0	690.0	0.00	777.17
13.400	5.00	11.0	701.0	701.0	0.00	777.20
13.500	5.00	10.0	711.0	711.0	0.00	777.22
13.600	5.00	10.0	721.0	721.0	0.00	777.24
13.700	4.00	9.0	730.0	730.0	0.00	777.26
13.800	4.00	8.0	738.0	738.0	0.00	777.28
13.900	4.00	8.0	746.0	746.0	0.00	777.29
14.000	4.00	8.0	754.0	754.0	0.00	777.31
14.100	4.00	8.0	762.0	762.0	0.00	777.33
14.200	3.00	7.0	769.0	769.0	0.00	777.34
14.300	3.00	6.0	775.0	775.0	0.00	777.36
14.400	3.00	6.0	781.0	781.0	0.00	777.37
14.500	3.00	6.0	787.0	787.0	0.00	777.38
14.600	3.00	6.0	793.0	793.0	0.00	777.39
14.700	3.00	6.0	799.0	799.0	0.00	777.41
14.800	3.00	6.0	805.0	805.0	0.00	777.42
14.900	3.00	6.0	811.0	811.0	0.00	777.43
15.000	3.00	6.0	817.0	817.0	0.00	777.44
15.100	3.00	6.0	823.0	823.0	0.00	777.46
200	3.00	6.0	829.0	829.0	0.00	777.47
300	2.00	5.0	834.0	834.0	0.00	777.48
15.400	2.00	4.0	838.0	838.0	0.00	777.49

File: A:POND2 .PND
...w Hydrograph: A:PIED2 .HYD
...tflow Hydrograph: A:PNDOUT2 .HYD

INFLOW HYDROGRAPH | ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	2.00	4.0	842.0	842.0	0.00	777.50
15.600	2.00	4.0	846.0	846.0	0.00	777.51
15.700	2.00	4.0	850.0	850.0	0.00	777.51
15.800	2.00	4.0	854.0	854.0	0.00	777.52
15.900	2.00	4.0	858.0	858.0	0.00	777.53
16.000	2.00	4.0	862.0	862.0	0.00	777.54
16.100	2.00	4.0	866.0	866.0	0.00	777.55
16.200	2.00	4.0	870.0	870.0	0.00	777.56
16.300	2.00	4.0	874.0	874.0	0.00	777.56
16.400	2.00	4.0	878.0	878.0	0.00	777.57
16.500	2.00	4.0	882.0	882.0	0.00	777.58
16.600	2.00	4.0	886.0	886.0	0.00	777.59
16.700	2.00	4.0	890.0	890.0	0.00	777.60
16.800	2.00	4.0	894.0	894.0	0.00	777.61
16.900	2.00	4.0	898.0	898.0	0.00	777.62
17.000	2.00	4.0	902.0	902.0	0.00	777.62
.100	2.00	4.0	906.0	906.0	0.00	777.63
200	2.00	4.0	910.0	910.0	0.00	777.64
.300	2.00	4.0	914.0	914.0	0.00	777.65
17.400	2.00	4.0	918.0	918.0	0.00	777.66
17.500	2.00	4.0	922.0	922.0	0.00	777.67
17.600	2.00	4.0	926.0	926.0	0.00	777.68
17.700	2.00	4.0	930.0	930.0	0.00	777.68
17.800	2.00	4.0	934.0	934.0	0.00	777.69
17.900	2.00	4.0	938.0	938.0	0.00	777.70
18.000	2.00	4.0	942.0	942.0	0.00	777.71
18.100	2.00	4.0	946.0	946.0	0.00	777.72
18.200	2.00	4.0	950.0	950.0	0.00	777.73
18.300	2.00	4.0	954.0	954.0	0.00	777.73
18.400	2.00	4.0	958.0	958.0	0.00	777.74
18.500	2.00	4.0	962.0	962.0	0.00	777.75
18.600	1.00	3.0	965.0	965.0	0.00	777.76
18.700	1.00	2.0	967.0	967.0	0.00	777.76
18.800	1.00	2.0	969.0	969.0	0.00	777.77
18.900	1.00	2.0	971.0	971.0	0.00	777.77
19.000	1.00	2.0	973.0	973.0	0.00	777.77
19.100	1.00	2.0	975.0	975.0	0.00	777.78
19.200	1.00	2.0	977.0	977.0	0.00	777.78
19.300	1.00	2.0	979.0	979.0	0.00	777.79
19.400	1.00	2.0	981.0	981.0	0.00	777.79
19.500	1.00	2.0	983.0	983.0	0.00	777.80
19.600	1.00	2.0	985.0	985.0	0.00	777.80
700	1.00	2.0	987.0	987.0	0.00	777.80
800	1.00	2.0	989.0	989.0	0.00	777.81
19.900	1.00	2.0	991.0	991.0	0.00	777.81
20.000	1.00	2.0	993.0	993.0	0.00	777.82

File: A:POND2 .PND
ow Hydrograph: A:PIED2 .HYD
Outflow Hydrograph: A:PNDOUT2 .HYD

INFLOW HYDROGRAPH / ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	995.0	995.0	0.00	777.82
20.200	1.00	2.0	997.0	997.0	0.00	777.83
20.300	1.00	2.0	999.0	999.0	0.00	777.83
20.400	1.00	2.0	1001.0	1001.0	0.00	777.83
20.500	1.00	2.0	1003.0	1003.0	0.00	777.84
20.600	1.00	2.0	1005.0	1005.0	0.00	777.84
20.700	1.00	2.0	1007.0	1007.0	0.00	777.85
20.800	1.00	2.0	1009.0	1009.0	0.00	777.85
20.900	1.00	2.0	1011.0	1011.0	0.00	777.86
21.000	1.00	2.0	1013.0	1013.0	0.00	777.86
21.100	1.00	2.0	1015.0	1015.0	0.00	777.86
21.200	1.00	2.0	1017.0	1017.0	0.00	777.87
21.300	1.00	2.0	1019.0	1019.0	0.00	777.87
21.400	1.00	2.0	1021.0	1021.0	0.00	777.88
21.500	1.00	2.0	1023.0	1023.0	0.00	777.88
21.600	1.00	2.0	1025.0	1025.0	0.00	777.88
[illegible].700	1.00	2.0	1027.0	1027.0	0.00	777.89
800	1.00	2.0	1029.0	1029.0	0.00	777.89
.900	1.00	2.0	1031.0	1031.0	0.00	777.90
22.000	1.00	2.0	1033.0	1033.0	0.00	777.90
22.100	1.00	2.0	1035.0	1035.0	0.00	777.91
22.200	1.00	2.0	1037.0	1037.0	0.00	777.91
22.300	1.00	2.0	1039.0	1039.0	0.00	777.91
22.400	1.00	2.0	1041.0	1041.0	0.00	777.92
22.500	1.00	2.0	1043.0	1043.0	0.00	777.92
22.600	1.00	2.0	1045.0	1045.0	0.00	777.93
22.700	1.00	2.0	1047.0	1047.0	0.00	777.93
22.800	1.00	2.0	1049.0	1049.0	0.00	777.94
22.900	1.00	2.0	1051.0	1051.0	0.00	777.94
23.000	1.00	2.0	1053.0	1053.0	0.00	777.94
23.100	1.00	2.0	1055.0	1055.0	0.00	777.95
23.200	1.00	2.0	1057.0	1057.0	0.00	777.95
23.300	1.00	2.0	1059.0	1059.0	0.00	777.96
23.400	1.00	2.0	1061.0	1061.0	0.00	777.96
23.500	1.00	2.0	1063.0	1063.0	0.00	777.97
23.600	1.00	2.0	1065.0	1065.0	0.00	777.97
23.700	1.00	2.0	1067.0	1067.0	0.00	777.97
23.800	1.00	2.0	1069.0	1069.0	0.00	777.98
23.900	1.00	2.0	1071.0	1071.0	0.00	777.98
24.000	0.00	1.0	1072.0	1072.0	0.00	777.98
24.100	0.00	0.0	1072.0	1072.0	0.00	777.98
24.200	0.00	0.0	1072.0	1072.0	0.00	777.98
300	0.00	0.0	1072.0	1072.0	0.00	777.98
400	0.00	0.0	1072.0	1072.0	0.00	777.98
24.500	0.00	0.0	1072.0	1072.0	0.00	777.98
24.600	0.00	0.0	1072.0	1072.0	0.00	777.98

P File: A:POND2 .PND
Hydrograph: A:PIED2 .HYD
Outflow Hydrograph: A:PNDOUT2 .HYD

INFLOW HYDROGRAPH — ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
24.700	0.00	0.0	1072.0	1072.0	0.00	777.98
24.800	0.00	0.0	1072.0	1072.0	0.00	777.98
24.900	0.00	0.0	1072.0	1072.0	0.00	777.98
25.000	0.00	0.0	1072.0	1072.0	0.00	777.98
25.100	0.00	0.0	1072.0	1072.0	0.00	777.98
25.200	0.00	0.0	1072.0	1072.0	0.00	777.98
25.300	0.00	0.0	1072.0	1072.0	0.00	777.98
25.400	0.00	0.0	1072.0	1072.0	0.00	777.98
25.500	0.00	0.0	1072.0	1072.0	0.00	777.98
25.600	0.00	0.0	1072.0	1072.0	0.00	777.98
25.700	0.00	0.0	1072.0	1072.0	0.00	777.98
25.800	0.00	0.0	1072.0	1072.0	0.00	777.98
25.900	0.00	0.0	1072.0	1072.0	0.00	777.98

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: A:POND2 .PND
Inflow Hydrograph: A:PIED2 .HYD
Outflow Hydrograph: A:PNDOUT2 .HYD

Starting Pond W.S. Elevation = 771.00 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	51.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	777.98 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	4.43 ac-ft
Total Storage in Pond	=	4.43 ac-ft

Warning: Inflow hydrograph truncated on left side.

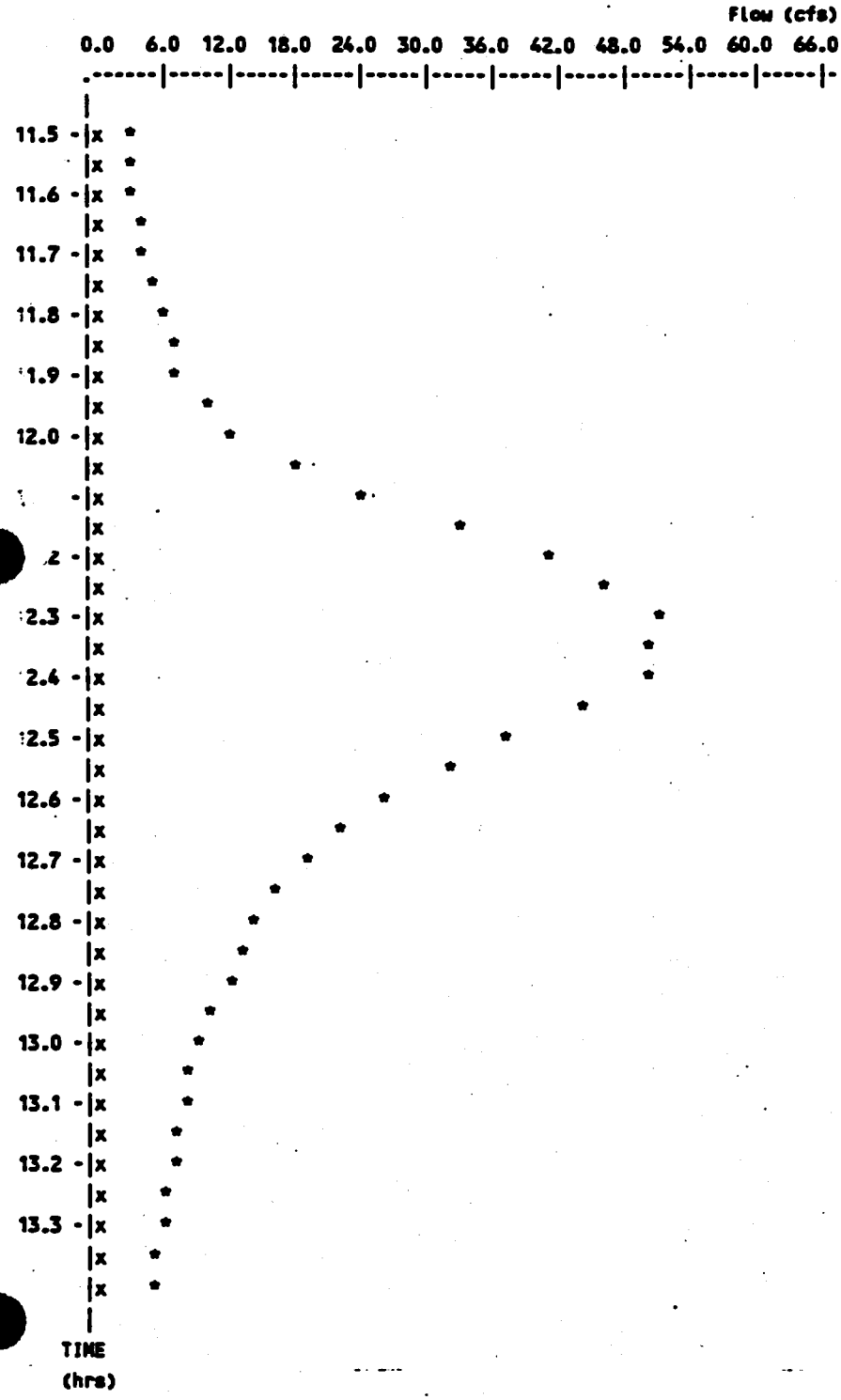
>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: A:POND2 .PND
Inflow Hydrograph: A:PIED2 .HYD
Outflow Hydrograph: A:PNDOUT2 .HYD

EXECUTED: 10-15-1993
10:13:26

Peak Inflow = 51.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 777.98 ft



* File: A:PIED2 .HYD Qmax = 51.0 cfs
x File: A:PNDOUT2 .HYD Qmax = 0.0 cfs

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:45:43
Watershed file: --> A:PIED311 .WSD
Hydrograph file: --> A:PIED311 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL1

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
311	6.01	80.0	0.50	0.00	8.40	6.00	.06 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 6.01 acres or 0.00939 sq.mi
Peak discharge = 30 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
311	0.51	0.00	0.50	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:45:43
Watershed file: --> A:PIED311 .WSD
Hydrograph file: --> A:PIED311 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL1

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
311	30	12.4
Composite Watershed	30	12.4

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:45:43
Watershed file: --> A:PIED311 .WSD
Hydrograph file: --> A:PIED311 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL1

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
311	1	1	2	3	5	10	17	26	30
Total (cfs)	1	1	2	3	5	10	17	26	30

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
311	29	23	17	13	8	5	4	3	3
Total (cfs)	29	23	17	13	8	5	4	3	3

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
311	3	2	2	2	2	1	1	1	1
Total (cfs)	3	2	2	2	2	1	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
311	1	1	1	1	0
Total (cfs)	1	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:45:43
Watershed file: --> A:PIED311 .WSD
Hydrograph file: --> A:PIED311 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL1

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	2
11.1	1	14.9	2
11.2	1	15.0	2
11.3	1	15.1	2
11.4	1	15.2	2
11.5	2	15.3	2
11.6	2	15.4	2
11.7	2	15.5	2
11.8	3	15.6	2
11.9	3	15.7	2
12.0	5	15.8	1
12.1	10	15.9	1
12.2	17	16.0	1
12.3	26	16.1	1
12.4	30	16.2	1
12.5	29	16.3	1
12.6	23	16.4	1
12.7	17	16.5	1
12.8	13	16.6	1
12.9	10	16.7	1
13.0	8	16.8	1
13.1	6	16.9	1
13.2	5	17.0	1
13.3	4	17.1	1
13.4	4	17.2	1
13.5	4	17.3	1
13.6	3	17.4	1
13.7	3	17.5	1
13.8	3	17.6	1
13.9	3	17.7	1
14.0	3	17.8	1
14.1	3	17.9	1
14.2	2	18.0	1
14.3	2	18.1	1
14.4	2	18.2	1
14.5	2	18.3	1
14.6	2	18.4	1

14.7	2	18.5	1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:45:43
Watershed file: --> A:PIED311 .WSD
Hydrograph file: --> A:PIED311 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL1

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* PIEDMONT LANDFILL AND RECYCLING CENTER *
* KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA *
* PHASE 3 - CELL 1 - SUBCELL 1 *
* 24 HR-25 YR STORM STORAGE *
*
****

Inflow Hydrograph: A:PIED4 .HYD
Rating Table file: A:POND4 .PND

----INITIAL CONDITIONS----
Elevation = 765.80 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
765.80	0.0	0.000
766.30	0.0	0.010
766.80	0.0	0.030
767.80	0.0	0.140
768.80	0.0	0.420
769.80	0.0	0.970
770.80	0.0	2.150
771.30	0.0	3.010

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
2.4	2.4
7.3	7.3
33.9	33.9
101.6	101.6
234.7	234.7
520.3	520.3
728.4	728.4

Time increment (t) = 0.100 hrs.

nd File: A:POND4 .PND
.flow Hydrograph: A:PIED4 .HYD
Outflow Hydrograph: A:PNDOUT4 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
11.000	1.00
11.100	1.00
11.200	1.00
11.300	1.00
11.400	1.00
11.500	2.00
11.600	2.00
11.700	2.00
11.800	3.00
11.900	3.00
12.000	5.00
12.100	10.00
12.200	17.00
12.300	26.00
12.400	30.00
12.500	29.00
12.600	23.00
12.700	17.00
12.800	13.00
12.900	10.00
13.000	8.00
13.100	6.00
13.200	5.00
13.300	4.00
13.400	4.00
13.500	4.00
13.600	3.00
13.700	3.00
13.800	3.00
13.900	3.00
14.000	3.00
14.100	3.00
14.200	2.00
14.300	2.00
14.400	2.00
14.500	2.00
14.600	2.00
14.700	2.00
14.800	2.00
14.900	2.00
15.000	2.00
15.100	2.00
15.200	2.00
15.300	2.00
15.400	2.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
-----	0.0	0.0	0.00	765.80
2.0	2.0	2.0	0.00	766.21
2.0	4.0	4.0	0.00	766.46
2.0	6.0	6.0	0.00	766.67
2.0	8.0	8.0	0.00	766.83
3.0	11.0	11.0	0.00	766.94
4.0	15.0	15.0	0.00	767.09
4.0	19.0	19.0	0.00	767.24
5.0	24.0	24.0	0.00	767.43
6.0	30.0	30.0	0.00	767.65
8.0	38.0	38.0	0.00	767.86
15.0	53.0	53.0	0.00	768.08
27.0	80.0	80.0	0.00	768.48
43.0	123.0	123.0	0.00	768.96
56.0	179.0	179.0	0.00	769.38
59.0	238.0	238.0	0.00	769.81
52.0	290.0	290.0	0.00	769.99
40.0	330.0	330.0	0.00	770.13
30.0	360.0	360.0	0.00	770.24
23.0	383.0	383.0	0.00	770.32
18.0	401.0	401.0	0.00	770.38
14.0	415.0	415.0	0.00	770.43
11.0	426.0	426.0	0.00	770.47
9.0	435.0	435.0	0.00	770.50
8.0	443.0	443.0	0.00	770.53
8.0	451.0	451.0	0.00	770.56
7.0	458.0	458.0	0.00	770.58
6.0	464.0	464.0	0.00	770.60
6.0	470.0	470.0	0.00	770.62
6.0	476.0	476.0	0.00	770.64
6.0	482.0	482.0	0.00	770.67
6.0	488.0	488.0	0.00	770.69
5.0	493.0	493.0	0.00	770.70
4.0	497.0	497.0	0.00	770.72
4.0	501.0	501.0	0.00	770.73
4.0	505.0	505.0	0.00	770.75
4.0	509.0	509.0	0.00	770.76
4.0	513.0	513.0	0.00	770.77
4.0	517.0	517.0	0.00	770.79
4.0	521.0	521.0	0.00	770.80
4.0	525.0	525.0	0.00	770.81
4.0	529.0	529.0	0.00	770.82
4.0	533.0	533.0	0.00	770.83
4.0	537.0	537.0	0.00	770.84
4.0	541.0	541.0	0.00	770.85

nd File: A:POND4 .PND
..flow Hydrograph: A:PIED4 .HYD
Outflow Hydrograph: A:PNDOUT4 .HYD

INFLOW HYDROGRAPH

ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	2.00	4.0	545.0	545.0	0.00	770.86
15.600	2.00	4.0	549.0	549.0	0.00	770.87
15.700	2.00	4.0	553.0	553.0	0.00	770.88
15.800	1.00	3.0	556.0	556.0	0.00	770.89
15.900	1.00	2.0	558.0	558.0	0.00	770.89
16.000	1.00	2.0	560.0	560.0	0.00	770.90
16.100	1.00	2.0	562.0	562.0	0.00	770.90
16.200	1.00	2.0	564.0	564.0	0.00	770.90
16.300	1.00	2.0	566.0	566.0	0.00	770.91
16.400	1.00	2.0	568.0	568.0	0.00	770.91
16.500	1.00	2.0	570.0	570.0	0.00	770.92
16.600	1.00	2.0	572.0	572.0	0.00	770.92
16.700	1.00	2.0	574.0	574.0	0.00	770.93
16.800	1.00	2.0	576.0	576.0	0.00	770.93
16.900	1.00	2.0	578.0	578.0	0.00	770.94
17.000	1.00	2.0	580.0	580.0	0.00	770.94
17.100	1.00	2.0	582.0	582.0	0.00	770.95
17.200	1.00	2.0	584.0	584.0	0.00	770.95
17.300	1.00	2.0	586.0	586.0	0.00	770.96
17.400	1.00	2.0	588.0	588.0	0.00	770.96
17.500	1.00	2.0	590.0	590.0	0.00	770.97
17.600	1.00	2.0	592.0	592.0	0.00	770.97
17.700	1.00	2.0	594.0	594.0	0.00	770.98
17.800	1.00	2.0	596.0	596.0	0.00	770.98
17.900	1.00	2.0	598.0	598.0	0.00	770.99
18.000	1.00	2.0	600.0	600.0	0.00	770.99
18.100	1.00	2.0	602.0	602.0	0.00	771.00
18.200	1.00	2.0	604.0	604.0	0.00	771.00
18.300	1.00	2.0	606.0	606.0	0.00	771.01
18.400	1.00	2.0	608.0	608.0	0.00	771.01
18.500	1.00	2.0	610.0	610.0	0.00	771.02
18.600	1.00	2.0	612.0	612.0	0.00	771.02
18.700	1.00	2.0	614.0	614.0	0.00	771.03
18.800	1.00	2.0	616.0	616.0	0.00	771.03
18.900	1.00	2.0	618.0	618.0	0.00	771.03
19.000	1.00	2.0	620.0	620.0	0.00	771.04
19.100	1.00	2.0	622.0	622.0	0.00	771.04
19.200	1.00	2.0	624.0	624.0	0.00	771.05
19.300	1.00	2.0	626.0	626.0	0.00	771.05
19.400	1.00	2.0	628.0	628.0	0.00	771.06
19.500	1.00	2.0	630.0	630.0	0.00	771.06
19.600	1.00	2.0	632.0	632.0	0.00	771.07
19.700	1.00	2.0	634.0	634.0	0.00	771.07
19.800	1.00	2.0	636.0	636.0	0.00	771.08
19.900	1.00	2.0	638.0	638.0	0.00	771.08
20.000	1.00	2.0	640.0	640.0	0.00	771.09

Pond File: A:POND4 .PND
Inflow Hydrograph: A:PIED4 .HYD
Outflow Hydrograph: A:PNDOUT4 .HYD

INFLOW HYDROGRAPH

ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	642.0	642.0	0.00	771.09
20.200	1.00	2.0	644.0	644.0	0.00	771.10
20.300	1.00	2.0	646.0	646.0	0.00	771.10
20.400	1.00	2.0	648.0	648.0	0.00	771.11
20.500	1.00	2.0	650.0	650.0	0.00	771.11
20.600	1.00	2.0	652.0	652.0	0.00	771.12
20.700	1.00	2.0	654.0	654.0	0.00	771.12
20.800	1.00	2.0	656.0	656.0	0.00	771.13
20.900	1.00	2.0	658.0	658.0	0.00	771.13
21.000	1.00	2.0	660.0	660.0	0.00	771.14
21.100	1.00	2.0	662.0	662.0	0.00	771.14
21.200	1.00	2.0	664.0	664.0	0.00	771.15
21.300	1.00	2.0	666.0	666.0	0.00	771.15
21.400	1.00	2.0	668.0	668.0	0.00	771.15
21.500	1.00	2.0	670.0	670.0	0.00	771.16
21.600	1.00	2.0	672.0	672.0	0.00	771.16
21.700	1.00	2.0	674.0	674.0	0.00	771.17
21.800	1.00	2.0	676.0	676.0	0.00	771.17
21.900	1.00	2.0	678.0	678.0	0.00	771.18
22.000	1.00	2.0	680.0	680.0	0.00	771.18
22.100	1.00	2.0	682.0	682.0	0.00	771.19
22.200	1.00	2.0	684.0	684.0	0.00	771.19
22.300	1.00	2.0	686.0	686.0	0.00	771.20
22.400	1.00	2.0	688.0	688.0	0.00	771.20
22.500	1.00	2.0	690.0	690.0	0.00	771.21
22.600	1.00	2.0	692.0	692.0	0.00	771.21
22.700	1.00	2.0	694.0	694.0	0.00	771.22
22.800	1.00	2.0	696.0	696.0	0.00	771.22
22.900	1.00	2.0	698.0	698.0	0.00	771.23
23.000	1.00	2.0	700.0	700.0	0.00	771.23
23.100	1.00	2.0	702.0	702.0	0.00	771.24
23.200	1.00	2.0	704.0	704.0	0.00	771.24
23.300	1.00	2.0	706.0	706.0	0.00	771.25
23.400	1.00	2.0	708.0	708.0	0.00	771.25
23.500	1.00	2.0	710.0	710.0	0.00	771.26
23.600	1.00	2.0	712.0	712.0	0.00	771.26
23.700	1.00	2.0	714.0	714.0	0.00	771.27
23.800	1.00	2.0	716.0	716.0	0.00	771.27
23.900	1.00	2.0	718.0	718.0	0.00	771.27
24.000	0.00	1.0	719.0	719.0	0.00	771.28
24.100	0.00	0.0	719.0	719.0	0.00	771.28
24.200	0.00	0.0	719.0	719.0	0.00	771.28
24.300	0.00	0.0	719.0	719.0	0.00	771.28
24.400	0.00	0.0	719.0	719.0	0.00	771.28
24.500	0.00	0.0	719.0	719.0	0.00	771.28
24.600	0.00	0.0	719.0	719.0	0.00	771.28

nd File: A:POND4 .PND
flow Hydrograph: A:PIED4 .HYD
utflow Hydrograph: A:PNDOUT4 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
24.700	0.00
24.800	0.00
24.900	0.00
25.000	0.00
25.100	0.00
25.200	0.00
25.300	0.00
25.400	0.00
25.500	0.00
25.600	0.00
25.700	0.00
25.800	0.00
25.900	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28
0.0	719.0	719.0	0.00	771.28

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: A:POND4 .PND
Inflow Hydrograph: A:PIED4 .HYD
Outflow Hydrograph: A:PNDOUT4 .HYD

Starting Pond W.S. Elevation = 765.80 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	30.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	771.28 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	2.97 ac-ft
Total Storage in Pond	=	2.97 ac-ft

Warning: Inflow hydrograph truncated on left side.

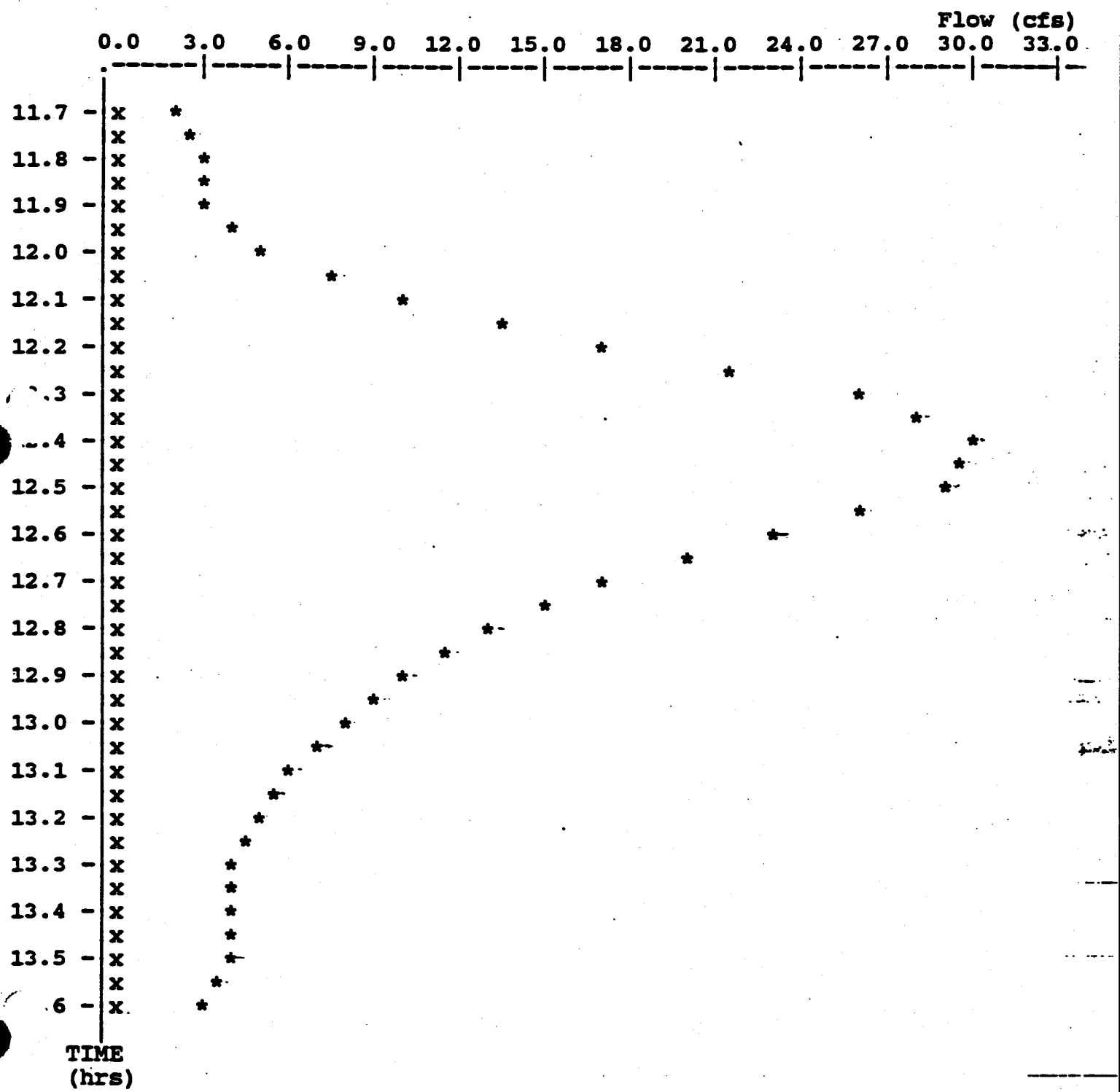
>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: A:POND4 .PND
Inflow Hydrograph: A:PIED4 .HYD
Outflow Hydrograph: A:PNDOUT4 .HYD

EXECUTED: 10-14-1993
10:50:39

Peak Inflow = 30.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 771.28 ft



```
*  File:  A:PIED4    .HYD   Qmax =    30.0 cfs
x  File:  A:PNDOUT4  .HYD   Qmax =     0.0 cfs
```

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:47:03
Watershed file: --> A:PIED312 .WSD
Hydrograph file: --> A:PIED312 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL2

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
312	6.73	80.0	0.30	0.00	8.40	6.00	.06 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 6.73 acres or 0.01052 sq.mi
Peak discharge = 43 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
312	0.34	0.00	0.30	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:47:03
Watershed file: --> A:PIED312 .WSD
Hydrograph file: --> A:PIED312 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL2

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
312	43	12.2
Composite Watershed	43	12.2

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:47:03
Watershed file: --> A:PIED312 .WSD
Hydrograph file: --> A:PIED312 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL2

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
312	1	2	3	7	15	28	43	43	29
Total (cfs)	1	2	3	7	15	28	43	43	29

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
312	18	12	9	7	5	4	4	3	3
Total (cfs)	18	12	9	7	5	4	4	3	3

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
312	3	2	2	2	2	2	1	1	1
Total (cfs)	3	2	2	2	2	2	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
312	1	1	1	1	0
Total (cfs)	1	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:47:03
Watershed file: --> A:PIED312 .WSD
Hydrograph file: --> A:PIED312 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL2

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	2
11.1	1	14.9	2
11.2	2	15.0	2
11.3	2	15.1	2
11.4	2	15.2	2
11.5	3	15.3	2
11.6	3	15.4	2
11.7	4	15.5	2
11.8	6	15.6	2
11.9	7	15.7	2
12.0	15	15.8	2
12.1	28	15.9	2
12.2	43	16.0	2
12.3	43	16.1	2
12.4	29	16.2	2
12.5	18	16.3	1
12.6	12	16.4	1
12.7	9	16.5	1
12.8	7	16.6	1
12.9	6	16.7	1
13.0	5	16.8	1
13.1	4	16.9	1
13.2	4	17.0	1
13.3	4	17.1	1
13.4	4	17.2	1
13.5	4	17.3	1
13.6	3	17.4	1
13.7	3	17.5	1
13.8	3	17.6	1
13.9	3	17.7	1
14.0	3	17.8	1
14.1	3	17.9	1
14.2	2	18.0	1
14.3	2	18.1	1
14.4	2	18.2	1
14.5	2	18.3	1
14.6	2	18.4	1

14.7	2	18.5	1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:47:03
Watershed file: --> A:PIED312 .WSD
Hydrograph file: --> A:PIED312 .HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL2

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* PIEDMONT LANDFILL AND RECYCLING CENTER *
* KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA *
* PHASE 3 - CELL 1 - SUBCELL 2 *
* STORAGE FOR 25 YR-24 HR STORM *
*
****

Inflow Hydrograph: A:PIED3 .HYD
Rating Table file: A:POND3 .PND

----INITIAL CONDITIONS----
Elevation = 765.00 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
765.00	0.0	0.000
766.00	0.0	0.040
767.00	0.0	0.170
768.00	0.0	0.470
769.00	0.0	0.990
770.00	0.0	1.850
771.00	0.0	3.080
772.00	0.0	4.880

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
9.7	9.7
41.1	41.1
113.7	113.7
239.6	239.6
447.7	447.7
745.4	745.4
1181.0	1181.0

Time increment (t) = 0.100 hrs.

nd File: A:POND3 .PND
.flow Hydrograph: A:PIED3 .HYD
Outflow Hydrograph: A:PNDOUT3 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
11.000	1.00
11.100	1.00
11.200	2.00
11.300	2.00
11.400	2.00
11.500	3.00
11.600	3.00
11.700	4.00
11.800	6.00
11.900	7.00
12.000	15.00
12.100	28.00
12.200	43.00
12.300	43.00
12.400	29.00
12.500	18.00
12.600	12.00
12.700	9.00
12.800	7.00
12.900	6.00
13.000	5.00
13.100	4.00
13.200	4.00
13.300	4.00
13.400	4.00
13.500	4.00
13.600	3.00
13.700	3.00
13.800	3.00
13.900	3.00
14.000	3.00
14.100	3.00
14.200	2.00
14.300	2.00
14.400	2.00
14.500	2.00
14.600	2.00
14.700	2.00
14.800	2.00
14.900	2.00
15.000	2.00
15.100	2.00
15.200	2.00
15.300	2.00
15.400	2.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
-----	0.0	0.0	0.00	765.00
2.0	2.0	2.0	0.00	765.21
3.0	5.0	5.0	0.00	765.52
4.0	9.0	9.0	0.00	765.93
4.0	13.0	13.0	0.00	766.11
5.0	18.0	18.0	0.00	766.26
6.0	24.0	24.0	0.00	766.46
7.0	31.0	31.0	0.00	766.68
10.0	41.0	41.0	0.00	767.00
13.0	54.0	54.0	0.00	767.18
22.0	76.0	76.0	0.00	767.48
43.0	119.0	119.0	0.00	768.04
71.0	190.0	190.0	0.00	768.61
86.0	276.0	276.0	0.00	769.17
72.0	348.0	348.0	0.00	769.52
47.0	395.0	395.0	0.00	769.75
30.0	425.0	425.0	0.00	769.89
21.0	446.0	446.0	0.00	769.99
16.0	462.0	462.0	0.00	770.05
13.0	475.0	475.0	0.00	770.09
11.0	486.0	486.0	0.00	770.13
9.0	495.0	495.0	0.00	770.16
8.0	503.0	503.0	0.00	770.19
8.0	511.0	511.0	0.00	770.21
8.0	519.0	519.0	0.00	770.24
8.0	527.0	527.0	0.00	770.27
7.0	534.0	534.0	0.00	770.29
6.0	540.0	540.0	0.00	770.31
6.0	546.0	546.0	0.00	770.33
6.0	552.0	552.0	0.00	770.35
6.0	558.0	558.0	0.00	770.37
6.0	564.0	564.0	0.00	770.39
5.0	569.0	569.0	0.00	770.41
4.0	573.0	573.0	0.00	770.42
4.0	577.0	577.0	0.00	770.43
4.0	581.0	581.0	0.00	770.45
4.0	585.0	585.0	0.00	770.46
4.0	589.0	589.0	0.00	770.47
4.0	593.0	593.0	0.00	770.49
4.0	597.0	597.0	0.00	770.50
4.0	601.0	601.0	0.00	770.52
4.0	605.0	605.0	0.00	770.53
4.0	609.0	609.0	0.00	770.54
4.0	613.0	613.0	0.00	770.56
4.0	617.0	617.0	0.00	770.57

nd File: A:POND3 .PND
nflow Hydrograph: A:PIED3 .HYD
Outflow Hydrograph: A:PNDOUT3 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
15.500	2.00
15.600	2.00
15.700	2.00
15.800	2.00
15.900	2.00
16.000	2.00
16.100	2.00
16.200	2.00
16.300	1.00
16.400	1.00
16.500	1.00
16.600	1.00
16.700	1.00
16.800	1.00
16.900	1.00
17.000	1.00
17.100	1.00
17.200	1.00
17.300	1.00
17.400	1.00
17.500	1.00
17.600	1.00
17.700	1.00
17.800	1.00
17.900	1.00
18.000	1.00
18.100	1.00
18.200	1.00
18.300	1.00
18.400	1.00
18.500	1.00
18.600	1.00
18.700	1.00
18.800	1.00
18.900	1.00
19.000	1.00
19.100	1.00
19.200	1.00
19.300	1.00
19.400	1.00
19.500	1.00
19.600	1.00
19.700	1.00
19.800	1.00
19.900	1.00
20.000	1.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
4.0	621.0	621.0	0.00	770.58
4.0	625.0	625.0	0.00	770.60
4.0	629.0	629.0	0.00	770.61
4.0	633.0	633.0	0.00	770.62
4.0	637.0	637.0	0.00	770.64
4.0	641.0	641.0	0.00	770.65
4.0	645.0	645.0	0.00	770.66
4.0	649.0	649.0	0.00	770.68
3.0	652.0	652.0	0.00	770.69
2.0	654.0	654.0	0.00	770.69
2.0	656.0	656.0	0.00	770.70
2.0	658.0	658.0	0.00	770.71
2.0	660.0	660.0	0.00	770.71
2.0	662.0	662.0	0.00	770.72
2.0	664.0	664.0	0.00	770.73
2.0	666.0	666.0	0.00	770.73
2.0	668.0	668.0	0.00	770.74
2.0	670.0	670.0	0.00	770.75
2.0	672.0	672.0	0.00	770.75
2.0	674.0	674.0	0.00	770.76
2.0	676.0	676.0	0.00	770.77
2.0	678.0	678.0	0.00	770.77
2.0	680.0	680.0	0.00	770.78
2.0	682.0	682.0	0.00	770.79
2.0	684.0	684.0	0.00	770.79
2.0	686.0	686.0	0.00	770.80
2.0	688.0	688.0	0.00	770.81
2.0	690.0	690.0	0.00	770.81
2.0	692.0	692.0	0.00	770.82
2.0	694.0	694.0	0.00	770.83
2.0	696.0	696.0	0.00	770.83
2.0	698.0	698.0	0.00	770.84
2.0	700.0	700.0	0.00	770.85
2.0	702.0	702.0	0.00	770.85
2.0	704.0	704.0	0.00	770.86
2.0	706.0	706.0	0.00	770.87
2.0	708.0	708.0	0.00	770.87
2.0	710.0	710.0	0.00	770.88
2.0	712.0	712.0	0.00	770.89
2.0	714.0	714.0	0.00	770.89
2.0	716.0	716.0	0.00	770.90
2.0	718.0	718.0	0.00	770.91
2.0	720.0	720.0	0.00	770.91
2.0	722.0	722.0	0.00	770.92
2.0	724.0	724.0	0.00	770.93
2.0	726.0	726.0	0.00	770.93

Pond File: A:POND3 .PND
Inflow Hydrograph: A:PIED3 .HYD
Outflow Hydrograph: A:PNDOUT3 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
20.100	1.00
20.200	1.00
20.300	1.00
20.400	1.00
20.500	1.00
20.600	1.00
20.700	1.00
20.800	1.00
20.900	1.00
21.000	1.00
21.100	1.00
21.200	1.00
21.300	1.00
21.400	1.00
21.500	1.00
21.600	1.00
21.700	1.00
21.800	1.00
21.900	1.00
22.000	1.00
22.100	1.00
22.200	1.00
22.300	1.00
22.400	1.00
22.500	1.00
22.600	1.00
22.700	1.00
22.800	1.00
22.900	1.00
23.000	1.00
23.100	1.00
23.200	1.00
23.300	1.00
23.400	1.00
23.500	1.00
23.600	1.00
23.700	1.00
23.800	1.00
23.900	1.00
24.000	0.00
24.100	0.00
24.200	0.00
24.300	0.00
24.400	0.00
24.500	0.00
24.600	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
2.0	728.0	728.0	0.00	770.94
2.0	730.0	730.0	0.00	770.95
2.0	732.0	732.0	0.00	770.96
2.0	734.0	734.0	0.00	770.96
2.0	736.0	736.0	0.00	770.97
2.0	738.0	738.0	0.00	770.98
2.0	740.0	740.0	0.00	770.98
2.0	742.0	742.0	0.00	770.99
2.0	744.0	744.0	0.00	771.00
2.0	746.0	746.0	0.00	771.00
2.0	748.0	748.0	0.00	771.01
2.0	750.0	750.0	0.00	771.01
2.0	752.0	752.0	0.00	771.02
2.0	754.0	754.0	0.00	771.02
2.0	756.0	756.0	0.00	771.02
2.0	758.0	758.0	0.00	771.03
2.0	760.0	760.0	0.00	771.03
2.0	762.0	762.0	0.00	771.04
2.0	764.0	764.0	0.00	771.04
2.0	766.0	766.0	0.00	771.05
2.0	768.0	768.0	0.00	771.05
2.0	770.0	770.0	0.00	771.06
2.0	772.0	772.0	0.00	771.06
2.0	774.0	774.0	0.00	771.07
2.0	776.0	776.0	0.00	771.07
2.0	778.0	778.0	0.00	771.07
2.0	780.0	780.0	0.00	771.08
2.0	782.0	782.0	0.00	771.08
2.0	784.0	784.0	0.00	771.09
2.0	786.0	786.0	0.00	771.09
2.0	788.0	788.0	0.00	771.10
2.0	790.0	790.0	0.00	771.10
2.0	792.0	792.0	0.00	771.11
2.0	794.0	794.0	0.00	771.11
2.0	796.0	796.0	0.00	771.12
2.0	798.0	798.0	0.00	771.12
2.0	800.0	800.0	0.00	771.13
2.0	802.0	802.0	0.00	771.13
2.0	804.0	804.0	0.00	771.13
1.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14

nd File: A:POND3 .PND
flow Hydrograph: A:PIED3 .HYD
Outflow Hydrograph: A:PNDOUT3 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
24.700	0.00
24.800	0.00
24.900	0.00
25.000	0.00
25.100	0.00
25.200	0.00
25.300	0.00
25.400	0.00
25.500	0.00
25.600	0.00
25.700	0.00
25.800	0.00
25.900	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14
0.0	805.0	805.0	0.00	771.14

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: A:POND3 .PND
Inflow Hydrograph: A:PIED3 .HYD
Outflow Hydrograph: A:PNDOUT3 .HYD

Starting Pond W.S. Elevation = 765.00 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	43.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	771.14 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	3.33 ac-ft
Total Storage in Pond	=	3.33 ac-ft

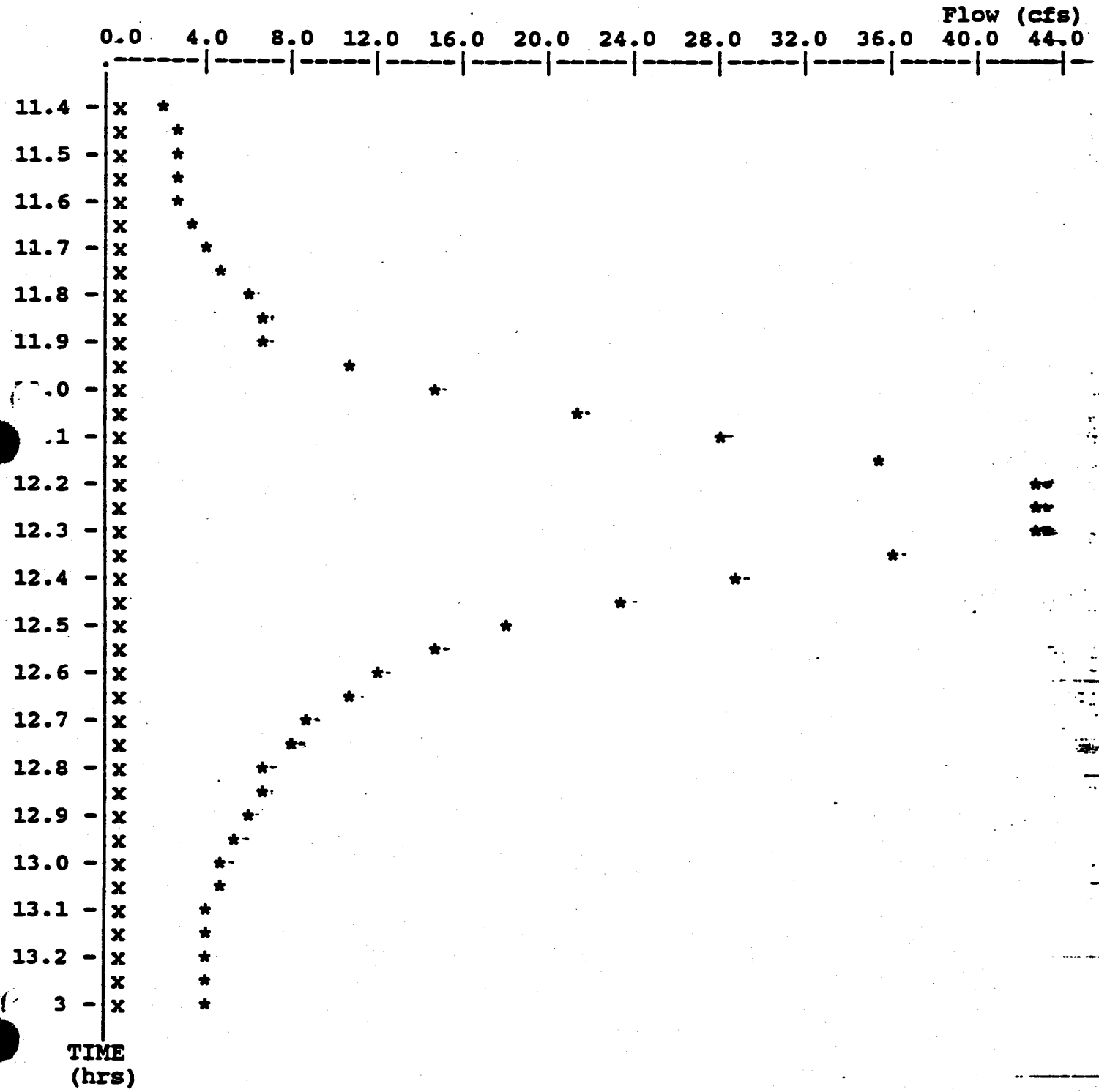
Warning: Inflow hydrograph truncated on left side.

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: A:POND3 .PND
Inflow Hydrograph: A:PIED3 .HYD
Outflow Hydrograph: A:PNDOUT3 .HYD

EXECUTED: 10-15-1993
15:40:11

Peak Inflow = 43.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 771.14 ft



```
*  File:  A:PIED3    .HYD   Qmax =    43.0 cfs
x  File:  A:PNDOUT3  .HYD   Qmax =     0.0 cfs
```

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:48:47
Watershed file: --> A:PIED313N.WSD
Hydrograph file: --> A:PIED313N.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 north

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
313n	4.48	80.0	0.40	0.00	8.40	6.00	.06 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 4.48 acres or 0.00700 sq.mi
Peak discharge = 25 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
313n	0.44	0.00	0.40	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:48:47
Watershed file: --> A:PIED313N.WSD
Hydrograph file: --> A:PIED313N.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 north

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
313n	25	12.3
Composite Watershed	25	12.3

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:48:47
Watershed file: --> A:PIED313N.WSD
Hydrograph file: --> A:PIED313N.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 north

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
313n	1	1	2	3	6	11	20	25	24
Total (cfs)	1	1	2	3	6	11	20	25	24

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
313n	18	13	9	7	4	3	3	2	2
Total (cfs)	18	13	9	7	4	3	3	2	2

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
313n	2	2	1	1	1	1	1	1	1
Total (cfs)	2	2	1	1	1	1	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
313n	1	1	1	1	0
Total (cfs)	1	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:48:47
Watershed file: --> A:PIED313N.WSD
Hydrograph file: --> A:PIED313N.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 north

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	1
11.1	1	14.9	1
11.2	1	15.0	1
11.3	1	15.1	1
11.4	1	15.2	1
11.5	2	15.3	1
11.6	2	15.4	1
11.7	2	15.5	1
11.8	3	15.6	1
11.9	3	15.7	1
12.0	6	15.8	1
12.1	11	15.9	1
12.2	20	16.0	1
12.3	25	16.1	1
12.4	24	16.2	1
12.5	18	16.3	1
12.6	13	16.4	1
12.7	9	16.5	1
12.8	7	16.6	1
12.9	5	16.7	1
13.0	4	16.8	1
13.1	3	16.9	1
13.2	3	17.0	1
13.3	3	17.1	1
13.4	3	17.2	1
13.5	3	17.3	1
13.6	2	17.4	1
13.7	2	17.5	1
13.8	2	17.6	1
13.9	2	17.7	1
14.0	2	17.8	1
14.1	2	17.9	1
14.2	2	18.0	1
14.3	2	18.1	1
14.4	2	18.2	1
14.5	1	18.3	1
14.6	1	18.4	1

14.7	1	18.5	1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:48:47
Watershed file: --> A:PIED313N.WSD
Hydrograph file: --> A:PIED313N.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 north

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* PIEDMONT LANDFILL AND RECYCLING CENTER *
* KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA *
* PHASE 3 - CELL 1 - SUBCELL 3 *
* STORAGE FOR 25 YR-24 HR STORM *
*
****

Inflow Hydrograph: A:PIED5 .HYD
Rating Table file: A:POND5 .PND

----INITIAL CONDITIONS----
Elevation = 768.50 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
768.50	0.0	0.000
769.00	0.0	0.030
769.50	0.0	0.090
770.50	0.0	0.330
771.50	0.0	0.940
772.50	0.0	1.870
773.50	0.0	3.350

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
7.3	7.3
21.8	21.8
79.9	79.9
227.5	227.5
452.5	452.5
810.7	810.7

Time increment (t) = 0.100 hrs.

Pond File: A:POND5 .PND
Inflow Hydrograph: A:PIED5 .HYD
Outflow Hydrograph: A:PNDOUT5 .HYD

INFLOW HYDROGRAPH

ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	1.00	-----	0.0	0.0	0.00	768.50
11.100	1.00	2.0	2.0	2.0	0.00	768.64
11.200	1.00	2.0	4.0	4.0	0.00	768.78
11.300	1.00	2.0	6.0	6.0	0.00	768.91
11.400	1.00	2.0	8.0	8.0	0.00	769.03
11.500	2.00	3.0	11.0	11.0	0.00	769.13
11.600	2.00	4.0	15.0	15.0	0.00	769.27
11.700	2.00	4.0	19.0	19.0	0.00	769.40
11.800	3.00	5.0	24.0	24.0	0.00	769.54
11.900	3.00	6.0	30.0	30.0	0.00	769.64
12.000	6.00	9.0	39.0	39.0	0.00	769.80
12.100	11.00	17.0	56.0	56.0	0.00	770.09
12.200	20.00	31.0	87.0	87.0	0.00	770.55
12.300	25.00	45.0	132.0	132.0	0.00	770.85
12.400	24.00	49.0	181.0	181.0	0.00	771.19
12.500	18.00	42.0	223.0	223.0	0.00	771.47
12.600	13.00	31.0	254.0	254.0	0.00	771.62
12.700	9.00	22.0	276.0	276.0	0.00	771.72
12.800	7.00	16.0	292.0	292.0	0.00	771.79
12.900	5.00	12.0	304.0	304.0	0.00	771.84
13.000	4.00	9.0	313.0	313.0	0.00	771.88
13.100	4.00	8.0	321.0	321.0	0.00	771.92
13.200	3.00	7.0	328.0	328.0	0.00	771.95
13.300	3.00	6.0	334.0	334.0	0.00	771.97
13.400	3.00	6.0	340.0	340.0	0.00	772.00
13.500	3.00	6.0	346.0	346.0	0.00	772.03
13.600	2.00	5.0	351.0	351.0	0.00	772.05
13.700	2.00	4.0	355.0	355.0	0.00	772.07
13.800	2.00	4.0	359.0	359.0	0.00	772.08
13.900	2.00	4.0	363.0	363.0	0.00	772.10
14.000	2.00	4.0	367.0	367.0	0.00	772.12
14.100	2.00	4.0	371.0	371.0	0.00	772.14
14.200	2.00	4.0	375.0	375.0	0.00	772.16
14.300	2.00	4.0	379.0	379.0	0.00	772.17
14.400	2.00	4.0	383.0	383.0	0.00	772.19
14.500	1.00	3.0	386.0	386.0	0.00	772.20
14.600	1.00	2.0	388.0	388.0	0.00	772.21
14.700	1.00	2.0	390.0	390.0	0.00	772.22
14.800	1.00	2.0	392.0	392.0	0.00	772.23
14.900	1.00	2.0	394.0	394.0	0.00	772.24
15.000	1.00	2.0	396.0	396.0	0.00	772.25
15.100	1.00	2.0	398.0	398.0	0.00	772.26
15.200	1.00	2.0	400.0	400.0	0.00	772.27
15.300	1.00	2.0	402.0	402.0	0.00	772.28
15.400	1.00	2.0	404.0	404.0	0.00	772.28

nd File: A:POND5 .PND
flow Hydrograph: A:PIED5 .HYD
Outflow Hydrograph: A:PNDOUT5 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	1.00	2.0	406.0	406.0	0.00	772.29
15.600	1.00	2.0	408.0	408.0	0.00	772.30
15.700	1.00	2.0	410.0	410.0	0.00	772.31
15.800	1.00	2.0	412.0	412.0	0.00	772.32
15.900	1.00	2.0	414.0	414.0	0.00	772.33
16.000	1.00	2.0	416.0	416.0	0.00	772.34
16.100	1.00	2.0	418.0	418.0	0.00	772.35
16.200	1.00	2.0	420.0	420.0	0.00	772.36
16.300	1.00	2.0	422.0	422.0	0.00	772.36
16.400	1.00	2.0	424.0	424.0	0.00	772.37
16.500	1.00	2.0	426.0	426.0	0.00	772.38
16.600	1.00	2.0	428.0	428.0	0.00	772.39
16.700	1.00	2.0	430.0	430.0	0.00	772.40
16.800	1.00	2.0	432.0	432.0	0.00	772.41
16.900	1.00	2.0	434.0	434.0	0.00	772.42
17.000	1.00	2.0	436.0	436.0	0.00	772.43
17.100	1.00	2.0	438.0	438.0	0.00	772.44
17.200	1.00	2.0	440.0	440.0	0.00	772.44
17.300	1.00	2.0	442.0	442.0	0.00	772.45
17.400	1.00	2.0	444.0	444.0	0.00	772.46
17.500	1.00	2.0	446.0	446.0	0.00	772.47
17.600	1.00	2.0	448.0	448.0	0.00	772.48
17.700	1.00	2.0	450.0	450.0	0.00	772.49
17.800	1.00	2.0	452.0	452.0	0.00	772.50
17.900	1.00	2.0	454.0	454.0	0.00	772.50
18.000	1.00	2.0	456.0	456.0	0.00	772.51
18.100	1.00	2.0	458.0	458.0	0.00	772.52
18.200	1.00	2.0	460.0	460.0	0.00	772.52
18.300	1.00	2.0	462.0	462.0	0.00	772.53
18.400	1.00	2.0	464.0	464.0	0.00	772.53
18.500	1.00	2.0	466.0	466.0	0.00	772.54
18.600	1.00	2.0	468.0	468.0	0.00	772.54
18.700	1.00	2.0	470.0	470.0	0.00	772.55
18.800	1.00	2.0	472.0	472.0	0.00	772.55
18.900	1.00	2.0	474.0	474.0	0.00	772.56
19.000	1.00	2.0	476.0	476.0	0.00	772.57
19.100	1.00	2.0	478.0	478.0	0.00	772.57
19.200	1.00	2.0	480.0	480.0	0.00	772.58
19.300	1.00	2.0	482.0	482.0	0.00	772.58
19.400	1.00	2.0	484.0	484.0	0.00	772.59
19.500	1.00	2.0	486.0	486.0	0.00	772.59
19.600	1.00	2.0	488.0	488.0	0.00	772.60
19.700	1.00	2.0	490.0	490.0	0.00	772.60
19.800	1.00	2.0	492.0	492.0	0.00	772.61
19.900	1.00	2.0	494.0	494.0	0.00	772.62
20.000	1.00	2.0	496.0	496.0	0.00	772.62

Pond File: A:POND5 .PND
Inflow Hydrograph: A:PIED5 .HYD
Outflow Hydrograph: A:PNDOUT5 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
20.100	1.00
20.200	1.00
20.300	1.00
20.400	1.00
20.500	1.00
20.600	1.00
20.700	1.00
20.800	1.00
20.900	1.00
21.000	1.00
21.100	1.00
21.200	1.00
21.300	1.00
21.400	1.00
21.500	1.00
21.600	1.00
21.700	1.00
21.800	1.00
21.900	1.00
22.000	1.00
22.100	1.00
22.200	1.00
22.300	1.00
22.400	1.00
22.500	1.00
22.600	1.00
22.700	1.00
22.800	1.00
22.900	1.00
23.000	1.00
23.100	1.00
23.200	1.00
23.300	1.00
23.400	1.00
23.500	1.00
23.600	1.00
23.700	1.00
23.800	1.00
23.900	1.00
24.000	0.00
24.100	0.00
24.200	0.00
24.300	0.00
24.400	0.00
24.500	0.00
24.600	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
2.0	498.0	498.0	0.00	772.63
2.0	500.0	500.0	0.00	772.63
2.0	502.0	502.0	0.00	772.64
2.0	504.0	504.0	0.00	772.64
2.0	506.0	506.0	0.00	772.65
2.0	508.0	508.0	0.00	772.65
2.0	510.0	510.0	0.00	772.66
2.0	512.0	512.0	0.00	772.67
2.0	514.0	514.0	0.00	772.67
2.0	516.0	516.0	0.00	772.68
2.0	518.0	518.0	0.00	772.68
2.0	520.0	520.0	0.00	772.69
2.0	522.0	522.0	0.00	772.69
2.0	524.0	524.0	0.00	772.70
2.0	526.0	526.0	0.00	772.71
2.0	528.0	528.0	0.00	772.71
2.0	530.0	530.0	0.00	772.72
2.0	532.0	532.0	0.00	772.72
2.0	534.0	534.0	0.00	772.73
2.0	536.0	536.0	0.00	772.73
2.0	538.0	538.0	0.00	772.74
2.0	540.0	540.0	0.00	772.74
2.0	542.0	542.0	0.00	772.75
2.0	544.0	544.0	0.00	772.76
2.0	546.0	546.0	0.00	772.76
2.0	548.0	548.0	0.00	772.77
2.0	550.0	550.0	0.00	772.77
2.0	552.0	552.0	0.00	772.78
2.0	554.0	554.0	0.00	772.78
2.0	556.0	556.0	0.00	772.79
2.0	558.0	558.0	0.00	772.79
2.0	560.0	560.0	0.00	772.80
2.0	562.0	562.0	0.00	772.81
2.0	564.0	564.0	0.00	772.81
2.0	566.0	566.0	0.00	772.82
2.0	568.0	568.0	0.00	772.82
2.0	570.0	570.0	0.00	772.83
2.0	572.0	572.0	0.00	772.83
2.0	574.0	574.0	0.00	772.84
1.0	575.0	575.0	0.00	772.84
0.0	575.0	575.0	0.00	772.84
0.0	575.0	575.0	0.00	772.84
0.0	575.0	575.0	0.00	772.84
0.0	575.0	575.0	0.00	772.84
0.0	575.0	575.0	0.00	772.84
0.0	575.0	575.0	0.00	772.84

nd File: A:POND5 .PND
flow Hydrograph: A:PIED5 .HYD
Outflow Hydrograph: A:PNDOUT5 .HYD

INFLOW HYDROGRAPH

ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
24.700	0.00	0.0	575.0	575.0	0.00	772.84
24.800	0.00	0.0	575.0	575.0	0.00	772.84
24.900	0.00	0.0	575.0	575.0	0.00	772.84
25.000	0.00	0.0	575.0	575.0	0.00	772.84
25.100	0.00	0.0	575.0	575.0	0.00	772.84
25.200	0.00	0.0	575.0	575.0	0.00	772.84
25.300	0.00	0.0	575.0	575.0	0.00	772.84
25.400	0.00	0.0	575.0	575.0	0.00	772.84
25.500	0.00	0.0	575.0	575.0	0.00	772.84
25.600	0.00	0.0	575.0	575.0	0.00	772.84
25.700	0.00	0.0	575.0	575.0	0.00	772.84
25.800	0.00	0.0	575.0	575.0	0.00	772.84
25.900	0.00	0.0	575.0	575.0	0.00	772.84

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: A:POND5 .PND
Inflow Hydrograph: A:PIED5 .HYD
Outflow Hydrograph: A:PNDOUT5 .HYD

Starting Pond W.S. Elevation = 768.50 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	25.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	772.84 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	2.38 ac-ft
Total Storage in Pond	=	2.38 ac-ft

Warning: Inflow hydrograph truncated on left side.

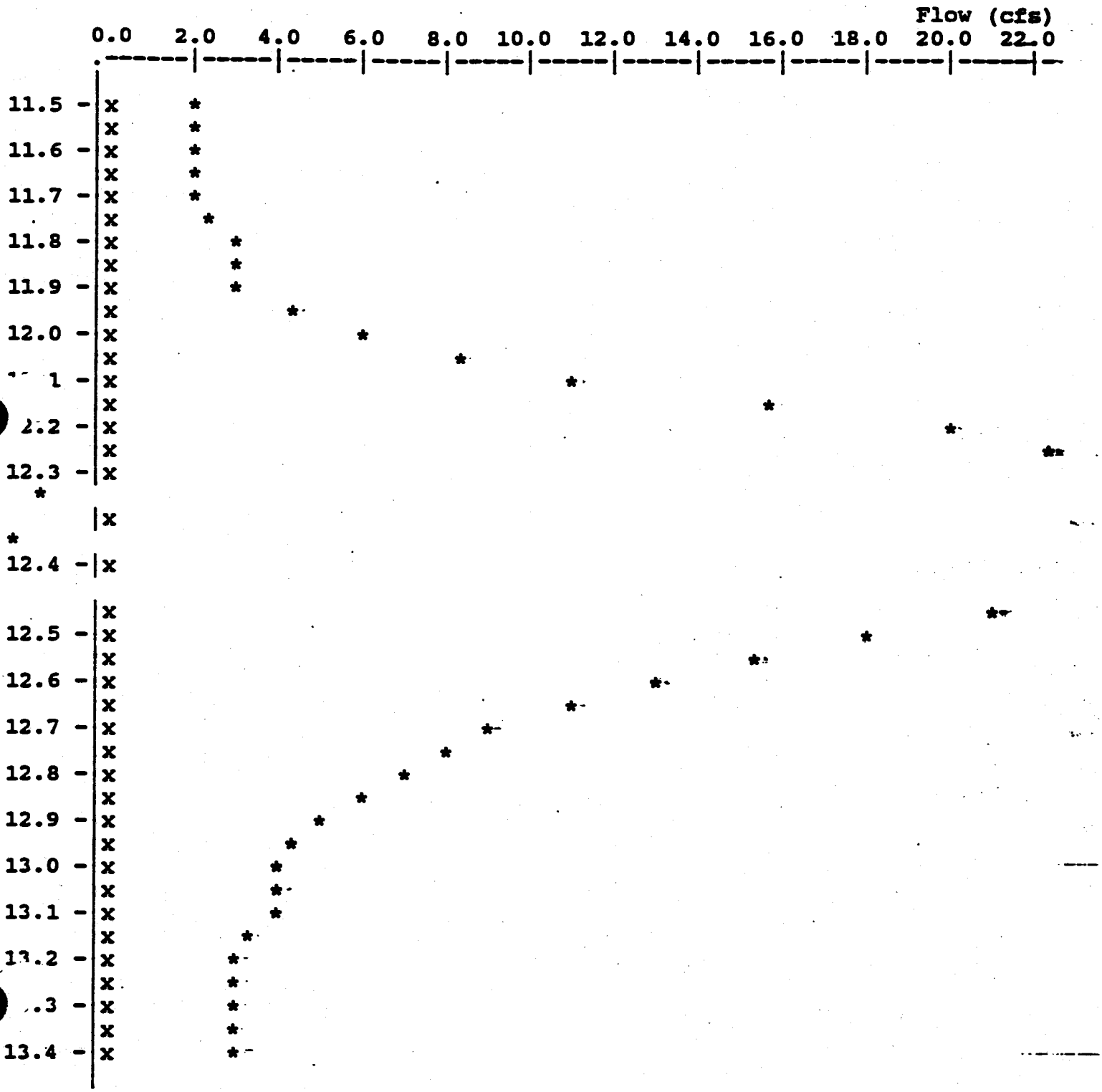
>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: A:POND5 .PND
Inflow Hydrograph: A:PIED5 .HYD
Outflow Hydrograph: A:PNDOUT5 .HYD

EXECUTED: 10-15-1993
15:42:45

Peak Inflow = 25.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 772.84 ft



TIME
(hrs)

* File: A:PIED5 .HYD Qmax = 25.0 cfs
File: A:PNDOUT5 .HYD Qmax = 0.0 cfs

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:50:07
Watershed file: --> A:PIED313S.WSD
Hydrograph file: --> A:PIED313S.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 south

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
313s	2.82	80.0	0.30	0.00	8.40	6.00	.06 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 2.82 acres or 0.00441 sq.mi
Peak discharge = 18 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
313s	0.34	0.00	0.30	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:50:07
Watershed file: --> A:PIED313S.WSD
Hydrograph file: --> A:PIED313S.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 south

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
313s	18	12.2
Composite Watershed	18	12.2

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:50:07
Watershed file: --> A:PIED313S.WSD
Hydrograph file: --> A:PIED313S.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 south

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
313s	1	1	1	3	6	12	18	18	12
Total (cfs)	1	1	1	3	6	12	18	18	12

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
313s	7	5	4	3	2	2	2	1	1
Total (cfs)	7	5	4	3	2	2	2	1	1

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
313s	1	1	1	1	1	1	1	1	1
Total (cfs)	1	1	1	1	1	1	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
313s	0	0	0	0	0
Total (cfs)	0	0	0	0	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:50:07
Watershed file: --> A:PIED313S.WSD
Hydrograph file: --> A:PIED313S.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 south

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	1
11.1	1	14.9	1
11.2	1	15.0	1
11.3	1	15.1	1
11.4	1	15.2	1
11.5	1	15.3	1
11.6	1	15.4	1
11.7	2	15.5	1
11.8	2	15.6	1
11.9	3	15.7	1
12.0	6	15.8	1
12.1	12	15.9	1
12.2	18	16.0	1
12.3	18	16.1	1
12.4	12	16.2	1
12.5	7	16.3	1
12.6	5	16.4	1
12.7	4	16.5	1
12.8	3	16.6	1
12.9	2	16.7	1
13.0	2	16.8	1
13.1	2	16.9	1
13.2	2	17.0	1
13.3	2	17.1	1
13.4	2	17.2	1
13.5	2	17.3	1
13.6	1	17.4	1
13.7	1	17.5	1
13.8	1	17.6	1
13.9	1	17.7	1
14.0	1	17.8	0
14.1	1	17.9	0
14.2	1	18.0	0
14.3	1	18.1	0
14.4	1	18.2	0
14.5	1	18.3	0
14.6	1	18.4	0

14.7	1	18.5	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-13-1994 09:50:07
Watershed file: --> A:PIED313S.WSD
Hydrograph file: --> A:PIED313S.HYD

PIEDMONT LRC
LEACHATE CONTAINMENT CALCULATIONS
CHECK OF 10-15-93 CALCULATIONS USING 100YEAR/24HOUR STORM
PHASE3-CELL1-SUBCELL3 south

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	0	22.4	0
18.7	0	22.5	0
18.8	0	22.6	0
18.9	0	22.7	0
19.0	0	22.8	0
19.1	0	22.9	0
19.2	0	23.0	0
19.3	0	23.1	0
19.4	0	23.2	0
19.5	0	23.3	0
19.6	0	23.4	0
19.7	0	23.5	0
19.8	0	23.6	0
19.9	0	23.7	0
20.0	0	23.8	0
20.1	0	23.9	0
20.2	0	24.0	0
20.3	0	24.1	0
20.4	0	24.2	0
20.5	0	24.3	0
20.6	0	24.4	0
20.7	0	24.5	0
20.8	0	24.6	0
20.9	0	24.7	0
21.0	0	24.8	0
21.1	0	24.9	0
21.2	0	25.0	0
21.3	0	25.1	0
21.4	0	25.2	0
21.5	0	25.3	0
21.6	0	25.4	0
21.7	0	25.5	0
21.8	0	25.6	0
21.9	0	25.7	0
22.0	0	25.8	0
22.1	0	25.9	0
22.2	0		
22.3	0		

****
*
* PIEDMONT LANDFILL AND RECYCLING CENTER *
* KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA *
* PHASE 3 - CELL 1 - SUBCELL 3 (SOUTHERN PORTION) *
* 25 YR-24 HR STORM STORAGE *
*
****

Inflow Hydrograph: A:PIED6 .HYD
Rating Table file: A:POND6 .PND

----INITIAL CONDITIONS----
Elevation = 774.00 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
774.00	0.0	0.000
776.00	0.0	0.050
778.00	0.0	0.350
780.00	0.0	1.340
782.00	0.0	2.600

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
12.1	12.1
84.7	84.7
324.3	324.3
629.2	629.2

Time increment (t) = 0.100 hrs.

nd File: A:POND6 .PND
.flow Hydrograph: A:PIED6 .HYD
Outflow Hydrograph: A:PND6OUT .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
11.000	1.00
11.100	1.00
11.200	1.00
11.300	1.00
11.400	1.00
11.500	1.00
11.600	1.00
11.700	2.00
11.800	2.00
11.900	3.00
12.000	6.00
12.100	12.00
12.200	18.00
12.300	18.00
12.400	12.00
12.500	7.00
12.600	5.00
2.700	4.00
12.800	3.00
12.900	2.00
13.000	2.00
13.100	2.00
13.200	2.00
13.300	2.00
13.400	2.00
13.500	2.00
13.600	1.00
13.700	1.00
13.800	1.00
13.900	1.00
14.000	1.00
14.100	1.00
14.200	1.00
14.300	1.00
14.400	1.00
14.500	1.00
14.600	1.00
14.700	1.00
14.800	1.00
14.900	1.00
15.000	1.00
15.100	1.00
15.200	1.00
15.300	1.00
15.400	1.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
-----	0.0	0.0	0.00	774.00
2.0	2.0	2.0	0.00	774.33
2.0	4.0	4.0	0.00	774.66
2.0	6.0	6.0	0.00	774.99
2.0	8.0	8.0	0.00	775.32
2.0	10.0	10.0	0.00	775.65
2.0	12.0	12.0	0.00	775.98
3.0	15.0	15.0	0.00	776.08
4.0	19.0	19.0	0.00	776.19
5.0	24.0	24.0	0.00	776.33
9.0	33.0	33.0	0.00	776.58
18.0	51.0	51.0	0.00	777.07
30.0	81.0	81.0	0.00	777.90
36.0	117.0	117.0	0.00	778.27
30.0	147.0	147.0	0.00	778.52
19.0	166.0	166.0	0.00	778.68
12.0	178.0	178.0	0.00	778.78
9.0	187.0	187.0	0.00	778.85
7.0	194.0	194.0	0.00	778.91
5.0	199.0	199.0	0.00	778.95
4.0	203.0	203.0	0.00	778.99
4.0	207.0	207.0	0.00	779.02
4.0	211.0	211.0	0.00	779.05
4.0	215.0	215.0	0.00	779.09
4.0	219.0	219.0	0.00	779.12
4.0	223.0	223.0	0.00	779.15
3.0	226.0	226.0	0.00	779.18
2.0	228.0	228.0	0.00	779.20
2.0	230.0	230.0	0.00	779.21
2.0	232.0	232.0	0.00	779.23
2.0	234.0	234.0	0.00	779.25
2.0	236.0	236.0	0.00	779.26
2.0	238.0	238.0	0.00	779.28
2.0	240.0	240.0	0.00	779.30
2.0	242.0	242.0	0.00	779.31
2.0	244.0	244.0	0.00	779.33
2.0	246.0	246.0	0.00	779.35
2.0	248.0	248.0	0.00	779.36
2.0	250.0	250.0	0.00	779.38
2.0	252.0	252.0	0.00	779.40
2.0	254.0	254.0	0.00	779.41
2.0	256.0	256.0	0.00	779.43
2.0	258.0	258.0	0.00	779.45
2.0	260.0	260.0	0.00	779.46
2.0	262.0	262.0	0.00	779.48

EXECUTED: 10-18-1993 15:46:20

nd File: A:POND6 .PND
flow Hydrograph: A:PIED6 .HYD
Outflow Hydrograph: A:PND6OUT .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	1.00	2.0	264.0	264.0	0.00	779.50
15.600	1.00	2.0	266.0	266.0	0.00	779.51
15.700	1.00	2.0	268.0	268.0	0.00	779.53
15.800	1.00	2.0	270.0	270.0	0.00	779.55
15.900	1.00	2.0	272.0	272.0	0.00	779.56
16.000	1.00	2.0	274.0	274.0	0.00	779.58
16.100	1.00	2.0	276.0	276.0	0.00	779.60
16.200	1.00	2.0	278.0	278.0	0.00	779.61
16.300	1.00	2.0	280.0	280.0	0.00	779.63
16.400	1.00	2.0	282.0	282.0	0.00	779.65
16.500	1.00	2.0	284.0	284.0	0.00	779.66
16.600	1.00	2.0	286.0	286.0	0.00	779.68
16.700	1.00	2.0	288.0	288.0	0.00	779.70
16.800	1.00	2.0	290.0	290.0	0.00	779.71
16.900	1.00	2.0	292.0	292.0	0.00	779.73
17.000	1.00	2.0	294.0	294.0	0.00	779.75
17.100	1.00	2.0	296.0	296.0	0.00	779.76
17.200	1.00	2.0	298.0	298.0	0.00	779.78
17.300	1.00	2.0	300.0	300.0	0.00	779.80
17.400	1.00	2.0	302.0	302.0	0.00	779.81
17.500	1.00	2.0	304.0	304.0	0.00	779.83
17.600	1.00	2.0	306.0	306.0	0.00	779.85
17.700	1.00	2.0	308.0	308.0	0.00	779.86
17.800	0.00	1.0	309.0	309.0	0.00	779.87
17.900	0.00	0.0	309.0	309.0	0.00	779.87
18.000	0.00	0.0	309.0	309.0	0.00	779.87
18.100	0.00	0.0	309.0	309.0	0.00	779.87
18.200	0.00	0.0	309.0	309.0	0.00	779.87
18.300	0.00	0.0	309.0	309.0	0.00	779.87
18.400	0.00	0.0	309.0	309.0	0.00	779.87
18.500	0.00	0.0	309.0	309.0	0.00	779.87
18.600	0.00	0.0	309.0	309.0	0.00	779.87
18.700	0.00	0.0	309.0	309.0	0.00	779.87
18.800	0.00	0.0	309.0	309.0	0.00	779.87
18.900	0.00	0.0	309.0	309.0	0.00	779.87
19.000	0.00	0.0	309.0	309.0	0.00	779.87
19.100	0.00	0.0	309.0	309.0	0.00	779.87
19.200	0.00	0.0	309.0	309.0	0.00	779.87
19.300	0.00	0.0	309.0	309.0	0.00	779.87
19.400	0.00	0.0	309.0	309.0	0.00	779.87
19.500	0.00	0.0	309.0	309.0	0.00	779.87
19.600	0.00	0.0	309.0	309.0	0.00	779.87
19.700	0.00	0.0	309.0	309.0	0.00	779.87
19.800	0.00	0.0	309.0	309.0	0.00	779.87
19.900	0.00	0.0	309.0	309.0	0.00	779.87
20.000	0.00	0.0	309.0	309.0	0.00	779.87

Id File: A:POND6 .PND
flow Hydrograph: A:PIED6 .HYD
Outflow Hydrograph: A:PND6OUT .HYD

INFLOW HYDROGRAPH

ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	0.00	0.0	309.0	309.0	0.00	779.87
20.200	0.00	0.0	309.0	309.0	0.00	779.87
20.300	0.00	0.0	309.0	309.0	0.00	779.87
20.400	0.00	0.0	309.0	309.0	0.00	779.87
20.500	0.00	0.0	309.0	309.0	0.00	779.87
20.600	0.00	0.0	309.0	309.0	0.00	779.87
20.700	0.00	0.0	309.0	309.0	0.00	779.87
20.800	0.00	0.0	309.0	309.0	0.00	779.87
20.900	0.00	0.0	309.0	309.0	0.00	779.87
21.000	0.00	0.0	309.0	309.0	0.00	779.87
21.100	0.00	0.0	309.0	309.0	0.00	779.87
21.200	0.00	0.0	309.0	309.0	0.00	779.87
21.300	0.00	0.0	309.0	309.0	0.00	779.87
21.400	0.00	0.0	309.0	309.0	0.00	779.87
21.500	0.00	0.0	309.0	309.0	0.00	779.87
21.600	0.00	0.0	309.0	309.0	0.00	779.87
21.700	0.00	0.0	309.0	309.0	0.00	779.87
21.800	0.00	0.0	309.0	309.0	0.00	779.87
21.900	0.00	0.0	309.0	309.0	0.00	779.87
22.000	0.00	0.0	309.0	309.0	0.00	779.87
22.100	0.00	0.0	309.0	309.0	0.00	779.87
22.200	0.00	0.0	309.0	309.0	0.00	779.87
22.300	0.00	0.0	309.0	309.0	0.00	779.87
22.400	0.00	0.0	309.0	309.0	0.00	779.87
22.500	0.00	0.0	309.0	309.0	0.00	779.87
22.600	0.00	0.0	309.0	309.0	0.00	779.87
22.700	0.00	0.0	309.0	309.0	0.00	779.87
22.800	0.00	0.0	309.0	309.0	0.00	779.87
22.900	0.00	0.0	309.0	309.0	0.00	779.87
23.000	0.00	0.0	309.0	309.0	0.00	779.87
23.100	0.00	0.0	309.0	309.0	0.00	779.87
23.200	0.00	0.0	309.0	309.0	0.00	779.87
23.300	0.00	0.0	309.0	309.0	0.00	779.87
23.400	0.00	0.0	309.0	309.0	0.00	779.87
23.500	0.00	0.0	309.0	309.0	0.00	779.87
23.600	0.00	0.0	309.0	309.0	0.00	779.87
23.700	0.00	0.0	309.0	309.0	0.00	779.87
23.800	0.00	0.0	309.0	309.0	0.00	779.87
23.900	0.00	0.0	309.0	309.0	0.00	779.87
24.000	0.00	0.0	309.0	309.0	0.00	779.87
24.100	0.00	0.0	309.0	309.0	0.00	779.87
24.200	0.00	0.0	309.0	309.0	0.00	779.87
24.300	0.00	0.0	309.0	309.0	0.00	779.87
24.400	0.00	0.0	309.0	309.0	0.00	779.87
24.500	0.00	0.0	309.0	309.0	0.00	779.87
24.600	0.00	0.0	309.0	309.0	0.00	779.87

nd File: A:POND6 .PND
..flow Hydrograph: A:PIED6 .HYD
Outflow Hydrograph: A:PND6OUT .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
24.700	0.00
24.800	0.00
24.900	0.00
25.000	0.00
25.100	0.00
25.200	0.00
25.300	0.00
25.400	0.00
25.500	0.00
25.600	0.00
25.700	0.00
25.800	0.00
25.900	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87
0.0	309.0	309.0	0.00	779.87

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: A:POND6 .PND
Inflow Hydrograph: A:PIED6 .HYD
Outflow Hydrograph: A:PND6OUT .HYD

Starting Pond W.S. Elevation = 774.00 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	18.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	779.87 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	1.28 ac-ft
Total Storage in Pond	=	1.28 ac-ft

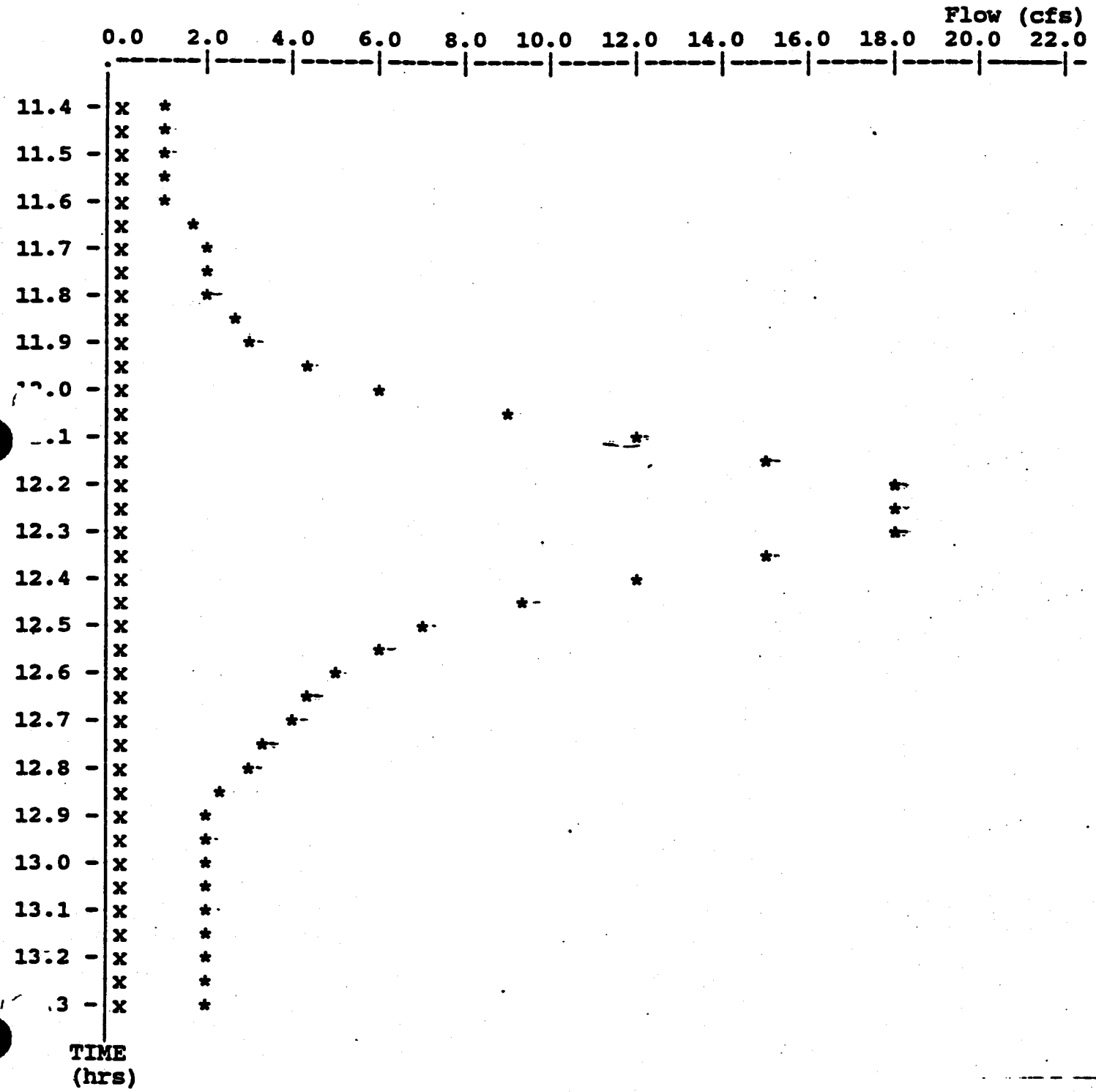
Warning: Inflow hydrograph truncated on left side.

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: A:POND6 .PND
Inflow Hydrograph: A:PIED6 .HYD
Outflow Hydrograph: A:PND6OUT .HYD

EXECUTED: 10-18-1993
15:46:20

Peak Inflow = 18.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 779.87 ft



```
 *  File:  A:PIED6    .HYD   Qmax =    18.0 cfs
 x  File:  A:PND6OUT  .HYD   Qmax =     0.0 cfs
```

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1A OF ____
PROJECT NO. 87221

CLIENT Piedmont LRC
PROJECT Permit Renewal.

SUBJECT Leachate Containment Calculations

Prepared By AJM Date 4/13/94
Reviewed By JM Date 4/13/94
Approved By ____ Date ____

Purpose: To revise the containment calculations pursuant to revisions to the Piedmont LRC base grading design.

Givens & Assumed:

- The revised calculations will follow the same methodology as the original calcs. (prepared 10-15-93)

- Based on the redesign, Phase 3 - Cell 1 - Subcells 1 and 2 have not changed; so they will not be reevaluated. Phase 3 - Cell 1 - Subcell 3 and Phase 3 - Cell 2 - Subcells 1 + 2 will be reevaluated based on the new design.

* • In revising these calculations, an error was discovered in the 10-15-93 calculations. Although the hydrograph input data should have used the 25-year / 24-hour design storm (6.0 inches), the hydrographs were based on the 100year / 24-hour storm (8.4 inches). The hydrograph output data is now included in the original calculations section.

- Because the error was discovered, Phase 3 - Cell 1 - Subcell 3 and Phase 3 - Cell 2 - Subcells 1 and 2 will be revised. However, because Phase 3 - Cell 1 - Subcells 1 and 2 did not change and were able to contain the 100-year/24-hour storm, they will not be reevaluted.

- The following data presents the revised calculations for the aforementioned three subcells @ the 25-year / 24 hour storm of 6.0".

Conclusion:

∴ Based on the output data from the routing of the unit hydrographs, the three subcells are each capable of containing the 25-year / 24-hour storm without overtopping. The use of pumps is not necessary to maintain containment.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 3 OF ____
PROJECT NO. 97221.400

CLIENT WMCI
PROJECT PIEDMONT LRC

SUBJECT CELL STORAGE QUANTITY & PUMP CAPACITY CALCULATIONS

Prepared By JM Date 4/6/94
Reviewed By ATM Date 4/13/94
Approved By ____ Date ____

CALCULATIONS:

PHASE 3, CELL 1 SUBCELL 3

ELEV. (FT)	POND AREA (AC)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT) V_2	VOID VOLUME (AC-FT) V_3	TOTAL VOLUME (AC-FT)
763.8	0	0	0	0	0
764	0.009	0	0	.0004	.0004
766	0.30	.0009	.0072	.1164	0.1245
768	0.83	0.33	0.24	0.212	0.782
770	1.53	1.743	.664	0.28	2.687
772	2.34	4.743	1.224	0.324	6.291
773	2.55	6.966	1.548	0.246	8.76

CELL WILL NOT OVERTOP AS LONG AS THE BERMS ARE A MINIMUM OF 773.0 FT MSL. CREST WILL OCCUR AT 769.71.

PHASE 3, CELL 2, SUBCELL 1

ELEV. (FT)	POND AREA (AC)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT) V_2	VOID VOLUME (AC-FT) V_3	TOTAL VOLUME (AC-FT)
757	0	0	0	0	0
758	0.14	0	0	0.028	0.028
760	0.78	0.07	0.112	0.256	0.438
762	1.82	1.17	.624	.416	2.21
762.9	2.49	2.434	.998	.497	3.929

CELL WILL NOT OVERTOP AS LONG AS BERMS ARE A MINIMUM OF 762.9 FT (MSL). POND WILL CREST AT AN ELEVATION OF 762.47

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 4 OF ____

PROJECT NO. 97221.400

CLIENT WMCI

SUBJECT CELL STORAGE QUANTITY & PUMP CAPACITY CALCULATIONS

Prepared By JRM Date 4/6/94

PROJECT PIEDMONT LRC

Reviewed By CJM Date 4/13/94

Approved By ____ Date ____

CALCULATIONS:

PHASE 3, CELL 2 SUBCELL 2

ELEV. (FT)	POND AREA (AC)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT) V_2	VOID VOLUME (AC-FT) V_3	TOTAL VOLUME (AC-FT)
770.2	0	0	0	0	0
772	0.23	0	0	.092	0.092
774	1.00	0.207	0.184	.308	0.699
776	2.12	1.90	0.80	.448	3.148

CELL WILL NOT OVERTOP IF THE BERMS ARE A MINIMUM OF 776.0 FT (MSL). CREST WILL OCCUR AT 775.79.

PHASE 3, CELL ___, SUBCELL ___

ELEV. (FT)	POND AREA (AC)	AIRSPACE VOLUME (AC-FT)	VOID VOLUME (AC-FT) V_2	VOID VOLUME (AC-FT) V_3	TOTAL VOLUME (AC-FT)

TR-55 GRAPHICAL METHOD

CURVE NUMBER (CN)= 80
RAINFALL DISTRIBUTION TYPE= II
MANNING'S NUMBER FOR SHEET FLOW: n= 0.20
(SEE TABLE 3-1)
INITIAL ABSTRACTION: Ia=
(SEE TABLE 4-1)

Notes: PIEDMONT LRC
CELL STORAGE
QUANTITY AND PUMP
CAPACITY CALCULATIONS

DESIGN STORM: 25yr-24HR
RAINFALL DEPTH= =P
RUNOFF DEPTH= 0 =Q
2yr-24hr STORM DEPTH= 3.5IN =P2
Ia/P=

Sub-area Segment	Area sq. mi.	CONCENTRATION TIME TO SUBAREA OUTFALL: SHEET FLOW Length L1 ft	Slope S1 ft/ft	Time(1) hours	SHALLOW CONC. FLOW Length L2 ft	Slope S2 ft/ft	Velocity V2 fps	Time(2) hours	TRAVEL TIME: CHANNELIZED FLOW Length L3 ft	Slope S3 ft/ft	Velocity V3 fps	Time(3) hours	Tc hours	Tt hours	Total Time T hours	Unit PeaK Disch. cms/in	Peak Disch. Qp cfs	POND-PACK FILE NAMES
PHASE 3, CELL 1 & 3	7.30	250	0.022	0.394	120	0.05	3.65	0.009					0.481					PIED1
		50	0.05	0.078														
PHASE 3, CELL 2 & 1	9.94	300	0.022	0.456	200	0.022	2.40	0.023					0.479					PIED 2
PHASE 3, CELL 2 & 2	9.27	300	0.022	0.456	310	0.022	2.40	0.036					0.492					PIED3

L1<=300ft
V2 FOR SHALLOW CONC. FLOW (FIGURE 3-1)
Tc=t1+t2
Tt=t3 FROM SUBAREA OUTFALL TO WATERSHED OUTFALL

$V3 = \frac{1.49(R)^{0.667}(S)^{0.5}}{n}$

MANNING'S n FOR CHANNEL FLOW=
Qp=(AREA)(Q)(qu)

$t1 = \frac{0.007(nL1)^{0.8}}{(P2)^{0.5}(s)^{0.4}}$

$t2 = \frac{L2}{3600\ V2}$

$t3 = \frac{L3}{3600\ V^3}$

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

CLIENT WMCI
PROJECT PIEDMONT LRC
SUBJECT CELL STORAGE QUANTITY AND PUMP CAPACITY CALCULATIONS
PAGE 5 OF
PROJECT NO. 57221.400
Prepared By JCM Date 4/7/94
Reviewed By RJM Date 4/13/94
Approved By ____ Date ____

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:03
Watershed file: --> c:PIED1 .WSD
Hydrograph file: --> c:PIED1 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 1 - Subcell 3
24 hr - 25 yr Storm Storage

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
Ph3,Cell1,Sc3	7.30	80.0	0.50	0.00	6.00	3.78	.08 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 7.30 acres or 0.01141 sq.mi
Peak discharge = 23 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
Ph3,Cell1,Sc3	0.48	0.00	0.50	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:03
Watershed file: --> c:PIED1 .WSD
Hydrograph file: --> c:PIED1 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 1 - Subcell 3
24 hr - 25 yr Storm Storage

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
Ph3,Cell1,Sc3	23	12.4
Composite Watershed	23	12.4

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:03
Watershed file: --> c:PIED1 .WSD
Hydrograph file: --> c:PIED1 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 1 - Subcell 3
24 hr - 25 yr Storm Storage

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
Ph3,Cell1,Sc3	1	1	1	2	4	7	13	20	23
Total (cfs)	1	1	1	2	4	7	13	20	23

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
Ph3,Cell1,Sc3	22	17	13	10	6	4	3	3	2
Total (cfs)	22	17	13	10	6	4	3	3	2

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
Ph3,Cell1,Sc3	2	2	2	1	1	1	1	1	1
Total (cfs)	2	2	2	1	1	1	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
Ph3,Cell1,Sc3	1	1	1	1	0
Total (cfs)	1	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:03
Watershed file: --> c:PIED1 .WSD
Hydrograph file: --> c:PIED1 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 1 - Subcell 3
24 hr - 25 yr Storm Storage

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	1
11.1	1	14.9	1
11.2	1	15.0	1
11.3	1	15.1	1
11.4	1	15.2	1
11.5	1	15.3	1
11.6	1	15.4	1
11.7	1	15.5	1
11.8	2	15.6	1
11.9	2	15.7	1
12.0	4	15.8	1
12.1	7	15.9	1
12.2	13	16.0	1
12.3	20	16.1	1
12.4	23	16.2	1
12.5	22	16.3	1
12.6	17	16.4	1
12.7	13	16.5	1
12.8	10	16.6	1
12.9	8	16.7	1
13.0	6	16.8	1
13.1	5	16.9	1
13.2	4	17.0	1
13.3	3	17.1	1
13.4	3	17.2	1
13.5	3	17.3	1
13.6	3	17.4	1
13.7	2	17.5	1
13.8	2	17.6	1
13.9	2	17.7	1
14.0	2	17.8	1
14.1	2	17.9	1
14.2	2	18.0	1
14.3	2	18.1	1
14.4	2	18.2	1
14.5	2	18.3	1
14.6	2	18.4	1

14.7 2 18.5 1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:03
Watershed file: --> c:PIED1 .WSD
Hydrograph file: --> c:PIED1 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 1 - Subcell 3
24 hr - 25 yr Storm Storage

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* Piedmont Landfill and Recycling Center *
* Kernersville, Forsyth County, North Carolina *
* Phase 3 - Cell 1 - Subcell 3 *
* 24 hr - 25 yr Storm Storage *
*
****

Inflow Hydrograph: c:PIED1 .HYD
Rating Table file: c:PIED1 .PND

----INITIAL CONDITIONS----
Elevation = 763.80 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
763.80	0.0	0.000
764.00	0.0	0.000
766.00	0.0	0.125
768.00	0.0	0.782
770.00	0.0	2.687
772.00	0.0	6.291
773.00	0.0	8.760

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
0.1	0.1
30.1	30.1
189.2	189.2
650.3	650.3
1522.4	1522.4
2119.9	2119.9

Time increment (t) = 0.100 hrs.

Pond File: c:PIED1 .PND
Inflow Hydrograph: c:PIED1 .HYD
Outflow Hydrograph: c:POND1 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	1.00	-----	0.0	0.0	0.00	763.80
11.100	1.00	2.0	2.0	2.0	0.00	764.13
11.200	1.00	2.0	4.0	4.0	0.00	764.26
11.300	1.00	2.0	6.0	6.0	0.00	764.39
11.400	1.00	2.0	8.0	8.0	0.00	764.53
11.500	1.00	2.0	10.0	10.0	0.00	764.66
11.600	1.00	2.0	12.0	12.0	0.00	764.79
11.700	1.00	2.0	14.0	14.0	0.00	764.93
11.800	2.00	3.0	17.0	17.0	0.00	765.13
11.900	2.00	4.0	21.0	21.0	0.00	765.39
12.000	4.00	6.0	27.0	27.0	0.00	765.79
12.100	7.00	11.0	38.0	38.0	0.00	766.10
12.200	13.00	20.0	58.0	58.0	0.00	766.35
12.300	20.00	33.0	91.0	91.0	0.00	766.77
12.400	23.00	43.0	134.0	134.0	0.00	767.31
12.500	22.00	45.0	179.0	179.0	0.00	767.87
12.600	17.00	39.0	218.0	218.0	0.00	768.12
12.700	13.00	30.0	248.0	248.0	0.00	768.25
12.800	10.00	23.0	271.0	271.0	0.00	768.35
12.900	8.00	18.0	289.0	289.0	0.00	768.43
13.000	6.00	14.0	303.0	303.0	0.00	768.49
13.100	5.00	11.0	314.0	314.0	0.00	768.54
13.200	4.00	9.0	323.0	323.0	0.00	768.58
13.300	4.00	8.0	331.0	331.0	0.00	768.61
13.400	3.00	7.0	338.0	338.0	0.00	768.65
13.500	3.00	6.0	344.0	344.0	0.00	768.67
13.600	3.00	6.0	350.0	350.0	0.00	768.70
13.700	2.00	5.0	355.0	355.0	0.00	768.72
13.800	2.00	4.0	359.0	359.0	0.00	768.74
13.900	2.00	4.0	363.0	363.0	0.00	768.75
14.000	2.00	4.0	367.0	367.0	0.00	768.77
14.100	2.00	4.0	371.0	371.0	0.00	768.79
14.200	2.00	4.0	375.0	375.0	0.00	768.81
14.300	2.00	4.0	379.0	379.0	0.00	768.82
14.400	2.00	4.0	383.0	383.0	0.00	768.84
14.500	2.00	4.0	387.0	387.0	0.00	768.86
14.600	2.00	4.0	391.0	391.0	0.00	768.88
14.700	2.00	4.0	395.0	395.0	0.00	768.89
14.800	2.00	4.0	399.0	399.0	0.00	768.91
14.900	1.00	3.0	402.0	402.0	0.00	768.92
15.000	1.00	2.0	404.0	404.0	0.00	768.93
15.100	1.00	2.0	406.0	406.0	0.00	768.94
15.200	1.00	2.0	408.0	408.0	0.00	768.95
15.300	1.00	2.0	410.0	410.0	0.00	768.96
15.400	1.00	2.0	412.0	412.0	0.00	768.97

Pond File: c:PIED1 .PND
Inflow Hydrograph: c:PIED1 .HYD
Outflow Hydrograph: c:POND1 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
15.500	1.00
15.600	1.00
15.700	1.00
15.800	1.00
15.900	1.00
16.000	1.00
16.100	1.00
16.200	1.00
16.300	1.00
16.400	1.00
16.500	1.00
16.600	1.00
16.700	1.00
16.800	1.00
16.900	1.00
17.000	1.00
17.100	1.00
17.200	1.00
17.300	1.00
17.400	1.00
17.500	1.00
17.600	1.00
17.700	1.00
17.800	1.00
17.900	1.00
18.000	1.00
18.100	1.00
18.200	1.00
18.300	1.00
18.400	1.00
18.500	1.00
18.600	1.00
18.700	1.00
18.800	1.00
18.900	1.00
19.000	1.00
19.100	1.00
19.200	1.00
19.300	1.00
19.400	1.00
19.500	1.00
19.600	1.00
19.700	1.00
19.800	1.00
19.900	1.00
20.000	1.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
2.0	414.0	414.0	0.00	768.98
2.0	416.0	416.0	0.00	768.98
2.0	418.0	418.0	0.00	768.99
2.0	420.0	420.0	0.00	769.00
2.0	422.0	422.0	0.00	769.01
2.0	424.0	424.0	0.00	769.02
2.0	426.0	426.0	0.00	769.03
2.0	428.0	428.0	0.00	769.04
2.0	430.0	430.0	0.00	769.04
2.0	432.0	432.0	0.00	769.05
2.0	434.0	434.0	0.00	769.06
2.0	436.0	436.0	0.00	769.07
2.0	438.0	438.0	0.00	769.08
2.0	440.0	440.0	0.00	769.09
2.0	442.0	442.0	0.00	769.10
2.0	444.0	444.0	0.00	769.11
2.0	446.0	446.0	0.00	769.11
2.0	448.0	448.0	0.00	769.12
2.0	450.0	450.0	0.00	769.13
2.0	452.0	452.0	0.00	769.14
2.0	454.0	454.0	0.00	769.15
2.0	456.0	456.0	0.00	769.16
2.0	458.0	458.0	0.00	769.17
2.0	460.0	460.0	0.00	769.17
2.0	462.0	462.0	0.00	769.18
2.0	464.0	464.0	0.00	769.19
2.0	466.0	466.0	0.00	769.20
2.0	468.0	468.0	0.00	769.21
2.0	470.0	470.0	0.00	769.22
2.0	472.0	472.0	0.00	769.23
2.0	474.0	474.0	0.00	769.24
2.0	476.0	476.0	0.00	769.24
2.0	478.0	478.0	0.00	769.25
2.0	480.0	480.0	0.00	769.26
2.0	482.0	482.0	0.00	769.27
2.0	484.0	484.0	0.00	769.28
2.0	486.0	486.0	0.00	769.29
2.0	488.0	488.0	0.00	769.30
2.0	490.0	490.0	0.00	769.30
2.0	492.0	492.0	0.00	769.31
2.0	494.0	494.0	0.00	769.32
2.0	496.0	496.0	0.00	769.33
2.0	498.0	498.0	0.00	769.34
2.0	500.0	500.0	0.00	769.35
2.0	502.0	502.0	0.00	769.36
2.0	504.0	504.0	0.00	769.37

Pond File: c:PIED1 .PND
Inflow Hydrograph: c:PIED1 .HYD
Outflow Hydrograph: c:POND1 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	506.0	506.0	0.00	769.37
20.200	1.00	2.0	508.0	508.0	0.00	769.38
20.300	1.00	2.0	510.0	510.0	0.00	769.39
20.400	1.00	2.0	512.0	512.0	0.00	769.40
20.500	1.00	2.0	514.0	514.0	0.00	769.41
20.600	1.00	2.0	516.0	516.0	0.00	769.42
20.700	1.00	2.0	518.0	518.0	0.00	769.43
20.800	1.00	2.0	520.0	520.0	0.00	769.43
20.900	1.00	2.0	522.0	522.0	0.00	769.44
21.000	1.00	2.0	524.0	524.0	0.00	769.45
21.100	1.00	2.0	526.0	526.0	0.00	769.46
21.200	1.00	2.0	528.0	528.0	0.00	769.47
21.300	1.00	2.0	530.0	530.0	0.00	769.48
21.400	1.00	2.0	532.0	532.0	0.00	769.49
21.500	1.00	2.0	534.0	534.0	0.00	769.50
21.600	1.00	2.0	536.0	536.0	0.00	769.50
21.700	1.00	2.0	538.0	538.0	0.00	769.51
21.800	1.00	2.0	540.0	540.0	0.00	769.52
21.900	1.00	2.0	542.0	542.0	0.00	769.53
22.000	1.00	2.0	544.0	544.0	0.00	769.54
22.100	1.00	2.0	546.0	546.0	0.00	769.55
22.200	1.00	2.0	548.0	548.0	0.00	769.56
22.300	1.00	2.0	550.0	550.0	0.00	769.57
22.400	1.00	2.0	552.0	552.0	0.00	769.57
22.500	1.00	2.0	554.0	554.0	0.00	769.58
22.600	1.00	2.0	556.0	556.0	0.00	769.59
22.700	1.00	2.0	558.0	558.0	0.00	769.60
22.800	1.00	2.0	560.0	560.0	0.00	769.61
22.900	1.00	2.0	562.0	562.0	0.00	769.62
23.000	1.00	2.0	564.0	564.0	0.00	769.63
23.100	1.00	2.0	566.0	566.0	0.00	769.63
23.200	1.00	2.0	568.0	568.0	0.00	769.64
23.300	1.00	2.0	570.0	570.0	0.00	769.65
23.400	1.00	2.0	572.0	572.0	0.00	769.66
23.500	1.00	2.0	574.0	574.0	0.00	769.67
23.600	1.00	2.0	576.0	576.0	0.00	769.68
23.700	1.00	2.0	578.0	578.0	0.00	769.69
23.800	1.00	2.0	580.0	580.0	0.00	769.70
23.900	1.00	2.0	582.0	582.0	0.00	769.70
24.000	0.00	1.0	583.0	583.0	0.00	769.71
24.100	0.00	0.0	583.0	583.0	0.00	769.71
24.200	0.00	0.0	583.0	583.0	0.00	769.71
24.300	0.00	0.0	583.0	583.0	0.00	769.71
24.400	0.00	0.0	583.0	583.0	0.00	769.71
24.500	0.00	0.0	583.0	583.0	0.00	769.71
24.600	0.00	0.0	583.0	583.0	0.00	769.71

Pond File: c:PIED1 .PND
Inflow Hydrograph: c:PIED1 .HYD
Outflow Hydrograph: c:POND1 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
24.700	0.00	0.0	583.0	583.0	.0.00	769.71
24.800	0.00	0.0	583.0	583.0	0.00	769.71
24.900	0.00	0.0	583.0	583.0	0.00	769.71
25.000	0.00	0.0	583.0	583.0	0.00	769.71
25.100	0.00	0.0	583.0	583.0	0.00	769.71
25.200	0.00	0.0	583.0	583.0	0.00	769.71
25.300	0.00	0.0	583.0	583.0	0.00	769.71
25.400	0.00	0.0	583.0	583.0	0.00	769.71
25.500	0.00	0.0	583.0	583.0	0.00	769.71
25.600	0.00	0.0	583.0	583.0	0.00	769.71
25.700	0.00	0.0	583.0	583.0	0.00	769.71
25.800	0.00	0.0	583.0	583.0	0.00	769.71
25.900	0.00	0.0	583.0	583.0	0.00	769.71

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: c:PIED1 .PND
Inflow Hydrograph: c:PIED1 .HYD
Outflow Hydrograph: c:POND1 .HYD

Starting Pond W.S. Elevation = 763.80 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	23.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	769.71 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	2.41 ac-ft
Total Storage in Pond	=	2.41 ac-ft

Warning: Inflow hydrograph truncated on left side.

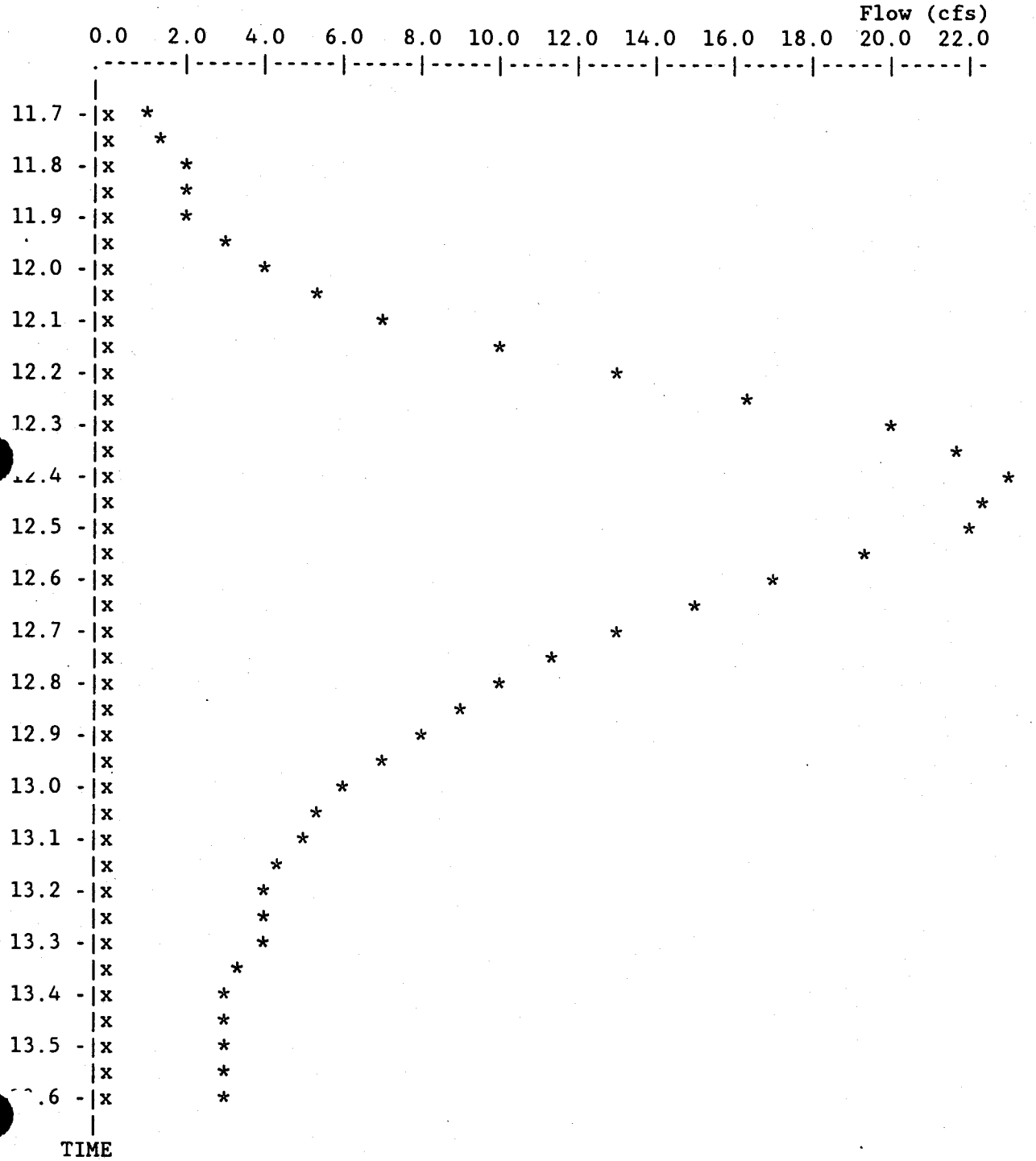
>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: c:PIED1 .PND
Inflow Hydrograph: c:PIED1 .HYD
Outflow Hydrograph: c:POND1 .HYD

EXECUTED: 04-08-1994
11:39:47

Peak Inflow = 23.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 769.71 ft



```
*  File:  c:PIED1    .HYD   Qmax =    23.0 cfs
x  File:  c:POND1    .HYD   Qmax =     0.0 cfs
```

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:27
Watershed file: --> c:PIED2 .WSD
Hydrograph file: --> c:PIED2 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 1
24 hr - 25 yr Storm Storage

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
Ph3,Cell2,Scl	9.94	80.0	0.50	0.00	6.00	3.78	.08 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 9.94 acres or 0.01553 sq.mi
Peak discharge = 31 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	Input Values * Tt (hr)	Rounded Values Tc (hr)	Rounded Values * Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
Ph3,Cell2,Scl	0.48	0.00	0.50	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:27
Watershed file: --> c:PIED2 .WSD
Hydrograph file: --> c:PIED2 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 1
24 hr - 25 yr Storm Storage

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
Ph3,Cell2,Scl	31	12.4
Composite Watershed	31	12.4

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:27
Watershed file: --> c:PIED2 .WSD
Hydrograph file: --> c:PIED2 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 1
24 hr - 25 yr Storm Storage

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
Ph3,Cell2,Scl	1	1	2	3	6	10	18	27	31
Total (cfs)	1	1	2	3	6	10	18	27	31

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
Ph3,Cell2,Scl	30	24	17	13	8	6	4	4	3
Total (cfs)	30	24	17	13	8	6	4	4	3

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
Ph3,Cell2,Scl	3	2	2	2	2	2	1	1	1
Total (cfs)	3	2	2	2	2	2	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
Ph3,Cell2,Scl	1	1	1	1	0
Total (cfs)	1	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:27
Watershed file: --> c:PIED2 .WSD
Hydrograph file: --> c:PIED2 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 1
24 hr - 25 yr Storm Storage

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	2
11.1	1	14.9	2
11.2	1	15.0	2
11.3	1	15.1	2
11.4	1	15.2	2
11.5	2	15.3	2
11.6	2	15.4	2
11.7	2	15.5	2
11.8	3	15.6	2
11.9	3	15.7	2
12.0	6	15.8	2
12.1	10	15.9	2
12.2	18	16.0	2
12.3	27	16.1	2
12.4	31	16.2	2
12.5	30	16.3	1
12.6	24	16.4	1
12.7	17	16.5	1
12.8	13	16.6	1
12.9	10	16.7	1
13.0	8	16.8	1
13.1	7	16.9	1
13.2	6	17.0	1
13.3	5	17.1	1
13.4	4	17.2	1
13.5	4	17.3	1
13.6	4	17.4	1
13.7	3	17.5	1
13.8	3	17.6	1
13.9	3	17.7	1
14.0	3	17.8	1
14.1	3	17.9	1
14.2	2	18.0	1
14.3	2	18.1	1
14.4	2	18.2	1
14.5	2	18.3	1
14.6	2	18.4	1

14.7	2	18.5	1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:27
Watershed file: --> c:PIED2 .WSD
Hydrograph file: --> c:PIED2 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 1
24 hr - 25 yr Storm Storage

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* Piedmont Landfill and Recycling Center *
* Kernersville, Forsyth County, North Carolina *
* Phase 3 - Cell 2 - Subcell 1 *
* 24 hr - 25 yr Storm Storage *
*
****

Inflow Hydrograph: c:PIED2 .HYD
Rating Table file: c:PIED2 .PND

----INITIAL CONDITIONS----
Elevation = 757.00 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
757.00	0.0	0.000
758.00	0.0	0.028
760.00	0.0	0.439
762.00	0.0	2.210
762.90	0.0	3.929

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
6.9	6.9
106.1	106.1
534.8	534.8
950.8	950.8

Time increment (t) = 0.100 hrs.

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	1.00	-----	0.0	0.0	0.00	757.00
11.100	1.00	2.0	2.0	2.0	0.00	757.29
11.200	1.00	2.0	4.0	4.0	0.00	757.58
11.300	1.00	2.0	6.0	6.0	0.00	757.87
11.400	1.00	2.0	8.0	8.0	0.00	758.02
11.500	2.00	3.0	11.0	11.0	0.00	758.08
11.600	2.00	4.0	15.0	15.0	0.00	758.16
11.700	2.00	4.0	19.0	19.0	0.00	758.24
11.800	3.00	5.0	24.0	24.0	0.00	758.35
11.900	3.00	6.0	30.0	30.0	0.00	758.47
12.000	6.00	9.0	39.0	39.0	0.00	758.65
12.100	10.00	16.0	55.0	55.0	0.00	758.97
12.200	18.00	28.0	83.0	83.0	0.00	759.53
12.300	27.00	45.0	128.0	128.0	0.00	760.10
12.400	31.00	58.0	186.0	186.0	0.00	760.37
12.500	30.00	61.0	247.0	247.0	0.00	760.66
12.600	24.00	54.0	301.0	301.0	0.00	760.91
12.700	17.00	41.0	342.0	342.0	0.00	761.10
12.800	13.00	30.0	372.0	372.0	0.00	761.24
12.900	10.00	23.0	395.0	395.0	0.00	761.35
13.000	8.00	18.0	413.0	413.0	0.00	761.43
13.100	7.00	15.0	428.0	428.0	0.00	761.50
13.200	6.00	13.0	441.0	441.0	0.00	761.56
13.300	5.00	11.0	452.0	452.0	0.00	761.61
13.400	4.00	9.0	461.0	461.0	0.00	761.66
13.500	4.00	8.0	469.0	469.0	0.00	761.69
13.600	4.00	8.0	477.0	477.0	0.00	761.73
13.700	4.00	8.0	485.0	485.0	0.00	761.77
13.800	3.00	7.0	492.0	492.0	0.00	761.80
13.900	3.00	6.0	498.0	498.0	0.00	761.83
14.000	3.00	6.0	504.0	504.0	0.00	761.86
14.100	3.00	6.0	510.0	510.0	0.00	761.88
14.200	2.00	5.0	515.0	515.0	0.00	761.91
14.300	2.00	4.0	519.0	519.0	0.00	761.93
14.400	2.00	4.0	523.0	523.0	0.00	761.94
14.500	2.00	4.0	527.0	527.0	0.00	761.96
14.600	2.00	4.0	531.0	531.0	0.00	761.98
14.700	2.00	4.0	535.0	535.0	0.00	762.00
14.800	2.00	4.0	539.0	539.0	0.00	762.01
14.900	2.00	4.0	543.0	543.0	0.00	762.02
15.000	2.00	4.0	547.0	547.0	0.00	762.03
15.100	2.00	4.0	551.0	551.0	0.00	762.04
15.200	2.00	4.0	555.0	555.0	0.00	762.04
15.300	2.00	4.0	559.0	559.0	0.00	762.05
15.400	2.00	4.0	563.0	563.0	0.00	762.06

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
15.500	2.00
15.600	2.00
15.700	2.00
15.800	2.00
15.900	2.00
16.000	2.00
16.100	2.00
16.200	2.00
16.300	1.00
16.400	1.00
16.500	1.00
16.600	1.00
16.700	1.00
16.800	1.00
16.900	1.00
17.000	1.00
17.100	1.00
17.200	1.00
17.300	1.00
17.400	1.00
17.500	1.00
17.600	1.00
17.700	1.00
17.800	1.00
17.900	1.00
18.000	1.00
18.100	1.00
18.200	1.00
18.300	1.00
18.400	1.00
18.500	1.00
18.600	1.00
18.700	1.00
18.800	1.00
18.900	1.00
19.000	1.00
19.100	1.00
19.200	1.00
19.300	1.00
19.400	1.00
19.500	1.00
19.600	1.00
19.700	1.00
19.800	1.00
19.900	1.00
20.000	1.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
4.0	567.0	567.0	0.00	762.07
4.0	571.0	571.0	0.00	762.08
4.0	575.0	575.0	0.00	762.09
4.0	579.0	579.0	0.00	762.10
4.0	583.0	583.0	0.00	762.10
4.0	587.0	587.0	0.00	762.11
4.0	591.0	591.0	0.00	762.12
4.0	595.0	595.0	0.00	762.13
3.0	598.0	598.0	0.00	762.14
2.0	600.0	600.0	0.00	762.14
2.0	602.0	602.0	0.00	762.15
2.0	604.0	604.0	0.00	762.15
2.0	606.0	606.0	0.00	762.15
2.0	608.0	608.0	0.00	762.16
2.0	610.0	610.0	0.00	762.16
2.0	612.0	612.0	0.00	762.17
2.0	614.0	614.0	0.00	762.17
2.0	616.0	616.0	0.00	762.18
2.0	618.0	618.0	0.00	762.18
2.0	620.0	620.0	0.00	762.18
2.0	622.0	622.0	0.00	762.19
2.0	624.0	624.0	0.00	762.19
2.0	626.0	626.0	0.00	762.20
2.0	628.0	628.0	0.00	762.20
2.0	630.0	630.0	0.00	762.21
2.0	632.0	632.0	0.00	762.21
2.0	634.0	634.0	0.00	762.21
2.0	636.0	636.0	0.00	762.22
2.0	638.0	638.0	0.00	762.22
2.0	640.0	640.0	0.00	762.23
2.0	642.0	642.0	0.00	762.23
2.0	644.0	644.0	0.00	762.24
2.0	646.0	646.0	0.00	762.24
2.0	648.0	648.0	0.00	762.24
2.0	650.0	650.0	0.00	762.25
2.0	652.0	652.0	0.00	762.25
2.0	654.0	654.0	0.00	762.26
2.0	656.0	656.0	0.00	762.26
2.0	658.0	658.0	0.00	762.27
2.0	660.0	660.0	0.00	762.27
2.0	662.0	662.0	0.00	762.28
2.0	664.0	664.0	0.00	762.28
2.0	666.0	666.0	0.00	762.28
2.0	668.0	668.0	0.00	762.29
2.0	670.0	670.0	0.00	762.29
2.0	672.0	672.0	0.00	762.30

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	674.0	674.0	0.00	762.30
20.200	1.00	2.0	676.0	676.0	0.00	762.31
20.300	1.00	2.0	678.0	678.0	0.00	762.31
20.400	1.00	2.0	680.0	680.0	0.00	762.31
20.500	1.00	2.0	682.0	682.0	0.00	762.32
20.600	1.00	2.0	684.0	684.0	0.00	762.32
20.700	1.00	2.0	686.0	686.0	0.00	762.33
20.800	1.00	2.0	688.0	688.0	0.00	762.33
20.900	1.00	2.0	690.0	690.0	0.00	762.34
21.000	1.00	2.0	692.0	692.0	0.00	762.34
21.100	1.00	2.0	694.0	694.0	0.00	762.34
21.200	1.00	2.0	696.0	696.0	0.00	762.35
21.300	1.00	2.0	698.0	698.0	0.00	762.35
21.400	1.00	2.0	700.0	700.0	0.00	762.36
21.500	1.00	2.0	702.0	702.0	0.00	762.36
21.600	1.00	2.0	704.0	704.0	0.00	762.37
21.700	1.00	2.0	706.0	706.0	0.00	762.37
21.800	1.00	2.0	708.0	708.0	0.00	762.37
21.900	1.00	2.0	710.0	710.0	0.00	762.38
22.000	1.00	2.0	712.0	712.0	0.00	762.38
22.100	1.00	2.0	714.0	714.0	0.00	762.39
22.200	1.00	2.0	716.0	716.0	0.00	762.39
22.300	1.00	2.0	718.0	718.0	0.00	762.40
22.400	1.00	2.0	720.0	720.0	0.00	762.40
22.500	1.00	2.0	722.0	722.0	0.00	762.40
22.600	1.00	2.0	724.0	724.0	0.00	762.41
22.700	1.00	2.0	726.0	726.0	0.00	762.41
22.800	1.00	2.0	728.0	728.0	0.00	762.42
22.900	1.00	2.0	730.0	730.0	0.00	762.42
23.000	1.00	2.0	732.0	732.0	0.00	762.43
23.100	1.00	2.0	734.0	734.0	0.00	762.43
23.200	1.00	2.0	736.0	736.0	0.00	762.44
23.300	1.00	2.0	738.0	738.0	0.00	762.44
23.400	1.00	2.0	740.0	740.0	0.00	762.44
23.500	1.00	2.0	742.0	742.0	0.00	762.45
23.600	1.00	2.0	744.0	744.0	0.00	762.45
23.700	1.00	2.0	746.0	746.0	0.00	762.46
23.800	1.00	2.0	748.0	748.0	0.00	762.46
23.900	1.00	2.0	750.0	750.0	0.00	762.47
24.000	0.00	1.0	751.0	751.0	0.00	762.47
24.100	0.00	0.0	751.0	751.0	0.00	762.47
24.200	0.00	0.0	751.0	751.0	0.00	762.47
24.300	0.00	0.0	751.0	751.0	0.00	762.47
24.400	0.00	0.0	751.0	751.0	0.00	762.47
24.500	0.00	0.0	751.0	751.0	0.00	762.47
24.600	0.00	0.0	751.0	751.0	0.00	762.47

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
24.700	0.00	0.0	751.0	751.0	0.00	762.47
24.800	0.00	0.0	751.0	751.0	0.00	762.47
24.900	0.00	0.0	751.0	751.0	0.00	762.47
25.000	0.00	0.0	751.0	751.0	0.00	762.47
25.100	0.00	0.0	751.0	751.0	0.00	762.47
25.200	0.00	0.0	751.0	751.0	0.00	762.47
25.300	0.00	0.0	751.0	751.0	0.00	762.47
25.400	0.00	0.0	751.0	751.0	0.00	762.47
25.500	0.00	0.0	751.0	751.0	0.00	762.47
25.600	0.00	0.0	751.0	751.0	0.00	762.47
25.700	0.00	0.0	751.0	751.0	0.00	762.47
25.800	0.00	0.0	751.0	751.0	0.00	762.47
25.900	0.00	0.0	751.0	751.0	0.00	762.47

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

Starting Pond W.S. Elevation = 757.00 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	31.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	762.47 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	3.10 ac-ft
Total Storage in Pond	=	3.10 ac-ft

Warning: Inflow hydrograph truncated on left side.

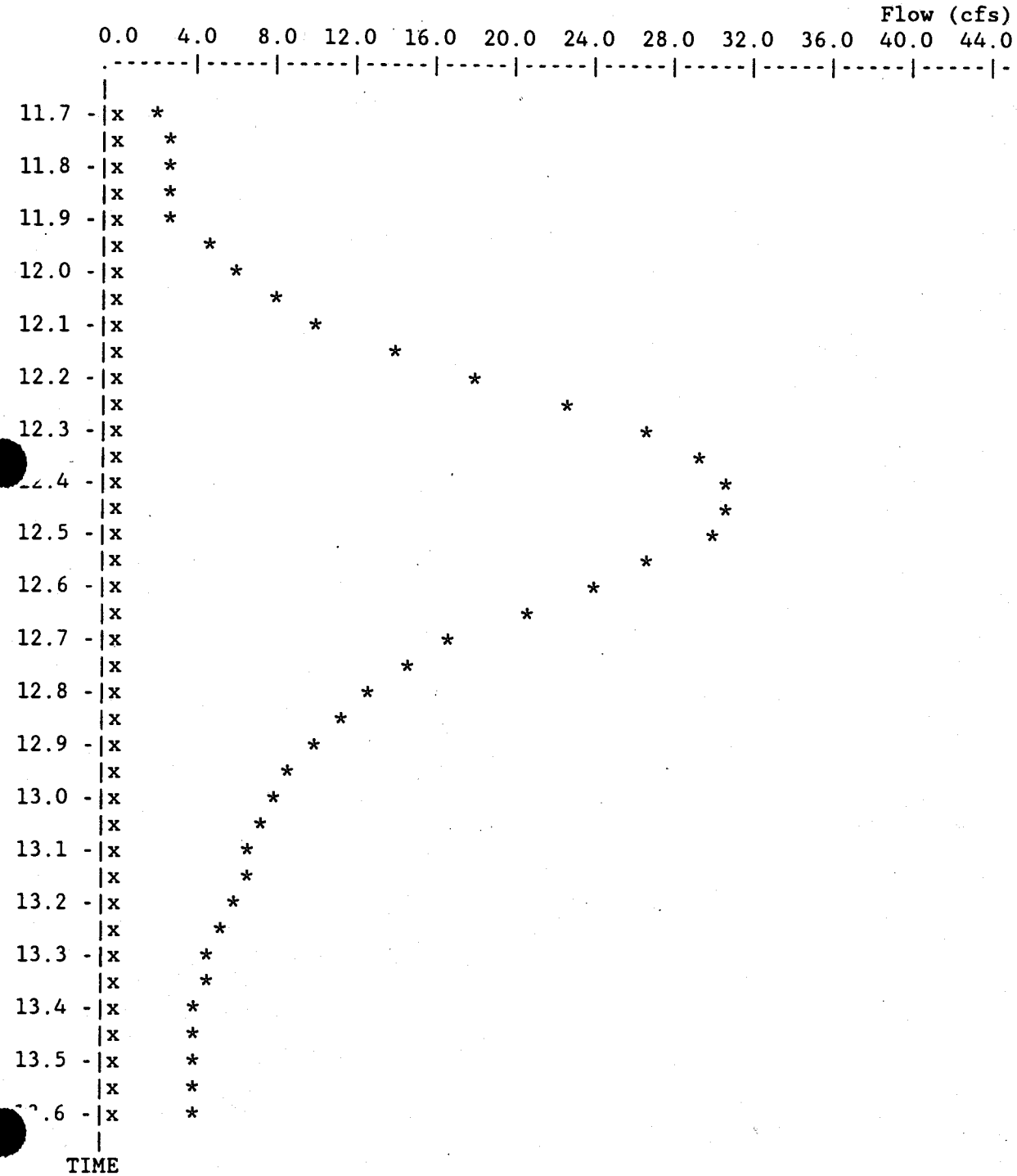
>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<
POND-2 Version: 5.15 S/N: 1295100074 Page 7

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

EXECUTED: 04-08-1994
11:40:28

Peak Inflow = 31.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 762.47 ft



```
*  File:  c:PIED2    .HYD   Qmax =     31.0 cfs
x  File:  c:POND2    .HYD   Qmax =      0.0 cfs
```

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:36
Watershed file: --> c:PIED3 .WSD
Hydrograph file: --> c:PIED3 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 2
24 hr - 25 yr Storm Storage

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
Ph3,Cell2,Sc2	9.27	80.0	0.50	0.00	6.00	3.78	.08 .10

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 9.27 acres or 0.01448 sq.mi
Peak discharge = 29 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	* Tt (hr)	Rounded Values Tc (hr)	* Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
Ph3,Cell2,Sc2	0.49	0.00	0.50	0.00	No	Computed Ia/p < .1

* Travel time from subarea outfall to composite watershed outfall point.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:36
Watershed file: --> c:PIED3 .WSD
Hydrograph file: --> c:PIED3 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 2
24 hr - 25 yr Storm Storage

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
Ph3,Cell2,Sc2	29	12.4
Composite Watershed	29	12.4

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:36
Watershed file: --> c:PIED3 .WSD
Hydrograph file: --> c:PIED3 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 2
24 hr - 25 yr Storm Storage

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
Ph3,Cell2,Sc2	1	1	2	3	5	9	17	26	29
Total (cfs)	1	1	2	3	5	9	17	26	29

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
Ph3,Cell2,Sc2	28	22	16	12	8	5	4	3	3
Total (cfs)	28	22	16	12	8	5	4	3	3

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
Ph3,Cell2,Sc2	3	2	2	2	2	1	1	1	1
Total (cfs)	3	2	2	2	2	1	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
Ph3,Cell2,Sc2	1	1	1	1	0
Total (cfs)	1	1	1	1	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:36
Watershed file: --> c:PIED3 .WSD
Hydrograph file: --> c:PIED3 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 2
24 hr - 25 yr Storm Storage

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	1	14.8	2
11.1	1	14.9	2
11.2	1	15.0	2
11.3	1	15.1	2
11.4	1	15.2	2
11.5	2	15.3	2
11.6	2	15.4	2
11.7	2	15.5	2
11.8	3	15.6	2
11.9	3	15.7	2
12.0	5	15.8	1
12.1	9	15.9	1
12.2	17	16.0	1
12.3	26	16.1	1
12.4	29	16.2	1
12.5	28	16.3	1
12.6	22	16.4	1
12.7	16	16.5	1
12.8	12	16.6	1
12.9	10	16.7	1
13.0	8	16.8	1
13.1	6	16.9	1
13.2	5	17.0	1
13.3	4	17.1	1
13.4	4	17.2	1
13.5	4	17.3	1
13.6	3	17.4	1
13.7	3	17.5	1
13.8	3	17.6	1
13.9	3	17.7	1
14.0	3	17.8	1
14.1	3	17.9	1
14.2	2	18.0	1
14.3	2	18.1	1
14.4	2	18.2	1
14.5	2	18.3	1
14.6	2	18.4	1

14.7 2 18.5 1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 04-08-1994 11:30:36
Watershed file: --> c:PIED3 .WSD
Hydrograph file: --> c:PIED3 .HYD

Piedmont Landfill and Recycling Center
Kernersville, Forsyth County, North Carolina
Phase 3 - Cell 2 - Subcell 2
24 hr - 25 yr Storm Storage

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	1
18.7	1	22.5	1
18.8	1	22.6	1
18.9	1	22.7	1
19.0	1	22.8	1
19.1	1	22.9	1
19.2	1	23.0	1
19.3	1	23.1	1
19.4	1	23.2	1
19.5	1	23.3	1
19.6	1	23.4	1
19.7	1	23.5	1
19.8	1	23.6	1
19.9	1	23.7	1
20.0	1	23.8	1
20.1	1	23.9	1
20.2	1	24.0	1
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	1	24.8	0
21.1	1	24.9	0
21.2	1	25.0	0
21.3	1	25.1	0
21.4	1	25.2	0
21.5	1	25.3	0
21.6	1	25.4	0
21.7	1	25.5	0
21.8	1	25.6	0
21.9	1	25.7	0
22.0	1	25.8	0
22.1	1	25.9	0
22.2	1		
22.3	1		

****
*
* Piedmont Landfill and Recycling Center *
* Kernersville, Forsyth County, North Carolina *
* Phase 3 - Cell 2 - Subcell 2 *
* 24 hr - 25 yr Storm Storage *
*
****

Inflow Hydrograph: c:PIED3 .HYD
Rating Table file: c:PIED3 .PND

----INITIAL CONDITIONS----
Elevation = 770.20 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
770.20	0.0	0.000
772.00	0.0	0.230
774.00	0.0	1.000
776.00	0.0	3.148

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
55.7	55.7
242.0	242.0
761.8	761.8

Time increment (t) = 0.100 hrs.

Pond File: c:PIED3 .PND
Inflow Hydrograph: c:PIED3 .HYD
Outflow Hydrograph: c:POND3 .HYD

INFLOW HYDROGRAPH | ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	1.00	-----	0.0	0.0	0.00	770.20
11.100	1.00	2.0	2.0	2.0	0.00	770.26
11.200	1.00	2.0	4.0	4.0	0.00	770.33
11.300	1.00	2.0	6.0	6.0	0.00	770.39
11.400	1.00	2.0	8.0	8.0	0.00	770.46
11.500	2.00	3.0	11.0	11.0	0.00	770.56
11.600	2.00	4.0	15.0	15.0	0.00	770.69
11.700	2.00	4.0	19.0	19.0	0.00	770.81
11.800	3.00	5.0	24.0	24.0	0.00	770.98
11.900	3.00	6.0	30.0	30.0	0.00	771.17
12.000	5.00	8.0	38.0	38.0	0.00	771.43
12.100	9.00	14.0	52.0	52.0	0.00	771.88
12.200	17.00	26.0	78.0	78.0	0.00	772.24
12.300	26.00	43.0	121.0	121.0	0.00	772.70
12.400	29.00	55.0	176.0	176.0	0.00	773.29
12.500	28.00	57.0	233.0	233.0	0.00	773.90
12.600	22.00	50.0	283.0	283.0	0.00	774.16
12.700	16.00	38.0	321.0	321.0	0.00	774.30
12.800	12.00	28.0	349.0	349.0	0.00	774.41
12.900	10.00	22.0	371.0	371.0	0.00	774.50
13.000	8.00	18.0	389.0	389.0	0.00	774.57
13.100	6.00	14.0	403.0	403.0	0.00	774.62
13.200	5.00	11.0	414.0	414.0	0.00	774.66
13.300	4.00	9.0	423.0	423.0	0.00	774.70
13.400	4.00	8.0	431.0	431.0	0.00	774.73
13.500	4.00	8.0	439.0	439.0	0.00	774.76
13.600	3.00	7.0	446.0	446.0	0.00	774.78
13.700	3.00	6.0	452.0	452.0	0.00	774.81
13.800	3.00	6.0	458.0	458.0	0.00	774.83
13.900	3.00	6.0	464.0	464.0	0.00	774.85
14.000	3.00	6.0	470.0	470.0	0.00	774.88
14.100	3.00	6.0	476.0	476.0	0.00	774.90
14.200	2.00	5.0	481.0	481.0	0.00	774.92
14.300	2.00	4.0	485.0	485.0	0.00	774.93
14.400	2.00	4.0	489.0	489.0	0.00	774.95
14.500	2.00	4.0	493.0	493.0	0.00	774.97
14.600	2.00	4.0	497.0	497.0	0.00	774.98
14.700	2.00	4.0	501.0	501.0	0.00	775.00
14.800	2.00	4.0	505.0	505.0	0.00	775.01
14.900	2.00	4.0	509.0	509.0	0.00	775.03
15.000	2.00	4.0	513.0	513.0	0.00	775.04
15.100	2.00	4.0	517.0	517.0	0.00	775.06
15.200	2.00	4.0	521.0	521.0	0.00	775.07
15.300	2.00	4.0	525.0	525.0	0.00	775.09
15.400	2.00	4.0	529.0	529.0	0.00	775.10

Pond File: c:PIED3 .PND
Inflow Hydrograph: c:PIED3 .HYD
Outflow Hydrograph: c:POND3 .HYD

INFLOW HYDROGRAPH / ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	2.00	4.0	533.0	533.0	0.00	775.12
15.600	2.00	4.0	537.0	537.0	0.00	775.14
15.700	2.00	4.0	541.0	541.0	0.00	775.15
15.800	1.00	3.0	544.0	544.0	0.00	775.16
15.900	1.00	2.0	546.0	546.0	0.00	775.17
16.000	1.00	2.0	548.0	548.0	0.00	775.18
16.100	1.00	2.0	550.0	550.0	0.00	775.19
16.200	1.00	2.0	552.0	552.0	0.00	775.19
16.300	1.00	2.0	554.0	554.0	0.00	775.20
16.400	1.00	2.0	556.0	556.0	0.00	775.21
16.500	1.00	2.0	558.0	558.0	0.00	775.22
16.600	1.00	2.0	560.0	560.0	0.00	775.22
16.700	1.00	2.0	562.0	562.0	0.00	775.23
16.800	1.00	2.0	564.0	564.0	0.00	775.24
16.900	1.00	2.0	566.0	566.0	0.00	775.25
17.000	1.00	2.0	568.0	568.0	0.00	775.25
17.100	1.00	2.0	570.0	570.0	0.00	775.26
17.200	1.00	2.0	572.0	572.0	0.00	775.27
17.300	1.00	2.0	574.0	574.0	0.00	775.28
17.400	1.00	2.0	576.0	576.0	0.00	775.29
17.500	1.00	2.0	578.0	578.0	0.00	775.29
17.600	1.00	2.0	580.0	580.0	0.00	775.30
17.700	1.00	2.0	582.0	582.0	0.00	775.31
17.800	1.00	2.0	584.0	584.0	0.00	775.32
17.900	1.00	2.0	586.0	586.0	0.00	775.32
18.000	1.00	2.0	588.0	588.0	0.00	775.33
18.100	1.00	2.0	590.0	590.0	0.00	775.34
18.200	1.00	2.0	592.0	592.0	0.00	775.35
18.300	1.00	2.0	594.0	594.0	0.00	775.35
18.400	1.00	2.0	596.0	596.0	0.00	775.36
18.500	1.00	2.0	598.0	598.0	0.00	775.37
18.600	1.00	2.0	600.0	600.0	0.00	775.38
18.700	1.00	2.0	602.0	602.0	0.00	775.39
18.800	1.00	2.0	604.0	604.0	0.00	775.39
18.900	1.00	2.0	606.0	606.0	0.00	775.40
19.000	1.00	2.0	608.0	608.0	0.00	775.41
19.100	1.00	2.0	610.0	610.0	0.00	775.42
19.200	1.00	2.0	612.0	612.0	0.00	775.42
19.300	1.00	2.0	614.0	614.0	0.00	775.43
19.400	1.00	2.0	616.0	616.0	0.00	775.44
19.500	1.00	2.0	618.0	618.0	0.00	775.45
19.600	1.00	2.0	620.0	620.0	0.00	775.45
19.700	1.00	2.0	622.0	622.0	0.00	775.46
19.800	1.00	2.0	624.0	624.0	0.00	775.47
19.900	1.00	2.0	626.0	626.0	0.00	775.48
20.000	1.00	2.0	628.0	628.0	0.00	775.49

Pond File: c:PIED3 .PND
Inflow Hydrograph: c:PIED3 .HYD
Outflow Hydrograph: c:POND3 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	630.0	630.0	0.00	775.49
20.200	1.00	2.0	632.0	632.0	0.00	775.50
20.300	1.00	2.0	634.0	634.0	0.00	775.51
20.400	1.00	2.0	636.0	636.0	0.00	775.52
20.500	1.00	2.0	638.0	638.0	0.00	775.52
20.600	1.00	2.0	640.0	640.0	0.00	775.53
20.700	1.00	2.0	642.0	642.0	0.00	775.54
20.800	1.00	2.0	644.0	644.0	0.00	775.55
20.900	1.00	2.0	646.0	646.0	0.00	775.55
21.000	1.00	2.0	648.0	648.0	0.00	775.56
21.100	1.00	2.0	650.0	650.0	0.00	775.57
21.200	1.00	2.0	652.0	652.0	0.00	775.58
21.300	1.00	2.0	654.0	654.0	0.00	775.59
21.400	1.00	2.0	656.0	656.0	0.00	775.59
21.500	1.00	2.0	658.0	658.0	0.00	775.60
21.600	1.00	2.0	660.0	660.0	0.00	775.61
21.700	1.00	2.0	662.0	662.0	0.00	775.62
21.800	1.00	2.0	664.0	664.0	0.00	775.62
21.900	1.00	2.0	666.0	666.0	0.00	775.63
22.000	1.00	2.0	668.0	668.0	0.00	775.64
22.100	1.00	2.0	670.0	670.0	0.00	775.65
22.200	1.00	2.0	672.0	672.0	0.00	775.65
22.300	1.00	2.0	674.0	674.0	0.00	775.66
22.400	1.00	2.0	676.0	676.0	0.00	775.67
22.500	1.00	2.0	678.0	678.0	0.00	775.68
22.600	1.00	2.0	680.0	680.0	0.00	775.69
22.700	1.00	2.0	682.0	682.0	0.00	775.69
22.800	1.00	2.0	684.0	684.0	0.00	775.70
22.900	1.00	2.0	686.0	686.0	0.00	775.71
23.000	1.00	2.0	688.0	688.0	0.00	775.72
23.100	1.00	2.0	690.0	690.0	0.00	775.72
23.200	1.00	2.0	692.0	692.0	0.00	775.73
23.300	1.00	2.0	694.0	694.0	0.00	775.74
23.400	1.00	2.0	696.0	696.0	0.00	775.75
23.500	1.00	2.0	698.0	698.0	0.00	775.75
23.600	1.00	2.0	700.0	700.0	0.00	775.76
23.700	1.00	2.0	702.0	702.0	0.00	775.77
23.800	1.00	2.0	704.0	704.0	0.00	775.78
23.900	1.00	2.0	706.0	706.0	0.00	775.79
24.000	0.00	1.0	707.0	707.0	0.00	775.79
24.100	0.00	0.0	707.0	707.0	0.00	775.79
24.200	0.00	0.0	707.0	707.0	0.00	775.79
24.300	0.00	0.0	707.0	707.0	0.00	775.79
24.400	0.00	0.0	707.0	707.0	0.00	775.79
24.500	0.00	0.0	707.0	707.0	0.00	775.79
24.600	0.00	0.0	707.0	707.0	0.00	775.79

Pond File: c:PIED3 .PND
Inflow Hydrograph: c:PIED3 .HYD
Outflow Hydrograph: c:POND3 .HYD

INFLOW HYDROGRAPH / ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
24.700	0.00	0.0	707.0	707.0	0.00	775.79
24.800	0.00	0.0	707.0	707.0	0.00	775.79
24.900	0.00	0.0	707.0	707.0	0.00	775.79
25.000	0.00	0.0	707.0	707.0	0.00	775.79
25.100	0.00	0.0	707.0	707.0	0.00	775.79
25.200	0.00	0.0	707.0	707.0	0.00	775.79
25.300	0.00	0.0	707.0	707.0	0.00	775.79
25.400	0.00	0.0	707.0	707.0	0.00	775.79
25.500	0.00	0.0	707.0	707.0	0.00	775.79
25.600	0.00	0.0	707.0	707.0	0.00	775.79
25.700	0.00	0.0	707.0	707.0	0.00	775.79
25.800	0.00	0.0	707.0	707.0	0.00	775.79
25.900	0.00	0.0	707.0	707.0	0.00	775.79

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: c:PIED3 .PND
Inflow Hydrograph: c:PIED3 .HYD
Outflow Hydrograph: c:POND3 .HYD

Starting Pond W.S. Elevation = 770.20 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	29.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	775.79 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	2.92 ac-ft
Total Storage in Pond	=	2.92 ac-ft

Warning: Inflow hydrograph truncated on left side.

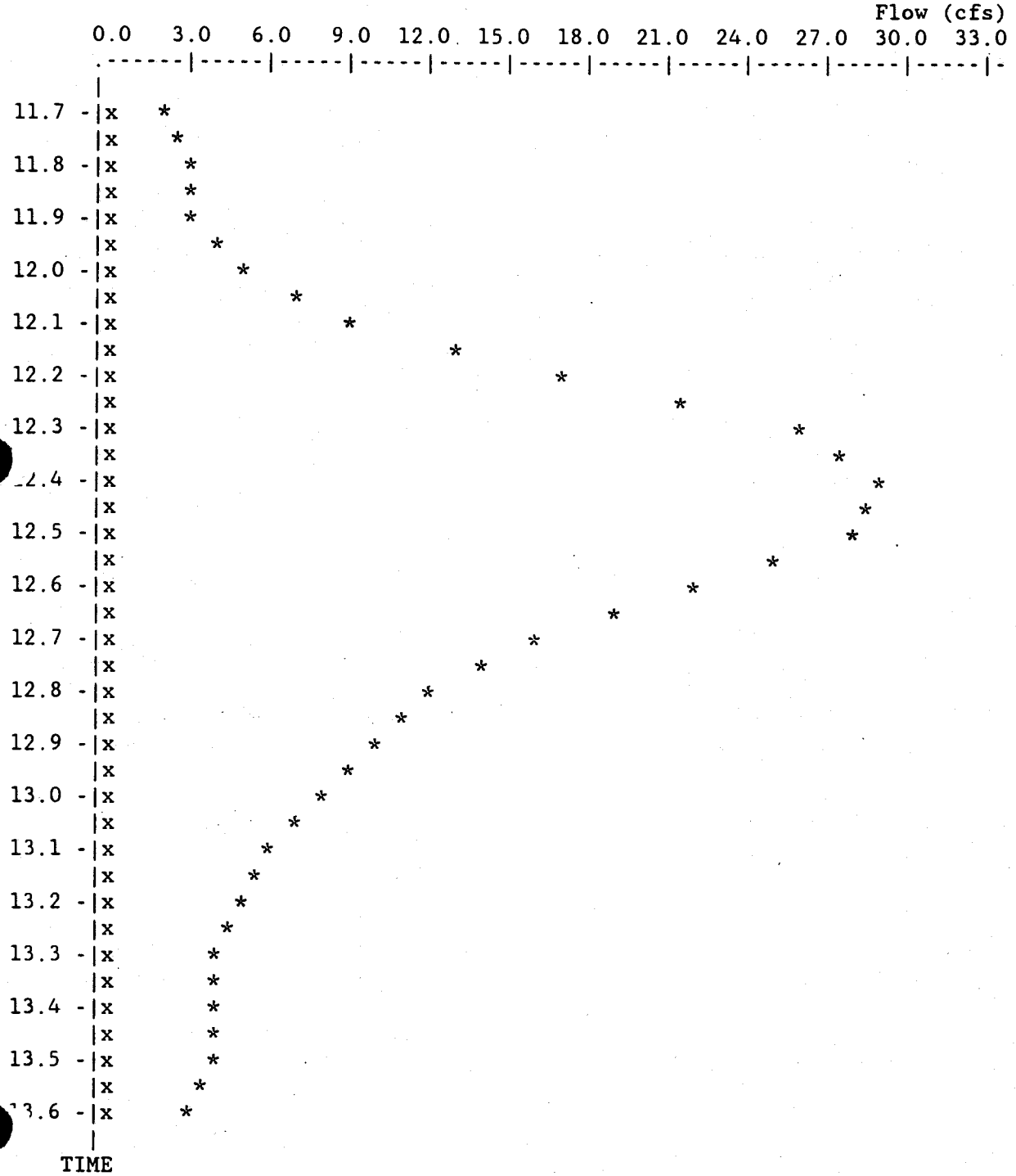
>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: c:PIED3 .PND
Inflow Hydrograph: c:PIED3 .HYD
Outflow Hydrograph: c:POND3 .HYD

EXECUTED: 04-08-1994
11:40:49

Peak Inflow = 29.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 775.79 ft



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*  File:  c:PIED3    .HYD   Qmax =     29.0 cfs
x  File:  c:POND3    .HYD   Qmax =      0.0 cfs
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RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 2
PROJECT NO. 27221

CLIENT Piedmont LRC
PROJECT Transition Application and Permit Renewal

SUBJECT Dewatering Time for Phase 3-Cell 2-Subcell 1

Prepared By JSM Date 4/13/94
Reviewed By JJM Date 4/13/94
Approved By ____ Date ____

Purpose: Determine the time required to empty the stormwater collected from the 25-year, 24-hour storm event within Phase 3-Cell 2-Subcell 1. The situation will be analyzed under "open cell" conditions, where no stormwater can run-off and out of the cell.

Given & Assumed:

- From the Leachate Containment Calculation, provided in Appendix D of the Engineering Report (calculation prepared 10/15/93 and revised 4/13/94)
  - Volume Available in Cell for Storage: 3.929 acre-ft
  - 25-year / 24-hour pond routing information included.

Calculation:

- Determine the total volume of water entering Phase 3-Cell 2-Subcell 1.

From attached data, the various inflow rates and lengths of time at these inflow rates are as follows:

Inflow (CFS)	Time (hours)	Volume (Ft^3)
1	8.2	29,520
2	2.4	17,280
3	0.6	6,480
4	0.4	5,760
5	0.1	1800
6	0.2	4,320
7	0.1	2,520
8	0.1	2,880
10	0.2	7,200
13	0.1	4,680
17	0.1	6,120

Inflow (CFS)	Time (hours)	Volume (Ft^3)
18	0.1	6,480
24	0.1	8,640
27	0.1	9,720
30	0.1	10,800
31	0.1	11,160

135,360 TOTAL

- Determine time to pump out the volume collected

$V = 135,360\ ft^3$

$Rate_{min} = 40\ gpm = 0.089\ cfs$

$$\text{Maximum time} = \frac{V}{R} = \frac{135,360\ ft^3}{\frac{0.089\ ft^3}{sec}} \left| \frac{1\ hour}{3600\ sec} \right| \frac{day}{24\ hr} = 17.6\ days \text{ for one 40 gpm pumps}$$

∴ Additional pumps may be added as necessary to expedite the discharge process to facilitate operational needs. This situation represents a worst case situation, when this cell has no waste in it and its struck with the 25-year, 24-hour storm. All liquids can be pumped as stormwater. If partial filling within the cell floor occurs prior to the storm, the volume available for water collection will decrease, thereby decreasing the time to drain the cell.

****
*
* Piedmont Landfill and Recycling Center *
* Kernersville, Forsyth County, North Carolina *
* Phase 3 - Cell 2 - Subcell 1 *
* 24 hr - 25 yr Storm Storage *
*
****

Inflow Hydrograph: c:PIED2 .HYD
Rating Table file: c:PIED2 .PND

----INITIAL CONDITIONS----
Elevation = 757.00 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA

ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)
757.00	0.0	0.000
758.00	0.0	0.028
760.00	0.0	0.439
762.00	0.0	2.210
762.90	0.0	3.929

INTERMEDIATE ROUTING COMPUTATIONS

2S/t (cfs)	2S/t + O (cfs)
0.0	0.0
6.9	6.9
106.1	106.1
534.8	534.8
950.8	950.8

Time increment (t) = 0.100 hrs.

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	1.00	-----	0.0	0.0	0.00	757.00
11.100	1.00	2.0	2.0	2.0	0.00	757.29
11.200	1.00	2.0	4.0	4.0	0.00	757.58
11.300	1.00	2.0	6.0	6.0	0.00	757.87
11.400	1.00	2.0	8.0	8.0	0.00	758.02
11.500	2.00	3.0	11.0	11.0	0.00	758.08
11.600	2.00	4.0	15.0	15.0	0.00	758.16
11.700	2.00	4.0	19.0	19.0	0.00	758.24
11.800	3.00	5.0	24.0	24.0	0.00	758.35
11.900	3.00	6.0	30.0	30.0	0.00	758.47
12.000	6.00	9.0	39.0	39.0	0.00	758.65
12.100	10.00	16.0	55.0	55.0	0.00	758.97
12.200	18.00	28.0	83.0	83.0	0.00	759.53
12.300	27.00	45.0	128.0	128.0	0.00	760.10
12.400	31.00	58.0	186.0	186.0	0.00	760.37
12.500	30.00	61.0	247.0	247.0	0.00	760.66
12.600	24.00	54.0	301.0	301.0	0.00	760.91
12.700	17.00	41.0	342.0	342.0	0.00	761.10
12.800	13.00	30.0	372.0	372.0	0.00	761.24
12.900	10.00	23.0	395.0	395.0	0.00	761.35
13.000	8.00	18.0	413.0	413.0	0.00	761.43
13.100	7.00	15.0	428.0	428.0	0.00	761.50
13.200	6.00	13.0	441.0	441.0	0.00	761.56
13.300	5.00	11.0	452.0	452.0	0.00	761.61
13.400	4.00	9.0	461.0	461.0	0.00	761.66
13.500	4.00	8.0	469.0	469.0	0.00	761.69
13.600	4.00	8.0	477.0	477.0	0.00	761.73
13.700	4.00	8.0	485.0	485.0	0.00	761.77
13.800	3.00	7.0	492.0	492.0	0.00	761.80
13.900	3.00	6.0	498.0	498.0	0.00	761.83
14.000	3.00	6.0	504.0	504.0	0.00	761.86
14.100	3.00	6.0	510.0	510.0	0.00	761.88
14.200	2.00	5.0	515.0	515.0	0.00	761.91
14.300	2.00	4.0	519.0	519.0	0.00	761.93
14.400	2.00	4.0	523.0	523.0	0.00	761.94
14.500	2.00	4.0	527.0	527.0	0.00	761.96
14.600	2.00	4.0	531.0	531.0	0.00	761.98
14.700	2.00	4.0	535.0	535.0	0.00	762.00
14.800	2.00	4.0	539.0	539.0	0.00	762.01
14.900	2.00	4.0	543.0	543.0	0.00	762.02
15.000	2.00	4.0	547.0	547.0	0.00	762.03
15.100	2.00	4.0	551.0	551.0	0.00	762.04
15.200	2.00	4.0	555.0	555.0	0.00	762.04
15.300	2.00	4.0	559.0	559.0	0.00	762.05
15.400	2.00	4.0	563.0	563.0	0.00	762.06

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	2.00	4.0	567.0	567.0	0.00	762.07
15.600	2.00	4.0	571.0	571.0	0.00	762.08
15.700	2.00	4.0	575.0	575.0	0.00	762.09
15.800	2.00	4.0	579.0	579.0	0.00	762.10
15.900	2.00	4.0	583.0	583.0	0.00	762.10
16.000	2.00	4.0	587.0	587.0	0.00	762.11
16.100	2.00	4.0	591.0	591.0	0.00	762.12
16.200	2.00	4.0	595.0	595.0	0.00	762.13
16.300	1.00	3.0	598.0	598.0	0.00	762.14
16.400	1.00	2.0	600.0	600.0	0.00	762.14
16.500	1.00	2.0	602.0	602.0	0.00	762.15
16.600	1.00	2.0	604.0	604.0	0.00	762.15
16.700	1.00	2.0	606.0	606.0	0.00	762.15
16.800	1.00	2.0	608.0	608.0	0.00	762.16
16.900	1.00	2.0	610.0	610.0	0.00	762.16
17.000	1.00	2.0	612.0	612.0	0.00	762.17
17.100	1.00	2.0	614.0	614.0	0.00	762.17
17.200	1.00	2.0	616.0	616.0	0.00	762.18
17.300	1.00	2.0	618.0	618.0	0.00	762.18
17.400	1.00	2.0	620.0	620.0	0.00	762.18
17.500	1.00	2.0	622.0	622.0	0.00	762.19
17.600	1.00	2.0	624.0	624.0	0.00	762.19
17.700	1.00	2.0	626.0	626.0	0.00	762.20
17.800	1.00	2.0	628.0	628.0	0.00	762.20
17.900	1.00	2.0	630.0	630.0	0.00	762.21
18.000	1.00	2.0	632.0	632.0	0.00	762.21
18.100	1.00	2.0	634.0	634.0	0.00	762.21
18.200	1.00	2.0	636.0	636.0	0.00	762.22
18.300	1.00	2.0	638.0	638.0	0.00	762.22
18.400	1.00	2.0	640.0	640.0	0.00	762.23
18.500	1.00	2.0	642.0	642.0	0.00	762.23
18.600	1.00	2.0	644.0	644.0	0.00	762.24
18.700	1.00	2.0	646.0	646.0	0.00	762.24
18.800	1.00	2.0	648.0	648.0	0.00	762.24
18.900	1.00	2.0	650.0	650.0	0.00	762.25
19.000	1.00	2.0	652.0	652.0	0.00	762.25
19.100	1.00	2.0	654.0	654.0	0.00	762.26
19.200	1.00	2.0	656.0	656.0	0.00	762.26
19.300	1.00	2.0	658.0	658.0	0.00	762.27
19.400	1.00	2.0	660.0	660.0	0.00	762.27
19.500	1.00	2.0	662.0	662.0	0.00	762.28
19.600	1.00	2.0	664.0	664.0	0.00	762.28
19.700	1.00	2.0	666.0	666.0	0.00	762.28
19.800	1.00	2.0	668.0	668.0	0.00	762.29
19.900	1.00	2.0	670.0	670.0	0.00	762.29
20.000	1.00	2.0	672.0	672.0	0.00	762.30

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	674.0	674.0	0.00	762.30
20.200	1.00	2.0	676.0	676.0	0.00	762.31
20.300	1.00	2.0	678.0	678.0	0.00	762.31
20.400	1.00	2.0	680.0	680.0	0.00	762.31
20.500	1.00	2.0	682.0	682.0	0.00	762.32
20.600	1.00	2.0	684.0	684.0	0.00	762.32
20.700	1.00	2.0	686.0	686.0	0.00	762.33
20.800	1.00	2.0	688.0	688.0	0.00	762.33
20.900	1.00	2.0	690.0	690.0	0.00	762.34
21.000	1.00	2.0	692.0	692.0	0.00	762.34
21.100	1.00	2.0	694.0	694.0	0.00	762.34
21.200	1.00	2.0	696.0	696.0	0.00	762.35
21.300	1.00	2.0	698.0	698.0	0.00	762.35
21.400	1.00	2.0	700.0	700.0	0.00	762.36
21.500	1.00	2.0	702.0	702.0	0.00	762.36
21.600	1.00	2.0	704.0	704.0	0.00	762.37
21.700	1.00	2.0	706.0	706.0	0.00	762.37
21.800	1.00	2.0	708.0	708.0	0.00	762.37
21.900	1.00	2.0	710.0	710.0	0.00	762.38
22.000	1.00	2.0	712.0	712.0	0.00	762.38
22.100	1.00	2.0	714.0	714.0	0.00	762.39
22.200	1.00	2.0	716.0	716.0	0.00	762.39
22.300	1.00	2.0	718.0	718.0	0.00	762.40
22.400	1.00	2.0	720.0	720.0	0.00	762.40
22.500	1.00	2.0	722.0	722.0	0.00	762.40
22.600	1.00	2.0	724.0	724.0	0.00	762.41
22.700	1.00	2.0	726.0	726.0	0.00	762.41
22.800	1.00	2.0	728.0	728.0	0.00	762.42
22.900	1.00	2.0	730.0	730.0	0.00	762.42
23.000	1.00	2.0	732.0	732.0	0.00	762.43
23.100	1.00	2.0	734.0	734.0	0.00	762.43
23.200	1.00	2.0	736.0	736.0	0.00	762.44
23.300	1.00	2.0	738.0	738.0	0.00	762.44
23.400	1.00	2.0	740.0	740.0	0.00	762.44
23.500	1.00	2.0	742.0	742.0	0.00	762.45
23.600	1.00	2.0	744.0	744.0	0.00	762.45
23.700	1.00	2.0	746.0	746.0	0.00	762.46
23.800	1.00	2.0	748.0	748.0	0.00	762.46
23.900	1.00	2.0	750.0	750.0	0.00	762.47
24.000	0.00	1.0	751.0	751.0	0.00	762.47
24.100	0.00	0.0	751.0	751.0	0.00	762.47
24.200	0.00	0.0	751.0	751.0	0.00	762.47
24.300	0.00	0.0	751.0	751.0	0.00	762.47
24.400	0.00	0.0	751.0	751.0	0.00	762.47
24.500	0.00	0.0	751.0	751.0	0.00	762.47
24.600	0.00	0.0	751.0	751.0	0.00	762.47

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

INFLOW HYDROGRAPH

TIME (hrs)	INFLOW (cfs)
24.700	0.00
24.800	0.00
24.900	0.00
25.000	0.00
25.100	0.00
25.200	0.00
25.300	0.00
25.400	0.00
25.500	0.00
25.600	0.00
25.700	0.00
25.800	0.00
25.900	0.00

ROUTING COMPUTATIONS

I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47
0.0	751.0	751.0	0.00	762.47

********** SUMMARY OF ROUTING COMPUTATIONS **********

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

Starting Pond W.S. Elevation = 757.00 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow	=	31.00 cfs
Peak Outflow	=	0.00 cfs
Peak Elevation	=	762.47 ft

***** Summary of Approximate Peak Storage *****

Initial Storage	=	0.00 ac-ft
Peak Storage From Storm	=	3.10 ac-ft
Total Storage in Pond	=	3.10 ac-ft

Warning: Inflow hydrograph truncated on left side.

>>>>>> Warning, peak outflow = last ordinate point. <<<<<<

Pond File: c:PIED2 .PND
Inflow Hydrograph: c:PIED2 .HYD
Outflow Hydrograph: c:POND2 .HYD

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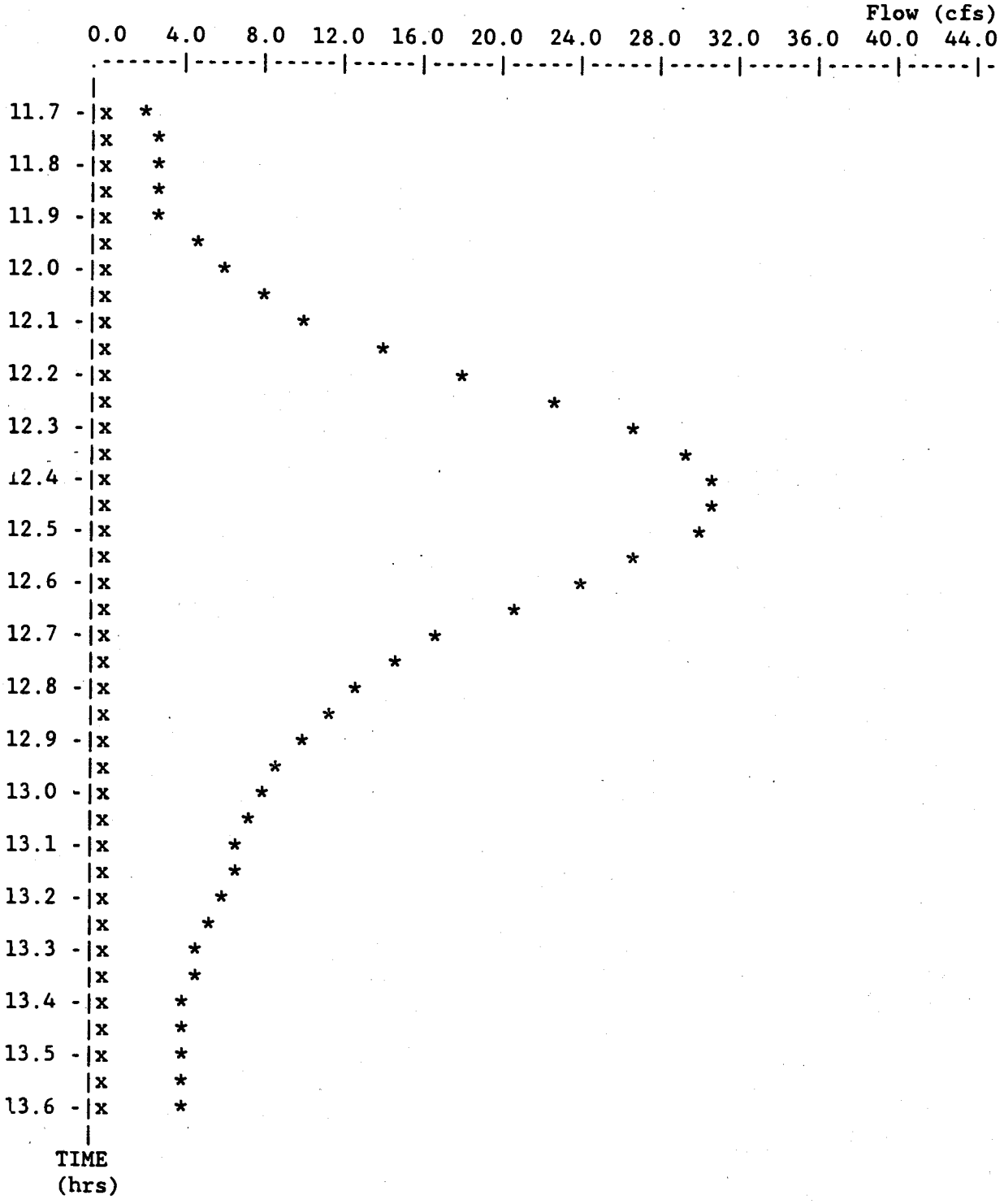
11:40:28

Peak Inflow = 31.00 cfs
Peak Outflow = 0.00 cfs
Peak Elevation = 762.47 ft



```
* File:  c:PIED2    .HYD   Qmax =     31.0 cfs
x File:  c:POND2    .HYD   Qmax =      0.0 cfs
```

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 1
PROJECT NO. 87221

CLIENT Piedmont LRC
PROJECT Transition Application + Permit Renewal
SUBJECT Leachate Collection System Sump Volume
Prepared By AMM Date 3/17/19
Reviewed By LMW Date 3/30/19
Approved By ____ Date ____

Purpose: To determine the liquid storage capacity of the leachate collection sump at the Piedmont LRC.

Givens & Assumed:

- The leachate collection sump plan view is attached (Attachment 1).
- Sump depth = 2.5' from top of sump to base.
- Sump riser trench depth = 2.0'
- Assumed porosity of stone = $n = .35 = \frac{V_{voids}}{V_{total}}$
- Volume of leachate collection trench below the top of sump will not be included.
- "Open Area" of leachate collection pipes will be corrected for.

Calculations:

- Sump Bottom Area = $\underbrace{(12')(10')}_{\text{Sump Bottom}} + \underbrace{(6')(6') + 2\left(\frac{1}{2}\right)(6')(2')}_{\text{Sump Bottom @ Riser Trench}} = 168\ Ft^2$

- Sump Top Area = $[(12'+(2)(3)(2.5')][(10'+(2)(3)(2.5')] + (6')(6') + 2(\frac{1}{2})(6')(2') = 723\ Ft^2$

- Using the pyramid volume equation:

$$\text{Gross Volume} = V_g = \frac{1}{3}\left[A_1 + A_2 + (A_1 \cdot A_2)^{1/2}\right] h$$
$$= \frac{1}{3}\left[168 + 723 + (168 \cdot 723)^{1/2}\right] 2.5$$
$$V_g = 1032.93\ Ft^3$$
$$V_n = (1032.93 Ft^3)(.35) = 361.5\ Ft^3 \text{ (Volume of Voids)}$$

of slope

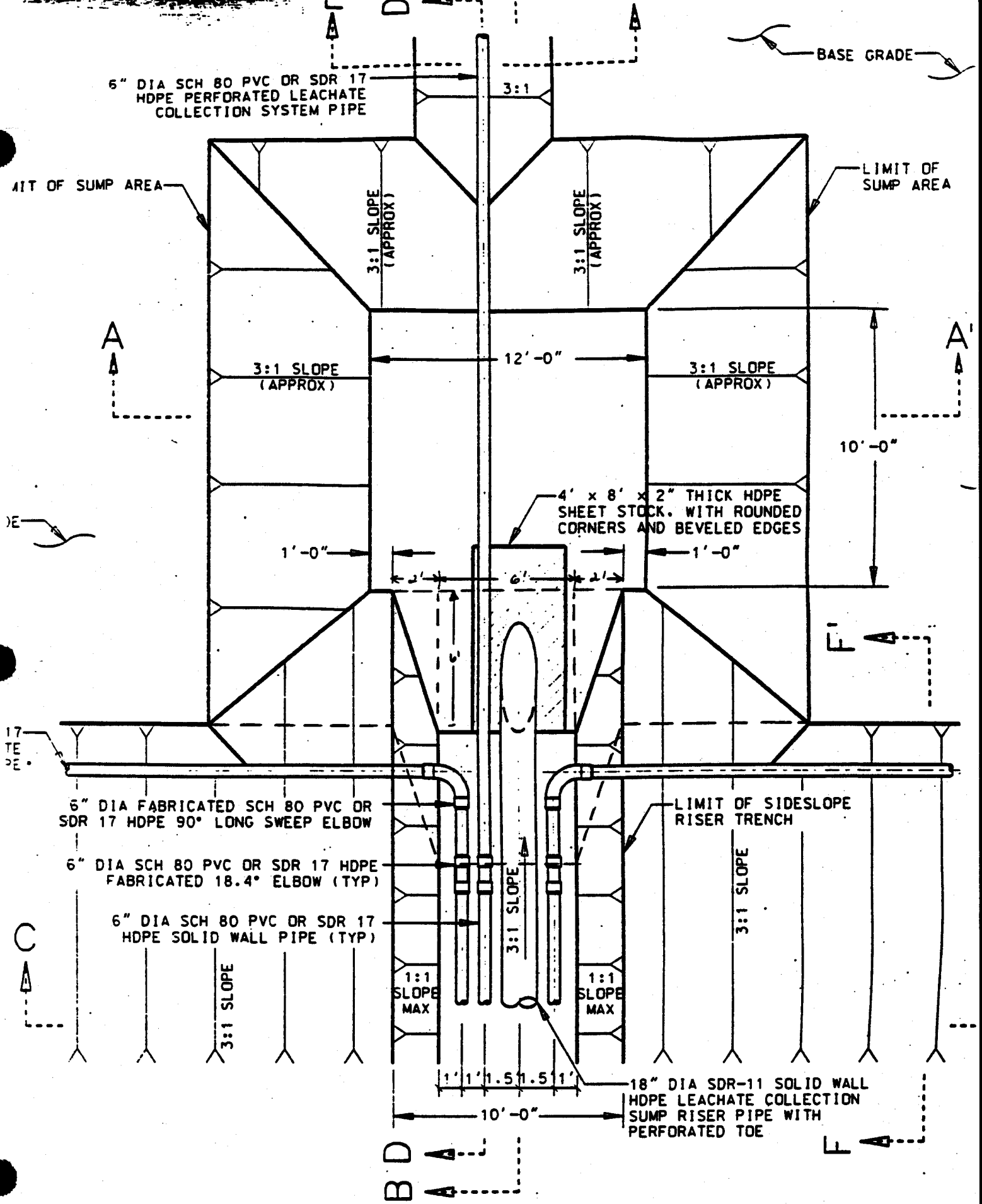
- Correct for "open area" of 6" ϕ and 18" ϕ pipes (only 6" ϕ pipe across the sump, not at t

$$\text{Pipe Volume} = \pi(0.5)^2(.25)(30') + \pi(1.5)^2(.25)(7.5') = 19.1\ Ft^3$$
$$(19.1 Ft^3)(.65) = 12.4\ Ft^3 \text{ (remaining "open" volume not accounted for due to p}$$

- Final Net Volume = $V_N = 361.5\ Ft^3 + 12.4\ Ft^3 = 373.9\ Ft^3$

- Leachate Storage = $(373.9\ ft^3)\left(\frac{7.48 \text{ gallons}}{Ft^3}\right) = 2,796.8$ gallons ⇒ Say 2,800

- Storage Volume is 2,800 gallons minimum, and will only increase as a result of t additional storage in the l.c.s. trenches below sump top, and pipes along toe of



ATTACHMENT 1

APPENDIX K

Protective Cover Stability Analysis

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

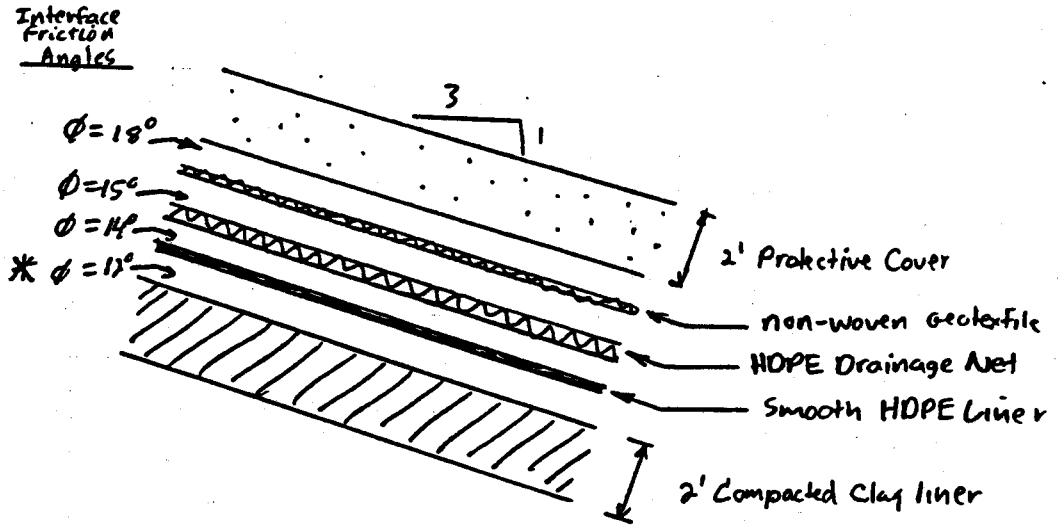
PAGE 1 OF 1
PROJECT NO. 87221.200

CLIENT WMT
PROJECT Piedmont Landfill

SUBJECT Stability of Soil Cover on Liner Side Slopes

Prepared By I.Y. Date 10/21/9
Reviewed By RJF Date 11/8/9
Approved By ____ Date ____

Side Slope Liner Configuration



* Critical Side-Slope liner friction angle is 12° between smooth 60 mil HDPE liner and compacted clay liner.

Calculate Max. Allowable Height of Protective Cover Without Putting Tension on Geosynthetic Layers (Hmax)

Use method of Giroud and Beech, 1989 "Stability of Soil Layers on Geosynthetic Lining Systems".

List of Parameters

T_c = thickness of soil cover = 2 ft

β = angle of slope = $\tan^{-1}\frac{1}{3} = 18.4°$

ϕ_c = friction angle of protective soil cover = 30°

ϕ_i = Min. interface friction angle of lining system = 12°

From Eq. 9 of paper:

$$H_{max} = \frac{T_c}{2\cos\beta}\left(1 + \frac{\sin\phi_c \cos\phi_i}{\cos(\beta+\phi_c)\sin(\beta-\phi_i)}\right)$$

$$= \frac{2}{2\cos 18.4°}\left(1 + \frac{\sin 30° \cos 12°}{\cos(18.4° + 30°)\sin(18.4° - 12°)}\right)$$

H_{max} = 8.02 feet ← Caution!! Leachate Management Plan indicates height of 3:1 slope exceeds 8 feet in some portions of toe berm and internal divider dikes.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 1
PROJECT NO. 87221.400

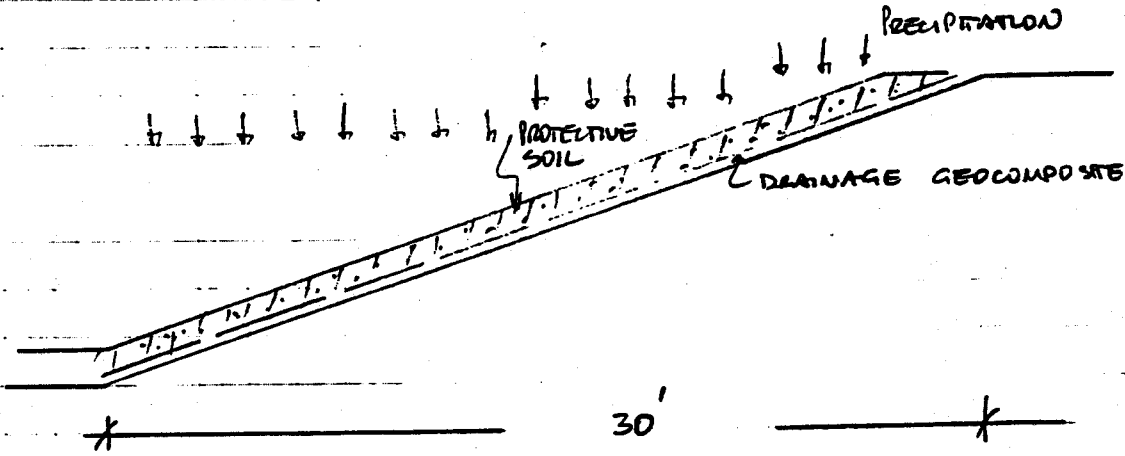
CLIENT PIEDMONT LANDFILL
SUBJECT SIDE LINER STABILITY
PROJECT EXPANSION

Prepared By ESM Date 3/28/94
Reviewed By RJM Date 3/29/94
Approved By _____ Date _____

PURPOSE : DETERMINE THE DRAINAGE ADEQUACY OF THE GEONET ON THE SIDESLOPE.

INFILTRATION REMOVAL CAPACITY OF DRAINAGE LAYER

ASSUMPTION : 5-YR 24-HR STORM EVENT THAT RESULTS IN 4.5 INCHES OF PRECIPITATION.



HYDRAULIC TRANSMISSIVITY OF GEONET-GEOTEXTILE COMPOSITE $\simeq 0.001\ m^2/s$ (0.0107 ft^2/s)

ASSUME HYDRAULIC GRADIENT $\simeq$ SIDE SLOPE = 0.333

FLOW CAPACITY OF DRAINAGE GEOCOMPOSITE

$$q_f = 0.0107 \times 0.333 \times 86400 = 307\ ft^3/ft/day$$

A 5-YR 24-HR STORM WILL RESULT IN

$$q_s = \frac{4.5}{12} \times 30 = 11.25\ ft^3/ft/day$$

THEREFORE THE FACTOR OF SAFETY IS:

$$FS = \frac{307}{11.25} = 27.3$$

APPENDIX L

Leachate Collection Pipe Strength Calculation

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE 1 OF 6
PROJECT NO. 87221.600

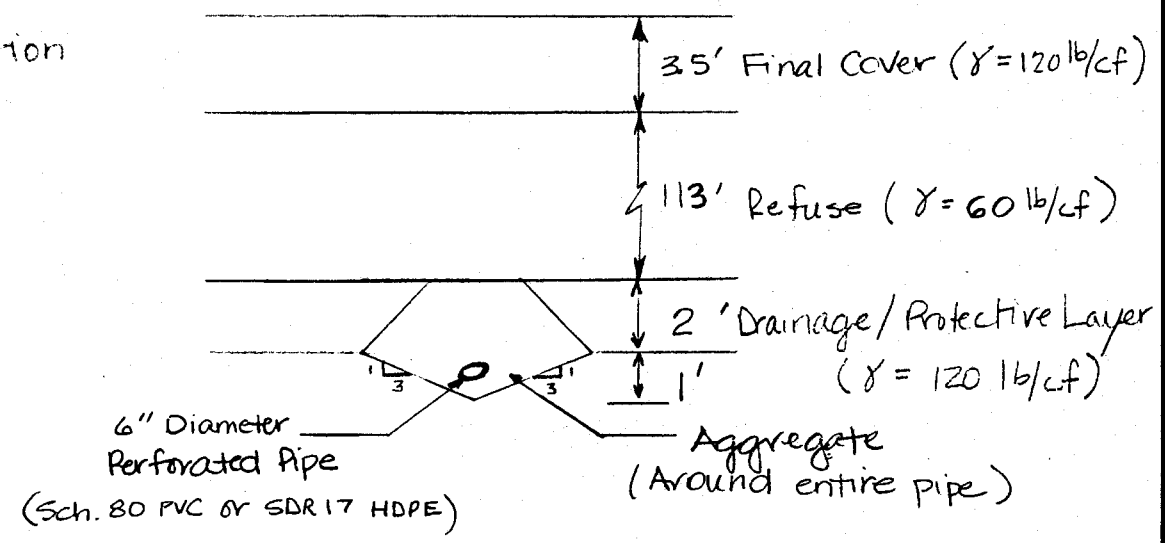
CLIENT Waste Management SUBJECT Leachate Collection Pipe Strength Prepared By AGF Date 4/6/94
PROJECT Piedmont Landfill Reviewed By [illegible] Date 4/8/94
Approved By ____ Date ____

Purpose: Analyze the performance of the leachate collection pipe to resist failure by the following modes:

1) wall crushing
2) wall buckling
3) ring deflection

Assumptions/Givens:

Cross-Section



References:

1. NIBCO - Chemical Plastic Piping Handbook
2. Phillips 66 - Driscopipe Product Information
3. "Modulus of Soil Reaction Values for Buried Flexible Pipe," A.K. Howard, Journal of the Geotechnical Engineering Division, January 1977.
4. Handbook of PVC Pipe, Unibell.
5. Plastic Piping Systems, Volume 1, 1980.
6. Polypipe Industries Product Information
7. Chapter 4.0, Subpart D, "Design Criteria," of April 1992 Draft EPA Design Manual.

Equations for Analyses:

1) Wall Crushing (Reference 2)

$$S_A = \frac{(SDR-1)}{2} P_T$$

where: S_A = Actual Compressive Stress (psi)
SDR = Standard Dimension Ratio
P_T = External Pressure (psi)

$$\text{Safety Factor} = \frac{\text{Compressive Yield Stress}}{S_A}$$

2) Wall Buckling (Reference 2)

$$P_{cb} = 0.8 \sqrt{E' \times P_c}$$

where: P_T = Total Vertical Soil Pressure at the top of the pipe (psi)
P_{cb} = Critical Buckling Soil Pressure at the top of the pipe (psi)
E' = Soil Modulus in psi calculated as the ratio of the vertical soil pressure to the vertical soil strain at a specified density.
P_c = Hydrostatic, critical-collapse differential pressure (psi)

$$P_c = \frac{2.32 \; E \; F_R}{(SDR)^3}$$

where: E = stress and time dependent tensile modulus of elasticity (psi)
F_R = multiplier for temperature derating external pressure differential

$$\text{Safety Factor} = \frac{P_{cb}}{P_T}$$

3) Ring Deflection (Reference 4)

$$\% \frac{\Delta Y}{D} = \frac{D_L K P (100)}{[2E/3 (SDR-1)^3] + 0.061 E'}$$

Where: D_L = Deflection Lag Factor ⇒ Assume $D_L = 1.0$
K = Bedding Constant ⇒ Assume $K = 0.1$
P = Prism Load (Soil Pressure), psi

Pipe Materials Characteristics / Variables:

1) HDPE Pipe, SDR = 17

Nominal Diameter = 6"
Min. Wall Thickness = 0.390" (Ref. 6)
Outside Diameter = 6.625" (Ref. 6)
Compressive Yield Strength = 1500 psi (Ref. 2)
F_R = 0.87 @ 90°F (Interpolated - Ref. 2)
E = 30,000 psi, Long Term (Ref. 6)
→ Should provide a conservative estimate for effects of time and temperature
[E = 100,000 psi @ 100°F (Ref. 2); E = 113,000 psi (Ref. 6)]

Allowable Ring Deflection = 4.2% (Ref 2)

2) PVC Pipe, Sch. 80

Nominal Diameter = 6"
Min. Wall Thickness = 0.432" (Ref. 4)
Outside Diameter = 6.625" (Ref. 4)
SDR = 6.625" / 0.432" = 15.34
Operating temperature = 100°F
F_R = 0.62 @ 100°F (Ref. 5)
E = 400,000 psi
Working Stress = 2000 psi @ 73.4°F (Ref. 5)
→ use as compressive yield strength in analysis
Recommended limit (conservative) for $\frac{\%Y}{D}$ = 7.5% (Ref. 4, P. 183)

General:

Load, P_T = (6 ft)(120 lb/cf) + (113 ft)(60 lb/cf) = 7500 psf

P_T = 52 psi ⟹ 52 psi x 1.075 = 56 psi *

E' ~ 3000 psi (Crushed Rock - Ref. 2)

* Increase pressure by 7.5% due to assumed vertical acceleration component (From Seismic Study.)

Calculations :

1) HDPE Pipe , SDR = 17 :

A) Wall Crushing

$$S_A = \frac{(SDR-1)}{2} P_T = \frac{(17-1)}{2}(56\ psi) = \underline{448\ psi} < 1500 \quad OK$$

$$FS = \frac{1500}{448} = \underline{3.35} > 3.0 \quad \therefore OK$$

B) Wall Buckling

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

$$P_c = \frac{2.32\ E\ F_R}{SDR^3} = \frac{(2.32)(30{,}000\ psi)(0.87)}{(17)^3} = 12.3\ psi$$

$$P_{cb} = 0.8\sqrt{(3000\ psi)(12.3\ psi)} = \underline{153.7\ psi}$$

$$FS = \frac{153.7\ psi}{56\ psi} = \underline{2.74} > 2 \quad \therefore OK$$

c) Ring Deflection

$$\frac{\%Y}{D} = \frac{D_L K P (100)}{[\text{~~2E~~}/3(SDR-1)^3] + 0.061 E'}$$

($2EF_R$)

$$= \frac{(1.0)(0.1)(56\ psi)(100)}{[2(87)(30{,}000)/(3(17-1)^3)] + 0.061(3000)}$$

$$= \frac{560}{4.25\ psi + 183\ psi} = \underline{2.99\%} < 4.2\% \quad \therefore OK$$

2) PVC Pipe, Sch. 80

A) Wall Crushing

$$S_A = \frac{(SDR-1)}{2} P_T = \frac{(15.34-1)}{2}(56\ psi) = \underline{401.5\ psi}$$

$$FS = \frac{2000\ psi}{401.5\ psi} = \underline{4.98} > 3 \quad \therefore OK$$

B) Wall Buckling

$$P_{cb} = 1.15\sqrt{E' \times P_c}$$

$$P_c = \frac{2E\,FR}{(1-.38^2)(SDR-1)^3} = \frac{(2)(400{,}000\ psi)(.62)}{(1-.38^2)(15.34-1)^3} = 196.6\ psi$$

$$P_{cb} = 1.15\sqrt{(3000\ psi)(196.6\ psi)} = \underline{883.16\ psi}$$

$$FS = \frac{883.16\ psi}{56\ psi} = \underline{15.8} > 2 \quad \therefore OK$$

C) Ring Deflection

$$\%\frac{Y}{D} = \frac{D_L K P (100)}{[2E/(3(SDR-1)^3)] + 0.061 E'}$$

$$= \frac{(1.0)(0.1)(56\ psi)(100)}{[2(.62\times 400{,}000)/(3(15.34-1)^3)] + 0.061(3000\ psi)}$$

$$= \frac{560\ psi}{56.1\ psi + 183\ psi} = \underline{2.34\% < 7.5\%} \quad \therefore OK$$

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE ____ OF ____

PROJECT NO. _____

CLIENT _____ SUBJECT _____ Prepared By _____ Date ____

PROJECT _____ _____ Reviewed By _____ Date ____

_____ _____ Approved By _____ Date ____

REFERENCE 1

Valves, Unions and Flanges

As with all other thermoplastic piping components, the maximum nonshock operating pressure is a function of temperature.

Maximum pressure rating for Chemtrol valves, unions and flanges is 150 psi. Above 100° F refer to the chart.

The 10" butterfly valve has a rating of ½ the values shown in the chart.

The 6" butterfly valve has a rating of ¾ the values shown in the chart.

Maximum Operating Pressure (psi)

Operating Temp. °F.	PVC	CPVC	PP	PVDF
100	150	150	150	150
110	135	140	140	150
120	110	130	130	150
130	75	120	118	150
140	50	110	105	150
150	N.R.	100	93	140
160	N.R.	90	80	133
170	N.R.	80	70	125
180	N.R.	70	50	115
190	N.R.	60	N.R.	106
200	N.R.	50	N.R.	97
250	N.R.	N.R.	N.R.	50
280	N.R.	N.R.	N.R.	25

N.R. – Not Recommended.

EXPANSION AND CONTRACTION OF PLASTIC PIPE

Calculating Stress

Plastics, like other piping materials, undergo dimensional changes as a result of temperature variations above and below the installation temperature. If movement resulting from these dimensional changes is restricted by adjacent equipment or by a vessel to which the pipe may be rigidly attached, the resultant stresses and forces may cause damage to such items or even to the pipe itself. In a special case, where compensation is not provided for these dimensional changes and where the piping system is rigidly held or restricted at both ends, an estimate of the magnitude of the resultant stresses can be obtained with the following formula. This formula relates the temperature differential to the temperature dependent modulus (See Table 1) and the expansion coefficient for the particular plastic material.

$S = EC(T_1-T_2)$

S = Stress (psi)

E = Modulus of Elasticity (psi) (See table below for specific values at various temperatures)

C = Coefficient of Expansion ($in/in/°F \times 10^{-5}$) (see physical property chart on page 14 for values)

T_1-T_2 = Temperature differential between the installation temperature and the maximum or minimum system temperature.

Table 1

Temperature Vs. Modulus (x 10^5) psi

Temperature °F	73	90	100	140	180	210	250
PVC	4.20	3.75	3.60	2.70	N/A	N/A	N/A
CPVC	4.23	4.00	3.85	3.25	2.69	2.20	N/A
PP	1.79	1.25	1.15	.72	.50	N/A	N/A
PVDF	2.19	1.88	1.74	1.32	1.12	.81	.59

N/A — Not Applicable.

RUST ENVIRONMENT & INFRASTRUCTURE

CALCULATION SHEET

PAGE ____ OF ____

PROJECT NO. _____

CLIENT _____ SUBJECT _____ Prepared By _____ Date ____

PROJECT _____ _____ Reviewed By _____ Date ____

_____ _____ Approved By _____ Date ____

REFERENCE 2

DRISCOPIPE 1000 series

Typical Physical Properties⊖

Property	Specification	Unit	Nominal Value
Material Designation	PPI / ASTM		PE 3408
Material Classification	ASTM D-1248		III C 5 P34
Cell Classification	ASTM D-3350		3 4 5 4 3 4 C
Density (3)	ASTM D-1505	gm/cm^3	0.955
Melt Index (4)	ASTM D-1238 (216 kg/190°C)	gm/10 min.	0.11 ♦
Flex Modus (5)	ASTM D-790	psi	135,000
Tensile Strength (4)	ASTM D-638	psi	3,200
ESCR (3)	ASTM D-1693	Fo, Hours	> 5,000**
HDB @73° F (4)	ASTM D-2837	psi	1,600
U-V Stabilizer (C)	ASTM D-1603	% C	2.5
Hardness	ASTM D-2240	Shore "D"	65
Compressive Strength (yield)	ASTM D-695	psi	1,600
Tensile Strength @ Yield (Type IV Spec.)	ASTM D-638 (2" / min.)	psi	3,200
Elongation @ Yield	ASTM D-638	%, minimum	8
Tensile Strength @ Break (Type IV Spec.)	ASTM D-638	psi	5,000
Elongation @ Break	ASTM D-638	%, minimum	750
Modulus of Elasticity	ASTM D-638	psi	130,000
ESCR			
(Cond. A,B,C: Mold. Slab)	ASTM D-1693	Fo, Hours	> 5,000
(Compressed Ring - pipe)	ASTM F-1248	Fo, Hours	> 3,500
Slow Crack Growth	Battelle Method	Days to Failure	> 64
Impact Strength (IZOD) (.125" Thick)	ASTM D-256 (Method A)	In-lb / in notch	42
Linear Thermal Expansion Coef.	ASTM D-696	in / in /°F	1.2 x 10^{-4}
Thermal Conductivity	ASTM D-177	BTU-in/ft^2/hrs/°F	2.7
Brittleness Temp.	ASTM D-746	° F	< -180
Vicat Soft. Temp.	ASTM D-1525	° F	257
Heat Fusion Cond.		psi @ ° F	75 @ 400

⊖ This list of typical physical properties is intended for basic characterization of the material and does not represent specfic determinations of specifications. The physical properties values reported herein were determined on compression molded specimens prepared in accordance with Procedure C of ASTM D 1928 and may differ from specimens taken from pipe.

** Tests were discontinued because no failures and no indication of stress crack initiation.

♦ Average Melt Index value with a standard deviation of 0.01

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Note: Beyond 122.8 inches (10.2 ft.) the soil friction will overcome the tensile force developed by thermal contraction of the pipeline. This is calculated by dividing the tensile force in the pipe by the frictional resistance of the soil (ie: 781.3 lbs. ÷ 6.363 lbs./in. = 122.8 Inches)

Theoretical Movement of *Unrestrained* Ends:

$\Delta L = L \cdot \varepsilon$

ΔL = (122.8 ins.)(.0024 in./in.) = .295 in.

Design of Collar:
By sidewall fusing branch saddles capable of taking the shear force onto the pipe near the terminal connection, and then pouring a square concrete collar around the pipe and branch saddles into undisturbed soil, the tensile force of 781.3 lbs. is removed from the pipe connection and is evenly distributed into the soil. Assume a collar 12 inches square and 6 inches wide is used.

Area of Collar = (12" × 12") – (Cross Sectional Area of Pipe)

A = (144 – 16) sq.in.

A = 128 sq. in. surface area

Compressive Stress on soil due to load transfer by collar face:

S = F ÷ A

S = 71.3 lbs. ÷ 128 sq. in. = 6.1 psi

EXAMPLE DESIGN SUMMARY

Under a 20°F instantaneous temperature change, a 4" SDR 15.5 pipeline 1000 ft. long buried five feet deep will try to change length .295" at each end. The pipe is restrained by soil friction from further contraction. A concrete collar with a square face of 12" × 12" will absorb the tensile force of 781.3 lbs. due to thermal contraction, and distribute it into the soil at a compressive soil stress of 6.1 psi.

Circumferential coefficient of expansion is 0.6×10^{-4} in/in/°F.

Chart 16

Driscopipe Pressure Rating (psi) vs. Temperature (°F)

Temp. °F	Hoop Stress (psi)	Pipe SDR 32.5	26	21	19	17	15.5	13.5	11	9.3
50	1820	58	73	90	100	113	125	145	180	215
60	1730	55	69	86	96	108	119	138	170	207
73.4	**1600**	**51**	**64**	**80**	**90**	**100**	**110**	**128**	**160**	**190**
80	1520	48	60	76	85	95	105	122	150	182
90	1390	44	56	70	77	87	96	111	140	167
100	1260	40	50	63	70	79	87	101	125	150
110	1130	36	45	57	63	71	78	90	113	135
120	1000	32	40	50	56	63	69	80	100	120
130	900	28	36	45	50	56	62	72	90	108
140	800	25	32	40	45	50	55	64	80	96

Chart 17

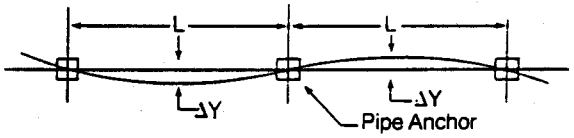
Instantaneous Modulus of Elasticity vs. Temperature

140°F	50,000 psi
100°F	100,000 psi
73.4°F	110,000 psi
50°F	165,000 psi
32°F	200,000 psi
0°F	260,000 psi
–20°F	300,000 psi

If flow is likely to be stopped, the designer may want to provide a means to drain the pipeline partially or completely. If, for some unforeseen reason, the Driscopipe pipeline should plug and freeze, the pipeline will not be damaged. The frozen fluid may swell the diameter of the pipe but it will return to nominal size as the fluid thaws. Due to the nature of polyethylene pipe, a flame (such as a propane or acetylene torch) cannot be used to thaw a frozen section of pipe. Other methods must be used.

The toughness and excellent abrasion resistance of Driscopipe will take the abuse of movement across sand and soil without detrimental effects on its strength or service life. However, in rocky areas, sharp rocks which could cut the pipe should be removed and may be replaced with a bed of sand or soil.

Lateral Deflection Due to Thermal Movement in Overland Pipelines.



The following formulae will allow the designer to calculate lateral deflection of the pipeline and anchor point spacing.

$$\Delta Y = L\sqrt{.50\alpha\Delta T} \qquad (1)$$

$$L = \frac{\Delta Y}{\sqrt{.50\alpha\Delta T}} \qquad (2)$$

$$L = \frac{D\sqrt{96\alpha\Delta T}}{\varepsilon} \qquad (3)$$

Where: ΔY = Lateral deflection (inches)
L = Length of pipe between anchors (inches)
α = Coefficient of thermal expansion (in/in/°F)
ΔT = Change in temperature (°F)
ε = Strain (inches/inch, or percent)
D = Pipe outside dia. (inches)

Examination of equation (1) or (2) shows that for any given set of thermal conditions an increase in ΔY will increase L and vice versa. Increasing ΔY and L to the maximum will reduce the number of anchor points needed but may increase wear on the pipe from movement and may increase the possibility of kinking the line if lateral movement does not occur uniformly. One practical approach to design is to calculate L using formula (3) for strain (ε) in the pipe wall equal to 1% and (ε) equal to 5%. The L value at 5% strain will give the shortest distance between anchor points and should be considered maximum for strain (ε) and minimum spacing for L. The spacing for L should be as large as possible considering other installation location factors, such as available right-of-way, slope of the ground, etc. Higher values for L mean less strain (ε) and fewer anchor points and, consequently, lower costs, generally.

Type 3: Buried Pipelines

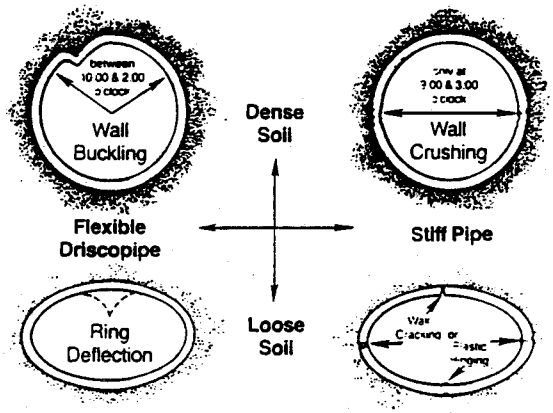
Introduction: When pipelines are buried, they are subjected to external loads. The effect of external pressure on flexible Driscopipe is more complex than the effect of internal pressure only. For design purposes, a distinction is usually made between rigid and flexible pipes. A rigid pipeline (such as concrete) is considered to be the total structure and must be designed to sustain all external loads as well as internal pressure. But, Driscopipe is a flexible pipe and is considered to be only one component of the "pipe-soil" system, as described more fully on page 35.

Thus, in a buried situation, the SDR of the pipe and the strength of the soil envelope must be specified in order to keep the three burial design parameters (wall crushing, wall buckling and ring deflection) within acceptable limits. The pipe and soil envelope become one system. The mutual interaction and strength contribution of the pipe to the soil and the soil to the pipe result in a highly successful integral structure. Correct design centers around two points: a) matching the proper wall thickness to the external soil pressure and b) the analysis of how Driscopipe and the soil surrounding the pipe accept the backfill earthloading and transfer it to the undisturbed walls of the ditch or trench such that the pipeline will deflect slightly into static equilibrium with the soil.

Design Considerations: Driscopipe performs effectively when buried because of its ability to deflect in the interacting pipe-soil system. The pipe actually gains strength from the surrounding soil and can support additional soil loads. The soil envelope surrounding the pipe will compress or deflect slightly under the additional loads of the dry weight of backfill above the pipe, the weight of the water table saturating the soil, the weight of nearby buildings or static structures, the weight of off-the-road trucks and tractors, railroad freight trains or any combination of all these loads. In a flexible "pipe-soil" system, the pipe deflection is the same as the soil deflection under the load of the secondary backfill and any surface loads.

When the pipeline is laid in the trench, the soil envelope is shoveled, washed, flushed or dumped into place, in layers, to an elevation above the top of the pipe. It is usually compacted to a specified Proctor density by mechanical or hydraulic means. As additional layers of soil are backfilled into the trench, the weight of the soil over the primary, compacted backfill is increased. This increased weight slightly compresses (strains downwards) the soil envelope. The amount of soil compression is related to the strength of the soil at a given density and to the intensity of the vertical soil pressure. Since the soil is not an elastic material, this compression or strain of the soil envelope by the final backfill is permanent. As the soil envelope is compressed (strained), it becomes further compacted. In its denser state, the soil develops increased resistance to the vertical soil pressure until it reaches static equilibrium without further compression or strain. This is the reaction of the compacted soil envelope.

Typical Performance Limits of Buried Pipes Due to External Soil Pressure

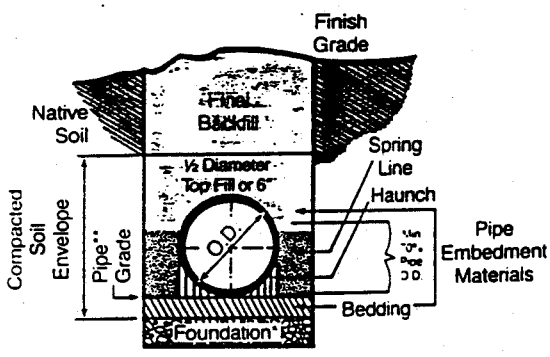


Through mathematical computations presented later, the ring deflection of Driscopipe can be calculated based upon the known properties of the pipe and the measured compressibility of the soil envelopes. As the pipe deflects with the soil, it forms a very slight ellipse by decreasing in the vertical diameter an amount ΔY and by increasing in the horizontal diameter an almost equal (but slightly less) amount ΔX. The horizontal diametrical increase is beneficial because it further compacts the sidefill soil and thus develops lateral support in a fashion similar to the way soil abutments offer support to a masonry arch bridge. The compacted soil actually becomes a masonry arch even though the soil is a low grade masonry using soil particles for bricks. The vertical decrease in diameter is beneficial because it relieves the pipe of vertical soil pressure concentrations and thus forces the soil to support the major share of the vertical load by arching action over the pipe.

Compressibility of the soil envelope is measured by the Soil Modulus, E', which is the ratio of soil pressure (stress) to soil deflection (strain) at a given soil compaction density, as shown in Chart 26. The degree of compaction of the soil envelope can be determined from a family of curves developed from laboratory tests. Chart 26 is typical of such curves and provides a general guide if laboratory data are not available to plot a family of curves for a specific soil type.

Deformation of buried flexible pipe becomes critical only when the pipe reaches that point of ring deflection beyond which it can no longer resist any increase in soil loading. By limiting ring deflection through proper soil compaction, the loading over a pipe is distributed through the soil and across the soil arch around the pipe. Thus the pipeline maintains its circular cross-section in stable soils and is protected against long term collapse.

Trench Construction and Terminology



***In suitable soils the construction of a bedding and/or a foundation may not be required.**

****Gravity flow Driscopipe sewers may be accurately laid to grade using laser-beam scoping.**

Simplified Burial Design: A conservative estimate of the ability of Driscopipe pipelines to perform in a buried environment is found in Chart 24. It is based on a minimum 2:1 safety factor and 50 year design service life. A detailed burial design starts on page 37. The detailed design should be used for critical or marginal applications or whenever a more precise solution is desired.

Detailed Burial Design:
Design by Wall Crushing: Wall crushing would theoretically occur when the stress in a pipe wall, due to the external vertical pressure, exceeded the long-term compressive strength of the pipe material. To ensure that the Driscopipe wall is strong enough to endure the external pressure the following check should be made:

$$S_A = \frac{(SDR - 1)}{2} P_T$$

Values of E'

Based on Soil Type (ASTM D2321) and Degree of Compaction

Soil Type of Initial Backfill Embedment Material	Description	E' (psi) for Degree of Compaction (Proctor Density, %)			
		Loose	Slight (70-85%)	Moderate (85-95%)	High (95%)
I	Manufactured angular, granular materials (crushed stone or rock, broken coral, cinders, etc.)	1,000	3,000	3,000	3,000
II	Coarse grained soils with little or no fines	N.R.	1,000	2,000	3,000
III	Coarse grained soils with fines	N.R.	N.R.	1,000	2,000
IV	Fine-grained soils	N.R.	N.R.	N.R.	N.R.
V	Organic soils (peat, muck, clay, etc.)	N.R.	N.R.	N.R.	N.R.

N.R. = Not Recommended for use by ASTM D2321 for pipe wall support

Chart 24

SDR	Maximum Burial Depth, ft. in dry soil of 100 lbs/cu. ft.			Maximum External Pressure psi			Maximum Deflection, % after installation		
	Soil Modulus, psi*			Soil Modulus, psi*			Soil Modulus, psi*		
	1000	2000	3000	1000	2000	3000	1000	2000	3000
32.5	25	32	37	17	22	26	1.7	0.9	0.6
26	33	45	52	23	31	36	2.3	1.2	0.8
21	46	61	71	32	42	49	3.2	1.6	1.1
19	52	69	81	36	48	56	3.6	1.8	1.2
17	61	121	181	42	84	126	4.2	2.1	1.4
15.5	56	112	168	39	78	117	3.9	2.0	1.3
13.5	49	98	147	34	68	102	3.4	1.7	1.1
11	39	78	117	27	54	81	2.7	1.4	0.9
9.3	33	68	101	23	47	70	2.3	1.2	0.8
8.3	30	61	89	21	42	62	2.1	1.1	0.7
7.3	26	52	79	18	36	55	1.8	0.9	0.6

*assumes no external loads

Where: S_A = Actual compressive stress, psi
SDR = Standard Dimension Ratio
P_T = External Pressure, psi

Safety Factor = 1500 psi ÷ S_A where 1500 psi is the Compressive Yield Strength of Driscopipe.

Design by Wall Buckling: Local wall buckling is a longitudinal wrinkling of the pipe wall. Tests of *non-pressurized* Driscopipe show that buckling and collapse do not occur when the soil envelope is in full contact with the pipe and is compacted to a dense state. However, it can be forced to occur over the long term in *non-pressurized* pipe if the total external soil pressure, P_t, is allowed to exceed the pipe-soil system's critical buckling pressure, P_{cb}. If $P_t > P_{cb}$, gradual collapse may occur over the long term. A calculated, conservative value for the critical buckling pressure may be obtained by the following approximate formula. All pipe diameters with the same SDR in the same burial situation have the same critical collapse and critical buckling endurance

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

Where:

P_t = Total vertical soil pressure at the top of the pipe, psi

P_{cb} = Critical buckling soil pressure at the top of the pipe, psi

E' = Soil modulus in psi calculated as the ratio of the vertical soil pressure to vertical soil strain at a specified density

P_C = Hydrostatic, critical-collapse differential pressure, psi

$$P_C = \frac{2E\,(t/D)^3\,(D_{MIN}/D_{MAX})^3}{(1-\mu^2)}$$

$$P_c = \frac{2.32\,E}{(SDR)^3}$$

Where: $(D_{MIN}/D_{MAX}) = .95$
μ = Poission's Ratio
$\mu = .45$ for Driscopipe
E = stress and time dependent tensile modulus of elasticity, psi

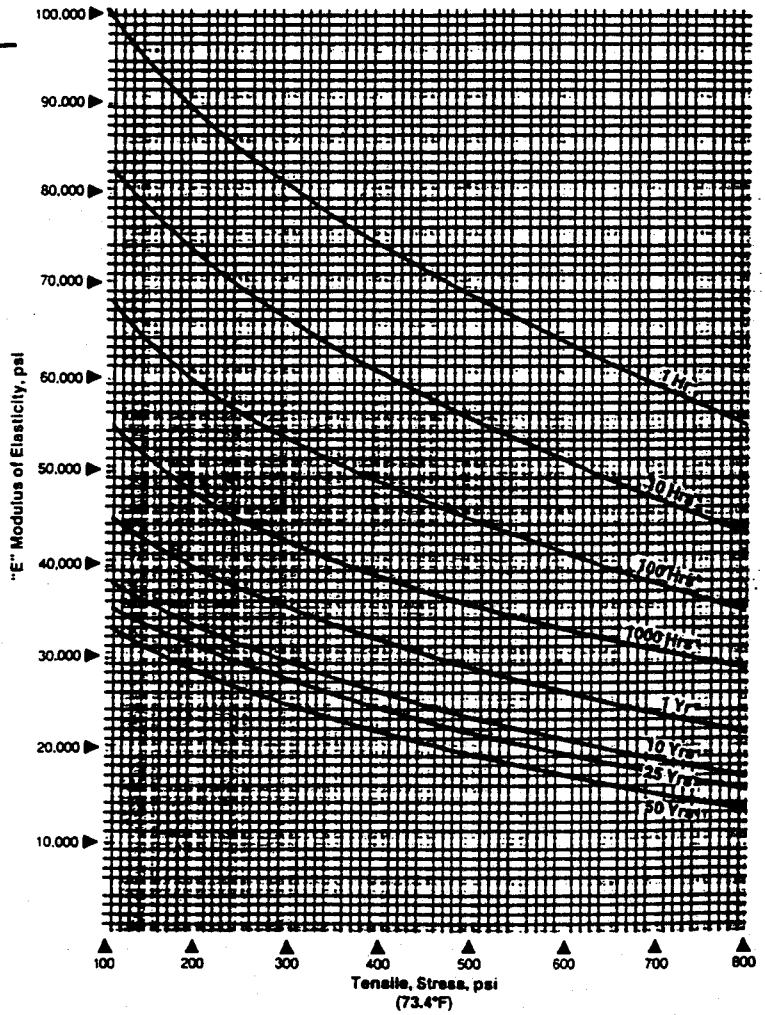
In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Design by Wall Buckling Guidelines:
Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_t < P_{cb}$.

1. Calculate or estimate the total soil pressure, P_t, at the top of the pipe.
2. Calculate the stress "S_A" in the pipe wall according to the formula:
$$S_A = \frac{(SDR - 1)\,P_t}{2}$$
3. Based upon the stress "S_A" and the estimated time duration of non-pressurization, use Chart 25 to find the value of the pipe's modulus of elasticity, E, in psi.

Chart 25

Time Dependent Modulus of Elasticity for Polyethylene Pipe vs. Stress Intensity (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

4. Based upon the pipe SDR and the value of the polyethylene modulus of elasticity, E, calculate the pipe's hydrostatic, critical-collapse differential pressure, P_C:

$$P_C = \frac{2.32\,(E)}{(SDR)^3}$$

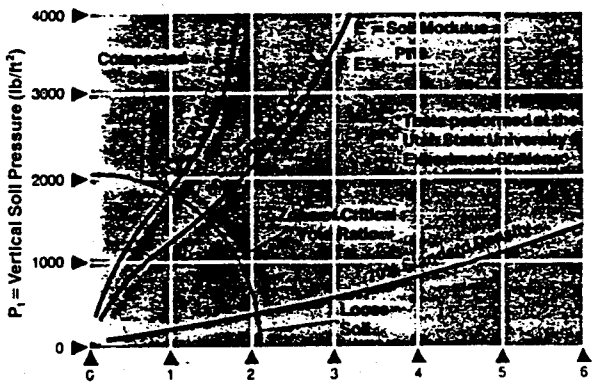
5. Calculate the soil modulus, E', by plotting the total external soil pressure, P_t, against a specified soil density to derive the soil strain as shown in the example problem on Chart 26.
6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

7. Calculate the Safety Factor: S.F. = $P_{CB} \div P_t$ In burial applications, a safety factor of 1.0 may be considered a minimum because of the margin of safety provided by the arching action of the soil. However, Driscopipe endorses using a more conservative value approaching or exceeding a 2.0 safety factor.
8. The above procedures could be reversed to derive the minimum pipe SDR required for a given soil pressure and an estimated soil density. However, this procedure should permit the engineer to optimize the system design quickly by examining several combinations.

Chart 26

Plot of Vertical Stress-Strain Data for Typical Trench Backfill (Except Clay) from Actual Tests*



EXAMPLE
Find: E' @ 2000 PSF and 80% Density
Formula: $E' = P_t/\varepsilon_s$
Calculations: E' = 2000 PSF / .018 = 111111 PSF = 771 psi

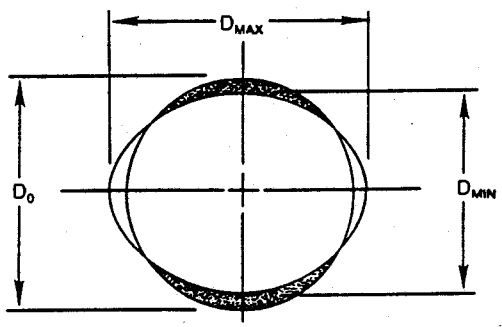
Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill, such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be examined by extrapolating the slope of the curve or by generating curves by testing at those higher soil pressures. Probable error of curves is about half the distance between adjacent lines.

Design by Ring Deflection: Ring deflection is defined as the ratio of the vertical change in diameter to the original diameter. It is often expressed as a percentage. Ring deflection for buried Driscopipe is conservatively the same as (no more than) the vertical compression of the soil envelope around the pipe. Design by ring deflection matches the ability of Driscopipe to accommodate, without structural distress, the vertical compression of the soil enveloping the buried pipeline. *Design by ring deflection comprises a calculation of vertical soil strain to ensure it will be less than the allowable ring deflection of the pipe.* See Chart 27. The tabulation shows that with lower values of SDR, the allowable deflection is less. For installations which require this thicker wall to resist the external soil pressure, actual ring deflection can easily be limited to the tabular values by proper compaction of the backfill around the pipe. The recommended allowable deflection for the various SDRs are:

Chart 27

SDR	Allowable Ring Deflection
32.5	8.1%
26.0	6.5%
21.0	5.2%
19.0	4.7%
17.0	4.2%
15.5	3.9%
13.5	3.4%
11.0	2.7%

The allowable ring deflection of polyethylene pipe is a function of the allowable tangential strain in the outer surface of the pipe wall. A conservative limit of 1-1½% tangential strain in the outer surface of the pipe wall due to vertical deflection of the pipe "ring" by soil compression can be understood by comparing two pipes of the same diameter but different wall thickness.



$$\%\ \text{Ring Deflection} = \left(1 - \frac{D_{MIN}}{D_0}\right) \times 100\%$$

NOTE: 5% deflection decreases flow-area by ¼%. 10% deflection decreases flow-area by 1%.

Assume each of the pipes is equally deflected under loads required to achieve that result. The tangential surface strain developed in the thickwall pipe is much greater than the surface strain in the thinwall pipe. The tangential strain varies directly as the wall thickness (i.e.: distance from the neutral axis) and is proportional to the amount of ring deflection. For a given ring deflection, the thicker the wall, the higher the strain.

Alternately, assume that each of the pipes is subjected to loads such that the tangential surface strain in the pipe's wall surface is equal for both pipes. For equal surface strain, the degree of vertical deflection of the pipe ring is different for the two pipes. Under these circumstances, the degree of deflection would be less for the thickwall pipe and greater for the thinwall pipe.

The percentage ring deflection based upon strain for a given SDR pipe can be calculated as follows:

$$\frac{\Delta Y}{D} = (0.25)(\varepsilon)(\frac{D}{t})$$

$$\frac{\Delta Y}{D} = (0.25)(\varepsilon)(SDR)$$

$$\frac{\Delta Y}{D} = (.0025)(SDR)$$

Where: ε = Tangential strain in the surface of the pipe ring due to deflection (conservatively 0.01 for Driscopipe)
D = Pipe OD, inches
t = Pipe wall thickness, inches
SDR = Standard Dimension Ratio, D/t
ΔY = Vertical deflection, inches

Driscopipe recognizes a tangential surface strain value of 0.01 due to ring deflection as a conservative yet responsible design parameter. This value is based upon the following:

- Most of the deflection of a flexible pipe occurs within a few hours or a few days after final backfilling and increases very little thereafter. This results in the development of the soil arch over the pipe which relieves the pipe of much of the vertical soil load by the arching action of the soil envelope and by the development of soil restraint at the sides of the pipe.
- With an allowable long term stress of 800 psi and a time of approximately four days (100 hours) to reach the maximum allowable ring deflection, the tangential strain would be 2% to 2½% (See Chart 25 for time and load dependent modulus of elasticity to calculate: strain = stress ÷ modulus.) Therefore, the use of 800 psi incorporates an additional margin of safety of over 2.0.
- An allowable strain value of 0.01 will allow for reasonable additional deflection due to disturbance of the backfill by earthquake, fluctuations of the water table, etc.
- An allowable design strain value of 0.01 allows for the normal deviation of temperature encountered during installation.

In summary, a soil density can be specified for the bedding and initial backfill so that the vertical strain of the sidefill soil under the total soil pressure, P_t, at the top of the pipe will be no greater than the maximum allowable ring deflection for a given SDR pipe.

Test Performance of Direct Burial Driscopipe: The density of the bedding and soil envelope determines the performance capability of the pipe-soil system with regard to ring deflection. Tests conducted on Driscopipe at Utah State University by Dr. Reynold K. Watkins show that Driscopipe will not buckle under ordinary conditions if the soil envelope is compacted and is in full contact with the pipe. A virtual fail-safe installation can be assured if soil density is generally over 85% of Standard Proctor (AASHTO T-99) Density. With the backfill compacted to 90% of standard density, the depth of laying for Driscopipe is almost unlimited. However, in the thinner wall series, due to the flexibility of the pipe, it is difficult to achieve the desired high soil density without buckling of the pipe cross-section, particularly when using backfill material other than sand or gravel.

Soil densities less than 90% are certainly adequate if depth of soil cover is in the range of most installations. Compaction to 75% standard density is relatively easy to achieve even in poor soils. This would cover a large percentage of all installations. However, 85% Standard Proctor Density (AASHTO T-99) should be considered a conservative minimum.

ASTM D-2321, Standard Recommended Practice for Underground Installation of Flexible Thermoplastic Sewer Pipe should be used as a guide for determining the method of placing and compacting the backfill.

Soil Density and Compaction: Standard tests are available for determining the density of the compacted soil. However, an experienced engineer can usually tell approximate density by his footmarks in the soil. If he has to backkick the soil with a corner of his heel to leave an impression, then the density is probably greater than 95%. A corner heel impression while walking probably indicates a soil density of 90%. A full heel print may indicate a density above 80%. A full footprint may indicate a density of 70%.

Various methods of compacting soil may be used, depending upon circumstances. All soils can be compacted mechanically at optimum moisture content. ASTM and AASHTO publications describe how this can be done. If the soil is granular, and if it drains quickly, it can be flushed into place using a water jet. The soil should actually be moved into place by the jet. Using this method with coarse sand the density is well over 90%. Finer, granular soils are usually denser than 80%. Granular soils that drain slower can be placed about the pipe, then flooded with water and allowed to drain. This usually accomplishes at least 80% density even in fine sand. If the backfill soil will not drain overnight then it is probably too fine to be compacted in this manner. Very dry sand or gravel can be vibrated into place at a density over 90% if there is little or no silt in it. Clean select aggregate will usually achieve 90% density or better just by dumping it around the pipe. The decision as to compaction technique is usually a decision in economics whereby the cost of the soil is weighed against the cost of compacting it. If the soil envelope requires mechanical compaction, economy may be realized by laying the pipe in a ditch only slightly wider than the pipe and specifying sand as the soil envelope material, provided the sand can be washed under the pipe haunches to offer proper support.

Soil bedding under the pipe requires attention. Pressure concentrations, as caused by large rocks, should be avoided. If the native soil is of very poor quality, the trench should be overexcavated about ⅛ the diameter or 3 inches, whichever is greater, and backfilled with sand. Flushing of the soil into place assures adequate support under the haunches. Ponding of the soil is usually adequate. If backfill soil which requires mechanical tamping is used at the sides of the pipe, it may be necessary to use select fill under the haunches to assure adequate density.

The effects of vibration or shock waves on soil compaction are much more complex than the effects of static loads and should be given careful and expert analysis. In some cases, vibration or shock may result in increased density in some backfill soils, resulting in increased stiffness of the pipe-soil system. However, if the pipe is installed below the water table, vibration may result in liquefaction of the backfill and adjacent soil, subjecting the pipe to hydrostatic loading which may result in possible hydrostatic failure of an apparently stable system.

Minimum Cover: There are no firm rules regarding minimum burial depth suitable for every installation. However, the following guidelines may be helpful:

- Consider a burial depth below the local frost line.
- Where there will be no overland traffic, the designer may wish to consider a cover of 18" or one diameter, whichever is greater.
- Where truck traffic may be expected, the designer may wish to consider a burial depth of 36" or one diameter, whichever is greater.
- Where heavy off-the-road truck or locomotive traffic is expected, the designer may wish to consider a minimum cover of 5 feet or more.
- Pavements designed for heavy truck traffic substantially reduce the pressure transmitted through a wheel to the subgrade and, consequently, to the underlying pipe. The pressure reduction is so great that, generally, the live load can be neglected. The pressure intensity, however, can be estimated, as will be discussed.
- For heavy duty asphalt or flexible pavements, the reduction in intensity will be comparable to that for concrete pavements.
- For intermediate thicknesses of asphalt or flexible pavements, there is no generally accepted theory for estimating load distribution effects.
- Relatively thin pavements do not reduce the pressure transmitted from the wheel to the subgrade to any significant degree. Such

Type 4: Marine Pipelines

Introduction: The primary design criteria for submerged and weighted pipelines are (a) the critical collapse pressure for empty or partially full pipelines, (b) weight of the concrete anchor and (c) spacing of the concrete anchors. Even though a marine pipeline is sometimes buried in an underwater trench, any support that the pipeline receives from the backfill material should be ignored for design purposes.

Driscopipe can be buried, rest on the bottom or floated on the surface of lakes, rivers, marshes or oceans. Its characteristics of flexibility, light weight, inertness to saltwater and chemicals, continuous pipeline due to butt fusion and the ability to float even when full of water give polyethylene many advantages.

The design of the external weights for water installations is a matter of preference. The weights are normally made to order by precast concrete manufacturers or by the contractor at the job site. These weights can be designed to hold the pipe away from the bottom using weights as legs or in a trench or directly on the bottom. It is advisable for weights for pipes larger than 12" to have reinforcing steel for strength. It is also recommended that one turn of rubber gasketing material or 2 to 3 turns of 5 to 10 mil polyethylene sheet be wrapped around the pipe under the weight to act as a cushion and prevent damage to the pipe.

Critical Collapse Pressure: A marine pipeline does not receive any structural support from the surrounding water. Therefore, an empty or partially filled non-pressure pipeline is subject to long-term hydrostatic collapse. A marine pipeline that is full of water at all times, such as an outfall or intake line, is not subject to collapse because the internal pressure will equal the external pressure at any specific depth of water.

Chart 32, shown below, gives design levels for empty or partially filled marine pipelines. These pressures represent the safe maximum differential pressures which can be applied to Driscopipe without buckling or collapsing the pipe. These values are based on extensive long-term differential pressure test data on actual lengths of pipe. The collected test data were mathematically analyzed and used to calculate performance limits for all pipe SDR s considering various degrees of original ovality. The values in Charts 32 and 33 are the calculated lower tolerance limits derived from the test data and represent the level of external differential pressure at which the risk of pipe collapse becomes insignificant. Pipe collapse in this test work was defined as that point where the major deflected diameter was 20% greater than the original undeflected diameter. Copies of the test results and data analysis are available upon request.

Chart 33

Multiplier for Temperature Rerating External Pressure Differential

Temperature °F	Temperature °C	Multiplier
50	10	1.14
73.4	23	1.00
100	38	0.79
120	49	0.62
140	60	0.50

Anchor Weights: The dry land weight of the concrete anchors may be calculated from the following formula. The designer should note that concrete varies in density between 140 lbs./cu. ft. and 155 lbs./ cu.ft. and the "K" constant is an anchor constant. Where K = 1.0, neutral buoyancy is achieved. Where K = 1.3, the pipe should be adequately anchored for lakes, ponds and streams. Where current or tides are encountered, the designer may want to increase the K value to nearly 1.5 depending upon design factors and his judgment.

$$Wt^{Conc} = \frac{L(Wt^{Driscopipe} + Wt^{Product}) - (K \times Den^{Water} \times V(out)^{Driscopipe} \times L)}{\left(\frac{K \times Den^{Water}}{Den^{Conc}}\right) - 1}$$

Chart 32

Maximum External Hydrostatic Pressure Differential

Feet of Water Head @ 73.4°F

Service Life	Pipe SDR 7	9.3	11	15.5	17	19	21	26	32.5
1 day	437	337	202	83	65	48	36	18	10
1 month	249	192	147	54	34	28	22	10	6
1 year	232	180	111	50	32	23	17	10	5
50 years	204	159	97	44	29	22	17	9	4

Based on critical collapse testing of actual pipe samples.
Full Vacuum is 34 feet of water.

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$$\text{pipe deflection} = \frac{\text{load on pipe}}{\text{pipe stiffness} + \text{soil stiffness}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This is basically the form of the Iowa Formula, widely used for predicting deflections of buried flexible pipe. A constant value for the soil stiffness has been used for all compacted soil types. The originator of the formula and others

TABLE 1.—Bureau of Reclamation Values of E' for Iowa Formula (For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System[a]) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch: Dumped (2)	Slight, <85% Proctor, <40% relative density (3)	Moderate, 85%–95% Proctor, 40%–70% relative density (4)	High, >95% Proctor, >70% relative density (5)
Fine-grained Soils (LL > 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; Otherwise use $E' = 0$			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles	100	400	1,000	2,000
Coarse-grained Soils with Fines GM, GC, SM, SC[c] contains more than 12% fines				
Coarse-grained Soils with Little or No Fines GW, GP, SW, SP[c] contains less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection[d]	±2	±2	±1	±0.5

[a]ASTM Designation D-2487, USBR Designation E-3.
[b]LL = Liquid limit.
[c]Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).
[d]For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m³) (ASTM D-698, AASHO T-99, USBR Designation E-11). 1 psi = 6.9 kN/m².

are now recognizing that the soil stiffness varies according to soil type and degree of compaction. However, there has been no successful effort to organize the information on buried flexible pipe deflections to determine what soil stiffness values should be used for various pipe support conditions.

Reclamation experience with laboratory and field tests of buried flexible pipe

has resulted in an empirical relationship between pipe deflection and soil stiffness values for different pipe bedding construction conditions. In Table 1 are the values of the soil stiffness (modulus of soil reaction E') found to represent the types of soils and degrees of compaction for buried flexible pipe. As examined later, the modulus of soil reaction, E', is a misnomer because it is more than just a property of the soil, although empirically it only varies with soil type and compaction effect.

Iowa Formula

In 1941, Spangler published a design procedure (6) for the underground installation of flexible pipe. Spangler and Watkins (7) later modified the formula to include a more realistic value for the soil parameter. The modified Iowa Formula is given as

$$\Delta X = D_l \frac{KWr^3}{EI + 0.061\,E'r^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which ΔX = horizontal deflection of the pipe, in inches; D_l = deflection lag factor to compensate for the volume change of the soil with time, dimensionless; K = bedding constant which varies with the angle of the bedding, dimensionless; W = load on the pipe per unit length, in pounds per linear inch; r = pipe radius, in inches; EI = pipe wall stiffness per inch length, in inch-pounds; and E' = modulus of soil reaction, in pounds per square inch.

Rearranged Iowa Formula.—If the Iowa Formula is rearranged as

$$\Delta X = \frac{D_l KW}{\frac{EI}{r^3} + 0.061E'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\Delta X = \frac{\text{Load Factor}}{\text{Ring Stiffness Factor} + \text{Soil Stiffness Factor}} \quad \ldots\ldots\ldots\ldots (4)$$

then the following terms can be used to describe the three separate factors that affect the pipe deflection: Load factor = $D_l KW$; ring stiffness factor = EI/r^3; and soil stiffness factor = $0.061E'$.

Load Factor $D_l KW$.—The load factor incorporates the parameters that have to do with the magnitude and distribution of the soil pressures on a buried pipe.

The pipe deflection is directly proportional to the load factor and, yet, less is known about its components than any others in the Iowa Formula. Changes in construction procedures or bedding materials along a pipeline could significantly vary the load factor.

Deflection Lag Factor D_l.—After soil has been initially loaded it continues to reduce in volume with time. The deflection lag factor converts the immediate deflection of the pipe to the deflection of the pipe after many years. Spangler recommends a value of 1.5 for D_l. The actual value, however, depends on when the immediate deflection is measured, the volume change rate of the soil, and the load on the soil. Deflection lag factor D_l is basically empirical and ranges from 1.0–6 in observed tests.

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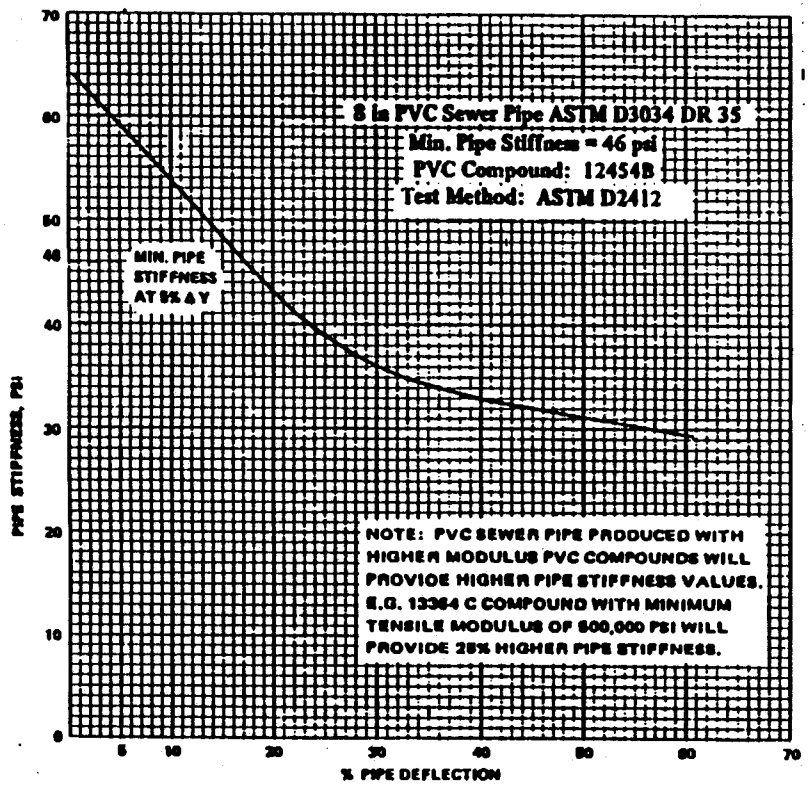
FIGURE 20

TYPICAL PIPE STIFFNESS TEST RESULTS
PIPE STIFFNESS VS. PERCENT DEFLECTION



Because a flexible conduit interacts with the surrounding soil in supporting the soil load, soil properties are very important. Just as bedding is important in limiting soil pressure concentrations on rigid pipes, soil compaction or soil density is an important parameter in limiting ring deflection in flexible pipes. Thus, soil and soil placement, as well as pipe properties, are important in the design of any buried pipe installation.

The manner in which flexible pipe performance differs from rigid pipe performance can be understood by visualizing pipe response to applied earth load. In a rigid pipe system, the applied earth load must be carried totally by the inherent strength of the unyielding, rigid pipe since the soil

at the sides of the pipe tends to compress and deform away from the load. In a flexible pipe system, the applied earth load is in large measure carried by the earth at the sides of the pipe, since the flexible pipe deflects away from the load. That portion of the load carried by the flexible pipe, a vertical vector of force, is transferred principally through the deflection mechanism into approximately horizontal force vectors assumed by the compressed soil at the sides of the pipe. Through the deflection mechanism, the distribution of earth load is carried principally by the surrounding soil envelope and to a lesser extent by the flexible pipe. The strength provided by most buried flexible pipe is derived through the deflection mechanism from the combined strength provided by the pipe-soil system.

Spangler's Iowa Deflection Formula. M. G. Spangler, a student of Anson Marston, observed that a theory of loads on buried pipe was not adequate for flexible pipe design. Spangler noted that flexible pipes may provide little inherent strength in comparison to rigid pipes, yet when buried a significant ability to support vertical loads is derived from the passive pressures induced as the sides of the pipe move outward against the earth. This fact coupled with the idea that the ring deflection may also be a basis for flexible pipe design prompted M. G. Spangler to publish his Iowa Formula in 1941.

Spangler's first step was to define the ability of a flexible pipe to resist ring deflection when not buried in the soil.

Applying the elastic theory of flexure to thin rings for deflections studied which were in the range of less than about 10%, he established the following relationships:

EQUATION 24

$$\Delta Y = 0.149 \frac{Wr^3}{EI}$$

EQUATION 25

$$\Delta X = 0.136 \frac{Wr^3}{EI}$$

EQUATION 26

$$\Delta X = 0.913 \Delta Y$$

FIGURE 22 – BEDDING ANGLE

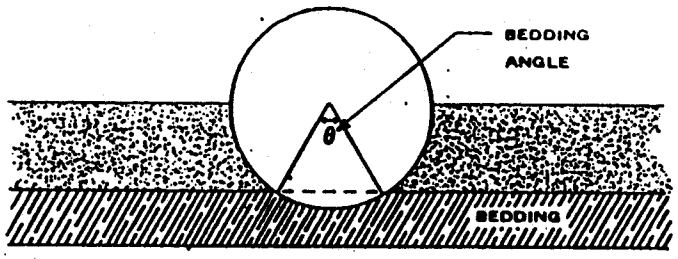


TABLE 28 – VALUES OF BEDDING CONSTANT, K

BEDDING ANGLE (DEGREES)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

In 1955, Reynold K. Watkins, a graduate student of Spangler, was investigating the modulus of passive resistance through model studies and examined the Iowa Formula dimensionally. The analysis determined that e could not possibly be a true property of the soil in that its dimensions are not those of a true modulus. As a result of Watkins' effort, another soil parameter was defined. This was the modulus of soil reaction, E' = er. Consequently, a new formula called the Modified Iowa Formula was written:

EQUATION 28

$$\Delta X = D_L \frac{KW_c r^3}{EI + 0.061E'r^3}$$

Two other observations from Watkins' work are of particular note. There is little point in evaluating E' by a model test and then using the modulus to predict ring deflection; the model gives ring deflection directly. Ring deflection may not be the only performance limit.

Many research efforts have attempted to measure E' without success. The most useful method has involved the measure of deflections for a pipe

under which other conditions were known followed by back-calculation through the Modified Iowa Formula to determine the correct value of E'. This requires assumptions regarding the load, bedding factor and deflection lag factor to be used and has led to a variation in reported values of E'.

One of the most recent and thorough attempts to acquire information on values of E' was conducted by Amster K. Howard of the United States Bureau of Reclamation. Howard reviewed both laboratory and field data from many sources. Using information from over 100 laboratory and field tests, he compiled a table of average E' values for various soil types and densities (see Table 29). He was able to do this by assuming values of

TABLE 29 – AVERAGE VALUES OF MODULUS OF SOIL REACTION, E'
(For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System[a]) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch			
	Dumped (2)	Slight, <85% Proctor, <40% relative density (3)	Moderate, 85%–95% Proctor, 40%–70% relative density (4)	High, >95% Proctor, >70% relative density (5)
Fine-grained Soils (LL > 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; Otherwise use E' = 0			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles	100	400	1,000	2,000
Coarse-grained Soils with Fines GM, GC, SM, SC[c] contains more than 12% fines				
Coarse-grained Soils with Little or No Fines GW, GP, SW, SP[c] contains less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection[d]	±2	±2	±1	±0.5

[a]ASTM Designation D-2487, USBR Designation E-3.
[b]LL = Liquid limit.
[c]Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).
[d]For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m³) (ASTM D-698, AASHO T-99, USBR Designation E-11). 1 psi = 6.9 kN/m².

SOURCE: "SOIL REACTION FOR BURIED FLEXIBLE PIPE" BY AMSTER K. HOWARD, U. S. BUREAU OF RECLAMATION, DENVER, COLORADO. REPRINTED WITH PERMISSION FROM AMERICAN SOCIETY OF CIVIL ENGINEERS JOURNAL OF GEOTECHNICAL ENGINEERING DIVISION, JANUARY 1977, PP. 29-48.

E', K and W_c and then using the Modified Iowa Formula to calculate a theoretical value of deflection. This theoretical deflection was then compared with actual measurements. By assuming the E' values of Table 29, a bedding constant K = 0.1 and deflection lag factor D_L = 1.0, Howard was able to correlate the theoretical and empirical results to within ±2 percent deflection when he used the prism soil load. This means that if theoretical deflections using Table 29 were approximately 5%, measured deflection would range between 3 and 7%. Although the vast majority of data from this study was taken from tests on steel and reinforced plastic mortar pipe with diameters greater than 24 inches, it does provide some useful information to guide designers of all flexible pipe including PVC pipe since it helps to give an understanding of the Modified Iowa Deflection Formula.

The only parameter remaining in the Iowa Formula now needed to calculate deflections is the deflection lag factor D_L. Spangler recognized that in pipe-soil systems, as with all engineering systems involving soil, the soil consolidation at the sides of the pipe continues with time after the maximum load reaches the top of the pipe. His experience had shown that deflections could increase by as much as 30 percent over a period of 40 years. For this reason he recommended the incorporation of a deflection lag factor of 1.5 as a conservative design procedure.

Time lag will be discussed in much greater detail in another section of this chapter.

Under most soil conditions, flexible PVC pipe tends to deflect into a nearly elliptical shape and the horizontal and vertical deflections may be considered equal for small deflections (Δ). Since most PVC pipe is described by either pipe stiffness (F/ΔY) or outside diameter to thickness ratio (DR), the Modified Iowa Equation (28) can be transposed and rewritten as follows:

EQUATION 29

$$\% \frac{\Delta Y}{D} = \frac{D_L KP(100)}{0.149 \frac{F}{\Delta Y} + 0.061E'}$$

EQUATION 30

$$\% \frac{\Delta Y}{D} = \frac{D_L KP(100)}{[2E/3(DR-1)^3] + 0.061E'}$$

Where: P = Prism Load (soil pressure), psi

The above equations may be used in conjunction with the values for the empirical constants E', D_L and K. The following example illustrates their use.

Example: What will be the deflection of a DR 35 PVC pipe if buried on a flat bottom trench in a fine grained soil with unit weight of 120 lbs per ft^3 and with liquid limit less than 50% if the depth of burial is 10 feet?

From Table 29, E' = 200 psi and incorporating a bedding constant, K = 0.1, and the prism load as assumed in the derivation of Table 30 along with a deflection lag factor of D_L = 1.0, the following results are derived:

$$P = wH \qquad \% \frac{\Delta Y}{D} = \frac{D_L KP(100)}{[2E/3(DR-1)^3] + 0.061E'}$$

$$P = 120 \frac{lb}{ft^3} \times 10 \text{ ft} \times \frac{1 \text{ ft}^2}{144 \text{ in}^2} = 8.33 \text{ psi}$$

$$\frac{2E}{3(DR-1)^3} = \frac{2(400{,}000)}{3(35-1)^3} = 6.78 \text{ psi}$$

$$\% \frac{\Delta Y}{D} = \frac{1.0(0.1)(8.33)(100)}{6.78 + 0.061(200)} = 4.4 \text{ percent}$$

For the general case, live loads should be added to the earth load to determine the total load at the depth being considered.

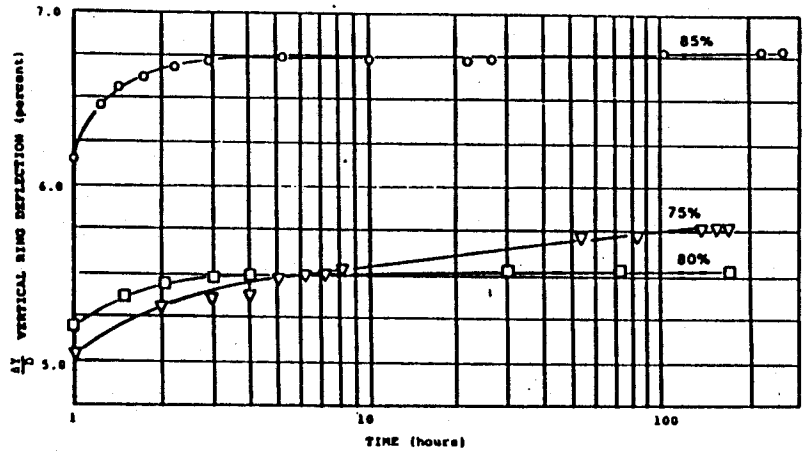
In Table 30, results of calculations for deflections of buried AWWA C900 DR 14, 18 and 25 PVC pipe are presented for cases where either highway or railway loads are present.

Deflection Lag and Creep. The length of time that a buried flexible pipe will continue to deflect after the maximum imposed load is realized is limited and is a function of soil density in the pipe zone. As soil density at the sides of the pipe increases, the time during which the pipe will continue to deflect decreases, and the total deflection in response to load decreases.

In fact, after the trench load reaches a maximum, the pipe-soil system continues to deflect only as long as the soil around the pipe is in the process of consolidation. Once the embedment soil has reached the density required to support the load, the pipe will not continue to deflect.

The full load on any buried pipe is not reached immediately after installation unless the final backfill is compacted to a high density. For a

FIGURE 24 – PVC PIPE CREEP RESPONSE



Time vs. vertical ring deflection for three soil densities, as % of standard proctor density.

SOURCE: UTAH STATE UNIVERSITY

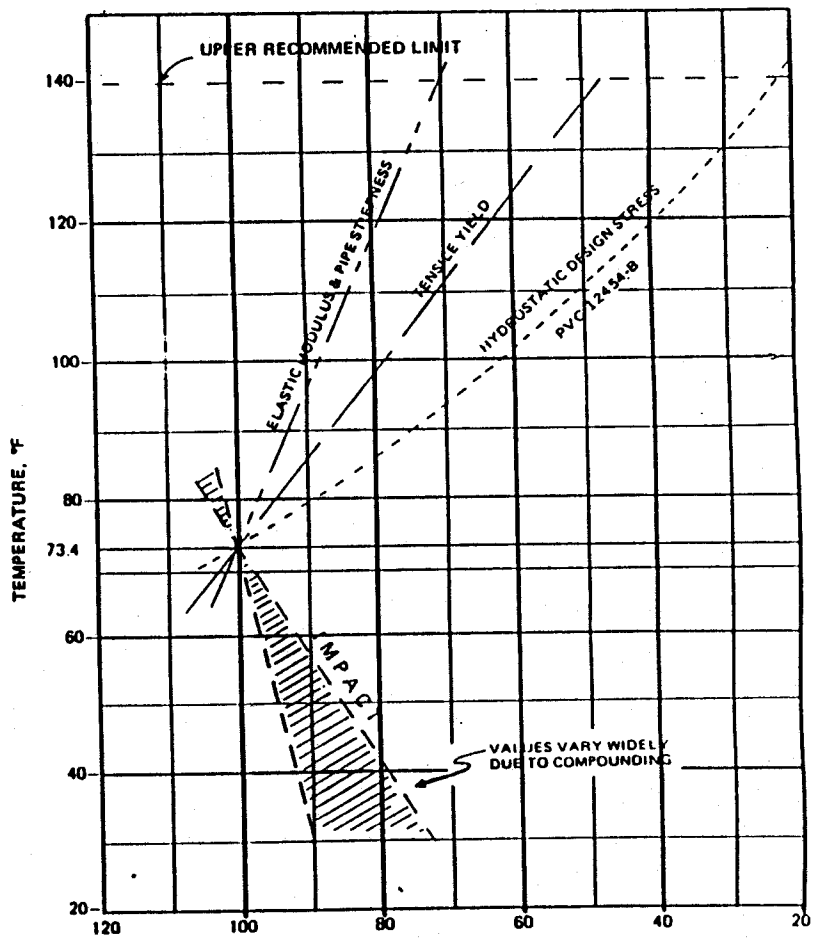
soil properties such as density exhibit great influence.

The theoretical strength properties of PVC pipe vary with temperature, (see Chapter III – Thermal Effects, and Chapter V – Hydrostatic Pressures). For flexible pipe considerations, the approximate relationships of pipe strength properties versus temperature are shown by curves in Figure 25.

Temperature controlled tests of buried PVC pipe were run to determine the temperature effect on the long-term behavior. Data from these tests are given in graphical form in Figure 26. The following procedures were used in conducting these tests. The pipe to be tested was placed in the load cell. It was then embedded in soil which was compacted to the specified percentage of proctor density. The load on the soil was then increased until the desired starting vertical deflection of the pipe was reached. At this point the load as well as the temperature was held constant, and the resulting time dependent deflection was determined. The starting deflections are somewhat arbitrary. Four of these tests were begun at about 4.75% deflection and two were begun between 9 and 9.5% deflection. The loads required to produce these deflections were different in each case. It should be noted that for the temperature range tested, an equilibrium state

FIGURE 25

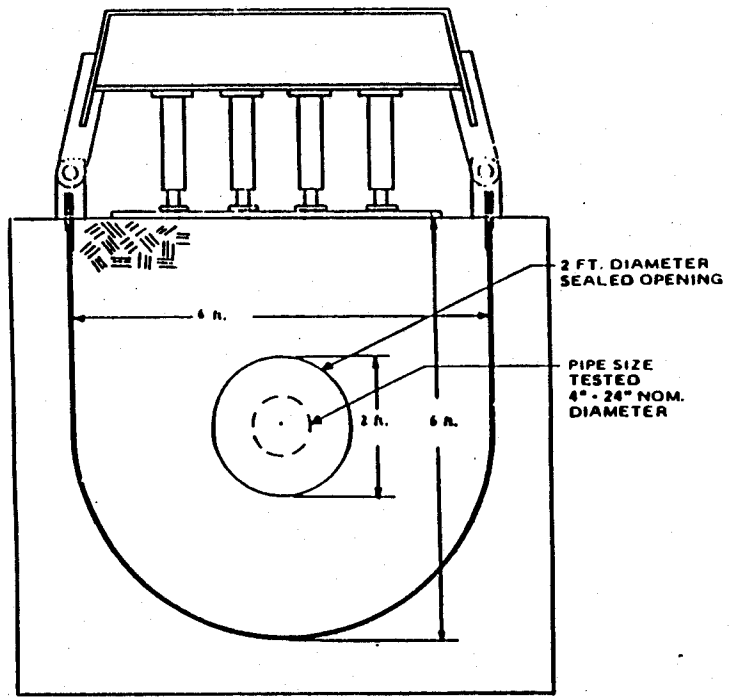
APPROXIMATE RELATIONSHIP FOR 12454-B PVC FOR FLEXIBLE PIPE STRENGTH PROPERTIES VS. TEMPERATURE



is reached and the pipe does not deflect beyond that point. The limiting deflection and the time required to reach it are largely controlled by the soil density. However, it is interesting to note Figure 26 for tests at different temperatures with the same soil density:

FIGURE 28

CELL CROSS SECTION



SOURCE: UTAH STATE UNIVERSITY

EQUATION 32

$$\text{Deflection} = \frac{\text{Load}}{\text{Pipe Stiffness} + (\text{Constant})\,(\text{Soil Stiffness})}$$

Upon analyzing data from many tests, Watkins wrote the Iowa Formula in terms of dimensionless ratios as follows:

EQUATION 33

$$\frac{\Delta Y}{D} = \frac{PR_s}{E_s AR_s + B}$$

Where: P = vertical nominal pressure at the level of the top of the pipe, psi

R_s = stiffness ratio. This is the ratio of soil stiffness, E_s, to pipe ring stiffness EI/D^3. This quantity includes all properties of materials, soil as well as pipe.

Since for a solid wall pipe of constant cross-section, $I = t^3/12$, then:

EQUATION 34

$$R_s = \frac{12\,E_s\,(D)^3}{E\,(t)}$$

Where: E_s equals slope of the stress-strain curve for the soil at the load in question in a one-dimensional consolidation test.

E_s = P/ϵ

ϵ = vertical soil strain

A, B = empirical constants which include such terms as D_L and K of the Iowa Formula

Through transposition, Equation 33 can be restated:

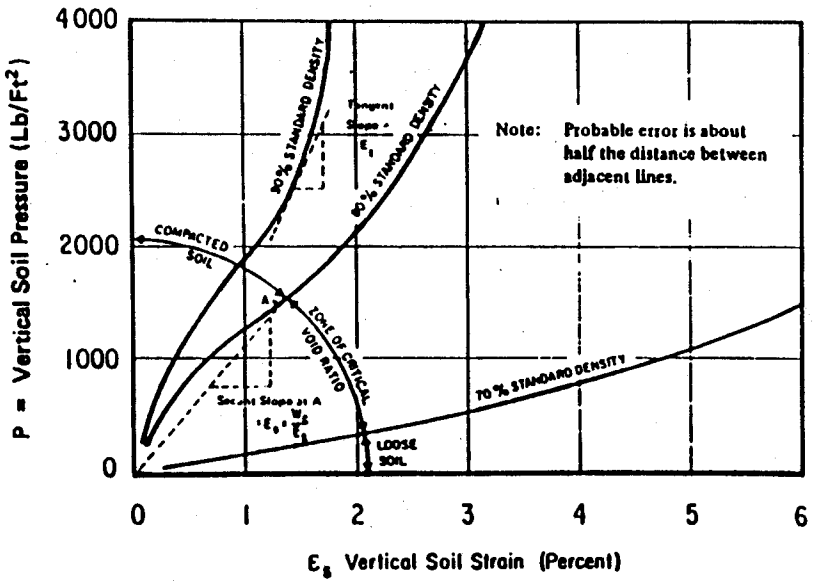
EQUATION 35

$$\frac{\Delta Y}{D\epsilon} = \frac{R_s}{AR_s + B}$$

In this form, the above equation represents a simple relationship between two dimensionless variables: ring deflection ratio $\frac{Y}{D\epsilon}$ and stiffness ratio R_s. Figure 29 represents the design curve which can be used for

FIGURE 31

PLOT OF VERTICAL STRESS STRAIN DATA FOR TYPICAL TRENCH BACKFILL (EXCEPT CLAY) FROM ACTUAL TESTS



SOURCE: UTAH STATE UNIVERSITY

Empirical Method. Each of the methods discussed so far for determining load and deflection has a theoretical basis, and except for the prism load theory, all require experimental investigation to determine the unknown constants. In the past few years, techniques have evolved whereby a model or prototype pipe is tested until failure occurs and the total performance of the pipe is studied. Suppose a pipe is to be designed with a certain earth cover in an embankment. Without a pipe in place, no arching occurs and the soil pressure at any height is easily calculated (the prism theory load at that depth). When a pipe having good flexibility is in place, the static pressure will not be greater than the prism load pressure applied. Trying to calculate this actual pressure has frustrated researchers for years. If a pipe is installed in a prism loaded condition (*e.g.*, soil cell) resulting deformation can be monitored without the need to calculate actual static pressure.

This procedure has been used with great success at Utah State University under the direction of Reynold K. Watkins and at the United States Bureau of Reclamation under the direction of Amster K. Howard as well as other places. Data obtained in this manner can be used directly in the design of pipe-soil systems and in the prediction of over-all performance. The possibility of buckling, over-deflection and wall crushing are all evaluated simultaneously by actual tests. No attempt to explain the pipe-soil interaction phenomenon is necessary in the use of this method and the end results leave nothing to be estimated on the basis of judgment.

For example, if tests show that for a given soil compaction at 25 feet (7.6 m) of cover, a flexible pipe deflects 3%, and in every other way performs well, the actual load on the pipe and the soil modulus are acadamic. Thus a pipe installation can be designed with a known factor of safety provided that enough empirical test data are available. In collection of this data, pipe was installed in a manner similar to that used in actual practice and the height of cover increased until performance levels were exceeded. The procedure was repeated many times and a reliable empirical curve of pipe performance versus height of fill was plotted. The use of these empirical curves or data eliminates the need to determine the actual soil pressure, since the pipe performance as a function of height of cover is determined directly. Equally good empirical approaches to study of the deflection mechanism are:

- the study of actual field installations, or
- the simulation of a large enough earth cover in a soil test box to exceed the performance limits of the pipe.

To avoid the problem of having to establish design data for the infinite variety of installations and bedding conditions that are found in the field, the following design bases have been chosen:

- The embankment condition is selected as critical. (The results are conservative for other than embankment conditions.)
- Time lag or settlement of the embankment is included by analyzing long-term values of deflection.

An added advantage of this system is that performance limits such as ring crushing, strain and wall buckling can be analyzed, as well as ring deflection, by means of a single test. Uni-Bell manufacturers have generated such data for their pipe products. The use of such data may be considered the most reliable method of design and is recommended when available. Some of the pipe products for which empirical test data have been deter-

Stress
Fatigue
Reverse Curvature
Longitudinal Bending

Other phenomena which have been shown not to be limiting performance considerations for PVC pipe include:

Buckling
Wall Crushing
Strain

The *stress* performance limit describes any internal pressure application in which the hoop stresses in the pipe wall due to applied pressure exceed the design strength of the pipe. The stress performance limit for PVC pipe is discussed in Chapter V.

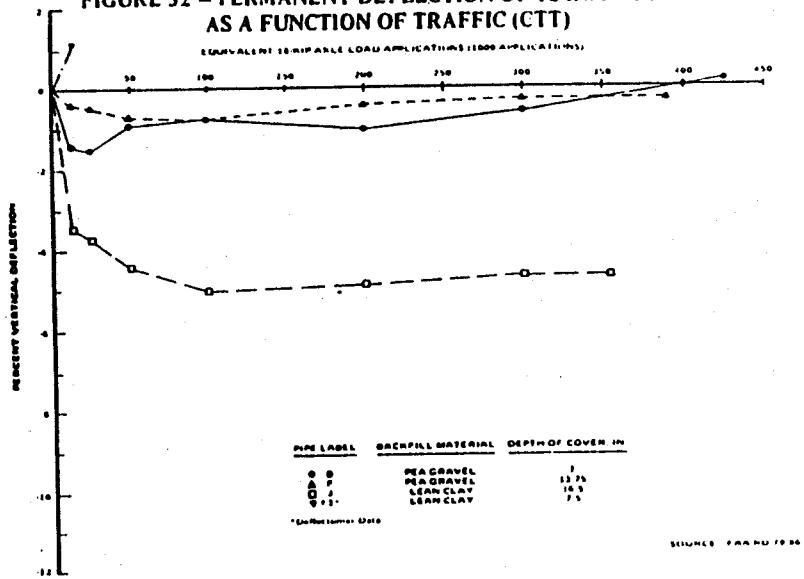
The *fatigue* performance limit may be a consideration in both pressure and gravity flow pipe applications. However, most potable water distribution systems and gravity flow sanitary systems function under conditions which do not warrant consideration of fatigue as a performance limit. PVC is similar to most materials in that it can fail at stresses lower than the strength of the material if a repeating stress application occurs at a sufficiently high frequency and magnitude on a continuous basis over a period of time. Cyclic stress variations can be induced internally by surge pressures and water hammer effects or the stress cycles can be caused from external loads such as traffic loadings on pipelines at shallow depths of burial.

In Chapter V, design performance limits for PVC pressure pipe subjected to repeated internal surge pressures are discussed in some detail, particularly relative to sewage force main design.

The performance of PVC sewer pipe exposed to dynamic wheel loadings while buried at shallow burial depths has been evaluated at the Geotechnical Laboratory, U. S. Army Engineer Waterways Experiment Station. Figure 32 is typical of the results obtained from the evaluation. It was concluded that PVC (DR 35) pipe performed very well under a range of loadings representative of highway and light to medium aircraft traffic. A minimum cover height of 12 inches is recommended for PVC (DR 35) pipe subjected to highway loads of up to 18 kip axle. Under light to medium aircraft loads of up to 320,000 pounds gross weight, a minimum burial depth of 2 feet is recommended.

It is recommended that special attention be given to the selection, placement and compaction of backfill material with shallow burial flexible

FIGURE 32 – PERMANENT DEFLECTION OF 12 IN. PVC PIPE AS A FUNCTION OF TRAFFIC (CTT)



pipe, such as PVC pipe underneath rigid pavement to prevent injurious cracking of the road surface.

A *reverse curvature* performance limit for flexible steel pipe was established shortly after publication of the Iowa Formula. It was determined that corrugated steel pipe would begin to reverse curvature at a deflection of about 20%. Design at that time called for a limit of 5% deflection, thus providing a structural safety factor of 4.0. From this early design consideration, an arbitrary design value of 5.0% deflection was selected.

Buried PVC sewer pipe (ASTM D 3034, DR 35) when deflecting in response to external loading may develop recognizable reversal of curvature at a deflection of 30%. This level of deflection has been commonly designated as a conservative performance limit for PVC sewer pipe. Research at Utah State University has demonstrated that the load carrying capacity of PVC sewer pipe continues to increase even when deflections increase substantially beyond the point of reversal of curvature. With consideration of this performance characteristic of PVC sewer pipe, engineers generally consider the 7.5% deflection limit recommended in ASTM D 3034 (Appendixes) to provide a very conversative factor of safety against structural failure.

$$P_{cr} = \frac{2(400{,}000)}{[1-(0.38)^2](35-1)^3} = 23.8 \text{ psi}$$

$$P_b = 1.15\sqrt{23.8\,(200)} = 79.34 \text{ psi} = 11{,}425 \text{ psf}$$

$$H = P/w = 11{,}425/120 = 95.2 \text{ feet}$$

Therefore, the safety factor for the critical failure mode by buckling of DR 35 PVC pipe is ample since sewer pipes are rarely buried in trenches deeper than 50 feet.

Research has established that flexible steel pipe walls can buckle at deflections considerably less than 20% if the load is large and the soil surrounding the pipe is extremely compacted. Based on these observations, H. L. White and J. P. Layer proposed the "Ring Compression Theory" for the design of buried flexible pipes. This theory assumed that the backfill was highly compacted, that deflection would be negligible and that the performance limit was *wall crushing*. The design concept is expressed by:

EQUATION 42

$$\sigma_c = \frac{P_y D}{2A} = \frac{\text{Yield Strength}}{\text{Safety Factor}}$$

Where:
- σ_c = compressive stress, psi
- P_y = vertical soil pressure on the crown of the conduit, psi
- A = cross-sectional area of conduit wall per unit length, unit length x t, in^2
- t = wall thickness, in.
- D = horizontal cross-sectional area of conduit, unit length x OD, in^2
- OD = outside diameter, in.

White and Layer assumed the vertical soil pressure (P_y) to be simply the prism load or the unit weight of the soil times the height of fill plus the influence of surface loads at the level of the top of the pipe.

Tests conducted by several researchers have confirmed the validity of the ring compression theory as applied to flexible steel pipe when the soil can be assumed to be rigid. These assumptions apply reasonably to PVC pipe when held rigidly, as in cases where the pipe is concrete cradled.

For this case the performance limit in ring compression may be calculated as follows:

Example: A PVC pipe (OD = 8.4 in., t = 0.240 in.) is concrete cradled. At what vertical soil pressure or depth of cover could one expect failure by ring compression?

$$\sigma_c = \frac{P_y D}{2A} \qquad P_y = wH = \frac{\sigma_c 2A}{D}$$

Conservatively assume σ_c = hydrostatic design basis or hoop tensile stress = 4,000 psi

$$P_y = \frac{4000\,(2)\,(0.240)}{8.4} = 228 \text{ lb/in}^2$$

$$H = \frac{P_y}{w} = \frac{228 \text{ lb/in}^2}{120 \text{ lb/ft}^3} \times 144 = 274 \text{ ft}$$

The previous example easily illustrates that ring compression is not a governing factor in design of either sewer or water PVC pipe systems.

Strain is generally not a performance limiting factor for buried PVC pipe. Total strain in a pipe wall can be caused by two actions: (1) flexure of the pipe as it deforms and (2) hoop stress in the pipe wall. Determination of strains due to hoop stress is straight forward. If a homogeneous wall is assumed and pressure concentrations are neglected, the formula follows:

EQUATION 43

$$\epsilon_h = \frac{PD}{2tE}$$

Where:
- ϵ_h = maximum strain in the pipe wall due to hoop stress, in/in
- P = pressure on pipe (may be internal and/or external pressure with the appropriate sign), psi
- E = modulus of elasticity of the pipe material, psi
- t = pipe wall thickness, in.
- D = pipe diameter, in.

PVC PIPE DIMENSIONS

Nominal Pipe Size	Wall Thickness		Outside Diameters		
	Minimum	Tolerance	Average OD	Tolerance Average	Tolerance Out-of-Roundness
ASTM D 1785, PVC PIPE, SCHEDULE 40					
1	0.133	+0.020	1.315	±0.005	±0.010
1¼	0.140	+0.020	1.660	±0.005	±0.012
1½	0.145	+0.020	1.900	±0.006	±0.012
2	0.154	+0.020	2.375	±0.006	±0.012
2½	0.203	+0.024	2.875	±0.007	±0.015
3	0.216	+0.026	3.500	±0.008	±0.015
3½	0.226	+0.027	4.000	±0.008	±0.050
4	0.237	+0.028	4.500	±0.009	±0.050
5	0.258	+0.031	5.563	±0.010	±0.050
6	0.280	+0.034	6.625	±0.011	±0.050
8	0.322	+0.039	8.625	±0.015	±0.075
10	0.365	+0.044	10.750	±0.015	±0.075
12	0.406	+0.049	12.750	±0.015	±0.075
ASTM D 1785, PVC PIPE, SCHEDULE 80					
1	0.179	+0.021	1.315	±0.005	±0.010
1¼	0.191	+0.023	1.660	±0.005	±0.012
1½	0.200	+0.024	1.900	±0.006	±0.012
2	0.218	+0.026	2.375	±0.006	±0.012
2½	0.276	+0.033	2.875	±0.007	±0.015
3	0.300	+0.036	3.500	±0.008	±0.015
3½	0.318	+0.038	4.000	±0.008	±0.015
4	0.337	+0.040	4.500	±0.009	±0.015
5	0.375	+0.045	5.563	±0.010	±0.030
6	0.432	+0.052	6.625	±0.011	±0.035
8	0.500	+0.060	8.625	±0.015	±0.075
10	0.593	+0.071	10.750	±0.015	±0.075
12	0.687	+0.082	12.750	±0.015	±0.075
ASTM D 2241, PVC PIPE (SDR-PR), SDR 21 (200)					
1	0.063	+0.020	1.315	±0.005	±0.015
1¼	0.079	+0.020	1.660	±0.005	±0.015
1½	0.090	+0.020	1.900	±0.006	±0.030
2	0.113	+0.020	2.375	±0.006	±0.030
2½	0.137	+0.020	2.875	±0.007	±0.030
3	0.167	+0.020	3.500	±0.008	±0.030
3½	0.190	+0.023	4.000	±0.008	±0.050
4	0.214	+0.026	4.500	±0.009	±0.050
5	0.265	+0.032	5.563	±0.010	±0.050

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Numerical relationships for service factors and design stresses of PVC are shown in Table I-C below.

Table I-C

Service Factors and Hydrostatic Design Stress (HDS)*. (Hydrostatic Design Basis equal 4000 psi) (27.6 MPa)

Service Factor	HDS
0.5	2000 psi (13.8 MPa)
0.4	1600 psi (11 MPa)

*Material: PVC Type I.

Maximum Pressures. The pressure ratings of PPS Schedule 80 thermoplastic pipe represent the maximum allowable operating pressure within a piping system for water at 73°F. (23.4°C.) based upon a service factor of 0.5. Table II-C gives these pressures for a number of pipe sizes in PVC/CPVC, PVDF (Kynar) and PP materials.

Table II-C

Pressure Ratings for Plastic Pipe—psi, Water, at 73° F. (23.4° C.)*

	MAXIMUM OPERATING PRESSURE (PSI) AT 75°F					
	Schedule 40 CPVC & PVC	Schedule 80 CPVC & PVC		Schedule 80 Polypropylene	Schedule 80 PVDF	
Nominal Pipe Size	Socket End	Socket End	Threaded End	Fused Joint**	Fused Joint	Threaded End
1/2	600	850	420	410	580	290
3/4	480	690	340	330	470	235
1	450	630	320	310	430	215
1-1/4	370	520	260	—	—	—
1-1/2	330	471	240	230	320	160
2	280	400	200	200	275	135
2-1/2	300	425	210***	185	—	—
3	260	375	190***	185	—	—
4	220	324	160***	160	—	—
6	180	280	N.R.	N.R.	—	—
8	160	250	N.R.	—	—	—
10	140	230	N.R.	—	—	—
12	130	230	N.R.	—	—	—

*Based on water service. for more severe service, an additional correction factor may be required.
**Threaded Polypropylene not recommended for pressure service.
***For threaded and backwelded joints.
N.R.—Not Recommended.
Plastic Piping is not recommended for air or compressed gas service.

External Pressures

Suction Lines. Polyvinyl chloride (PVC) pipe is frequently specified for situations where external pressures are applied to the pipe, such as in underwater applications. In such applications, the collapse rating of the pipe determines the maximum permissible pressure differential between external and internal pressures.

Table III-C gives these collapse ratings in psi at 73.4° F. for Schedule 80 pipe. Note that for other than room temperatures, the collapse pressures must be adjusted. For temperature correction (derating) factors, see Table IV-C in the discussion of "Temperature-Pressure Relationships." (Schedule 80 pipe only.)

Table III-C

Collapse Ratings of TYPE I PVC Pipe at 73.4° F.

Nominal Pipe Size (In.)	Collapse Rating—psi Schedule 80 (Type I)
1/2	1045
3/4	650
1	540
1-1/4	340
1-1/2	270
2	190
2-1/2	220
3	155
4	115
6	80

Vacuum Service. As is implied by the collapse ratings, PVC pipe is suitable for the vacuum or negative-pressure conditions that are found in many piping applications.

Laboratory tests have been conducted on Schedule 80 PVC pipe to determine performance under vacuum and at temperatures beyond recommended operating conditions. Pipe sizes under 6" show *no* deformation at temperatures to 170° F. and 27 inches of mercury vacuum. The 6" pipe showed slight deformation at 165° F. and 20 inches of mercury vacuum. Above this temperature, failure occurred owing to thread deformation.

Conclusion: All sizes of Schedule 80 PVC pipe are suitable for vacuum service up to 140° F. and 30 inches of mercury vacuum. Solvent-cemented joints are recommended for vacuum applications.

Temperature Derating. Pressure ratings for thermoplastic pipe are generally determined in a water thermoplastic pipe are generally determined in a water medium at room temperature (73° F.) As the system temperature increases, the thermoplastic pipe becomes more ductile, increases in impact strength and decreases accordingly.

The effects of temperature have been exhaustively studied and correction (derating) factors developed for each thermoplastic piping material. See Table IV-C. To determine the maximum operating pressure at any given temperature, multiply the pressure rating for the pipe size and type found in Table II-C by the temperature derating factor.

Example: To find the maximum operating pressure of 1-1/4" PVC Schedule 80 pipe at 110°F., refer first to Table II-C. This pressure rating is given as 520 psi. Referring to Table IV-C, the derating factor of PVC at 110°F. is .50. Therefore, maximum operating pressure at that temperature is 520 x .50 = 260 psi.

Table IV-C.

Temperature Derating Factors (f)—Schedule 80 Plastic Pipe.

Temperature °F	PVC	CPVC	PVDF	PP
70	1.0	1.0	1.0	—
80	0.88	—	—	0.93
90	0.75	—	—	—
→ 100	0.62	0.78	0.83	0.74
110	0.50	—	—	—
120	0.40	0.65	0.75	0.58
130	0.30	—	—	—

Characteristics of PPS Thermoplastics

Definition: Plastics are materials that contain as an essential ingredient an organic substance of large molecular weight, are solid in their finished state; and, at some stage in manufacture or processing into finished articles, can be shaped by flow.

Plastics are further divided into two groups known as *thermoplastics* and *thermosets* (or thermosetting plastics). Thermoplastics are capable of being repeatedly softened by increase in temperature and hardened by decrease in temperature. Thermosets, on the other hand, undergo cure in the molding or forming process and as a result of chemical reactions (produced by heat and/or added chemicals) become substantially infusible.

Choice of Thermoplastics. To meet any given set of engineering parameters, PPS pipe, fittings and valves are available in a choice of four outstanding thermoplastics. See Tables I-B and II-B for summaries of physical, thermal and chemical resistance properties.

Polyvinyl Chloride (PVC)

The most widely used thermoplastic in the manufacture of pipe, fittings and valves because of its economy, versatility, excellent chemical resistance, high tensile strength, good impact resistance and ability to withstand long-term pressures.

Chlorinated Polyvinyl Chloride (CPVC)

All the admirable properties of PVC—plus the ability to handle temperatures of 212°F. This makes CPVC pipe, fittings and valves suitable not only for hot corrosive service but also for hot-water distribution systems.

Polyvinylidene Fluoride (PVDF—Kynar®)

Features remarkable high-temperature performance. PVDF pipe and fittings can handle corrosive fluids at working temperatures to 280°F. Other PVDF qualities are excellent chemical resistance to such chemicals as the halogens, and resistance to weathering (U.V. resistant). It is also highly resistant to alpha and gamma radiation.

Polypropylene (PP)

The lightest of the PPS thermoplastics, PP is suitable for corrosive waste as well as pressure applications because of its inertness to a wide range of chemicals—including most solvents—and because of its ability to withstand working temperatures to 212°F., with peaks up to 230°F.

Table I-B

Physical and Thermal Properties—Thermoplastics vs. Steel

Property	PVC[1]	CPVC[2]	PVDF[3]	PP[4]	Steel	Comments
Specific Gravity	1.4	1.55	1.78	0.91	7.8	Plastics are lightweight. PVC is 1/6th the weight of steel, making it easy to handle.
IZOD Impact Strength (ft. lbs./inch of notch)	0.8	3.0	3.8	1.3	30	Plastics have less impact strength but greater flexibility.
Modulus of Elasticity, PSI	4.2×10^5	4.23×10^5	1.25×10^5	2×10^5	30×10^6	Plastics are easy to flex.
Ultimate Tensile Strength, PSI	8,000	8,400	7,000	5,000	60,000-90,000	Lower tensile strength of plastics is offset by other advantages such as no corrosion, easier to fabricate, no maintenance, and long life. Plastics offer an optimum balance of properties.
Working Stress @73°F.-PSI	2,000	2,000	1,275	705	20,000	
Williams & Hazen "C" Factor	150	150	150	150	65-110	Plastics' smooth interior bores yield higher flow rates . . . remain smooth indefinitely because they do not corrode or tuberculate.
Coefficient of Linear Expansion x 10^{-5} (In./in./°F.)	3.0	3.5	8.0	3.8	0.6	Plastics have high expansion rates, 5 to 10 times that of steel, but low modulus of elasticity.
Thermal Conductivity BTU/HR./Ft.²/in./°F.	1.10	0.95	1.50	1.3	26.20	Plastics are relatively self-insulating—have less heat loss or gain.

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Fig. 1
NOMINAL PHYSICAL PROPERTIES
POLY PIPE PE 3408 Pipe Material

Property	ASTM Reference	Nominal Value
Density, black	D 1505	0.955 gm/cc
Density, natural	D 1505	0.945 gm/cc
Melt flow	D 1238 (cond. E)	0.10 gm/10min.
Vicat softening temperature	D 1525	255°F
Brittleness temperature	D 746	◄– 180°F
Environmental stress cracking	D 1693 (cond. C)	►5000 hrs.
Tensile, yield (2"/min.)‡	D 638	►3200 psi
Tensile, break (2"/min.)	D 638	►5000 psi
Elongation (2"/min.)	D 638	►800%
Stiffness	D 747	62,000 psi
Modulus of elasticity	D 638	113,000 psi
Modulus of elasticity - long term		30,000 psi
Flexural modulus	D 3350	136,000 psi
Impact strength, 0.25"	D 256 (method A)	12
Impact strength, 0.125"	D 256 (method A)	Non-break
Hardness, shore D	D 2240	66
Thermal conductivity	C 177	2.7 Btu-in/ft.²/hr./°F
Thermal expansion coefficient	D 696	$9x10^{-5}$in/in/°F
Long term hydrostatic strength	D 2837	►1600 psi
Hydrostatic design basis	D 2837	1600 psi
Hydrostatic design stress	D 2837	800 psi
Material cell* classification	D 3350	PE 345434C

‡Rate of cooling of test sheet per ASTM D1928, procedure C.

*For explanation of **Poly Pipe** cell classification see page A-10.

Fi

PIPE INFORMATION: minimum wall, inches
weight per foot, pounds
nominal I.D., inches

HDPE

O.D., inches nominal	actual	7	7.3	9	9.3	11	11.5	13.5	15.5	17	21	26	32.5
		0.120	0.115	0.093	0.090	0.076	0.073						
1/2	0.840	0.12	0.11	0.09	0.09	0.08	0.08						
		0.590	0.601	0.646	0.652	0.681	0.688						
		0.150	0.144	0.117	0.113	0.095	0.091						
3/4	1.050	0.18	0.18	0.15	0.14	0.12	0.12						
		0.738	0.751	0.807	0.815	0.851	0.860						
		0.188	0.180	0.146	0.141	0.120	0.114						
1	1.315	0.29	0.28	0.23	0.22	0.19	0.19						
		0.924	0.940	1.011	1.021	1.066	1.077						
		0.237	0.227	0.184	0.178	0.151	0.144	0.123					
1 1/4	1.660	0.46	0.44	0.37	0.36	0.31	0.30	0.26					
		1.167	1.187	1.276	1.289	1.346	1.360	1.404					
		0.271	0.260	0.211	0.204	0.173	0.165	0.141	0.123				
1 1/2	1.900	0.60	0.58	0.48	0.47	0.40	0.39	0.33	0.29				
		1.335	1.359	1.461	1.475	1.541	1.556	1.607	1.645				
		0.339	0.325	0.264	0.255	0.216	0.207	0.176	0.153	0.140			
2	2.375	0.93	0.90	0.75	0.73	0.63	0.61	0.52	0.46	0.42			
		1.669	1.698	1.826	1.844	1.926	1.945	2.009	2.056	2.084			
		0.500	0.479	0.389	0.376	0.318	0.304	0.259	0.226	0.206	0.167	0.135	
3	3.500	2.03	1.96	1.64	1.59	1.37	1.32	1.14	1.00	0.92	0.75	0.61	
		2.460	2.503	2.691	2.717	2.838	2.867	2.961	3.030	3.072	3.153	3.220	
		0.643	0.616	0.500	0.484	0.409	0.391	0.333	0.290	0.265	0.214	0.173	0.138
4	4.500	3.35	3.24	2.71	2.63	2.26	2.17	1.88	1.65	1.52	1.24	1.01	0.82
		3.163	3.218	3.460	3.494	3.649	3.686	3.807	3.896	3.949	4.054	4.140	4.212
		0.795	0.762	0.618	0.598	0.506	0.484	0.412	0.359	0.327	0.265	0.214	0.171
5	5.563	5.13	4.95	4.13	4.02	3.46	3.32	2.87	2.53	2.32	1.90	1.55	1.25
		3.910	3.978	4.277	4.319	4.511	4.557	4.706	4.816	4.882	5.012	5.118	5.207
		0.946	0.908	0.736	0.712	0.602	0.576	0.491	0.427	0.390	0.315	0.255	0.204
6	6.625	7.27	7.02	5.86	5.70	4.91	4.71	4.07	3.58	3.29	2.69	2.20	1.77
		4.656	4.737	5.094	5.143	5.372	5.427	5.604	5.736	5.814	5.969	6.095	6.201
		1.232	1.182	0.958	0.927	0.784	0.750	0.639	0.556	0.507	0.411	0.332	0.265
8	8.625	12.32	11.90	9.94	9.66	8.32	7.99	6.90	6.07	5.57	4.56	3.72	3.00
		6.062	6.167	6.632	6.696	6.994	7.065	7.296	7.468	7.570	7.771	7.935	8.073
		1.536	1.473	1.194	1.156	0.977	0.935	0.796	0.694	0.632	0.512	0.413	0.331
10	10.750	19.14	18.48	15.44	15.00	12.98	12.41	10.72	9.45	8.65	7.09	5.78	4.66
		7.556	7.687	8.266	8.346	8.717	8.806	9.094	9.307	9.435	9.685	9.890	10.062

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CALCULATION SHEET

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PROJECT NO. _____

CLIENT _____ SUBJECT _____ Prepared By _____ Date ____

PROJECT _____ _____ Reviewed By _____ Date ____

_____ _____ Approved By _____ Date ____

REFERENCE 7

The pipe spacing may be determined by the Mound Model. In the Mound Model (see Figure 4-8), the maximum height of fluid between two parallel perforated drainage pipes is equal to (USEPA, 1989a):

$$h_{max} = \frac{L/c}{2} \left[\left((\tan^2 a)/2 \right) + 1 - \left((\tan a)/c \right) \left(\tan 2 a \right) + c \right]^{1/2}$$

where c = q/k
k = permeability
q = inflow rate
a = slope

The two unknowns in the equation are:

L = distance between the pipes;
c = amount of leachate.

Using a maximum allowable head, h_{max}, of 30 cm (12 in), the equation is usually solved for "L" (USEPA, 1989a).

One method for determining the value of "c" is by using the Water Balance Method described in USEPA (1989a). The computer program Hydrologic Evaluation Landfill Performance (HELP) Model can also be used to estimate "c." The HELP Model is a quasi-two-dimensional hydrologic model of water movement across, into, through, and out of landfills. The model uses climatologic, soil, and, landfill design data and incorporates a solution technique that accounts for the effects of surface storage, run-off, infiltration, percolation, soil-moisture storage, evapotranspiration, and lateral drainage. The program estimates run-off drainage and leachate that are expected to result from a wide variety of landfill conditions, including open, partially open, and closed landfill cells. The model also may be used to estimate the depth of leachate above the bottom liner of the landfill. The results may be used to compare designs or to aid in the design of leachate collection systems (USEPA, 1988a).

Once the percolation and pipe spacing are known, the design flow rate can be obtained using the curve in Figure 4-9. The amount of leachate percolation at the particular site is located on the x-axis. The required flow rate is the point at which this value intersects with the pipe spacing value determined from the Mound Model. Using this value of flow rate and the bottom slope of the site, the required diameter for the pipe can be determined (see Figure 4-10). Finally, the graphs in Figures 4-11 and 4-12 show two ways to determine whether the strength of the pipe is adequate for landfill design. In Figure 4-11 the vertical soil pressure is located on the y-axis. The density of the backfill material around the pipe is not governed by strength, so it will deform under pressure rather than break. Ten percent is the absolute limiting deflection value for plastic pipe. Using Figure 4-11, the applied pressure on the pipe is located and traced to the trench geometry, and then the pipe deflection value is checked for its adequacy (USEPA, 1989a).

The LCS specifications should include the (USEPA, 1988a):

- Type of piping material;
- Diameter and wall thickness;
- Size and distribution of slots and perforations;

APPENDIX R

Leachate Pump Information

Force Main Velocities

3" ID

Area $= \left(\frac{3}{12}\right)^2 \frac{\pi}{4} = 0.0491$ SF

@ 2 ft/sec Flow min = 44 gpm

SYSTEM CURVES

1. Pump for Sump C TSP 27-1
   76 gpm @ 93' TDH When operating alone

2. Pump for Sump B TSP-27-1
   80 gpm @ 94' TDH When operating alone

SYSTEM CURVES, CONT.

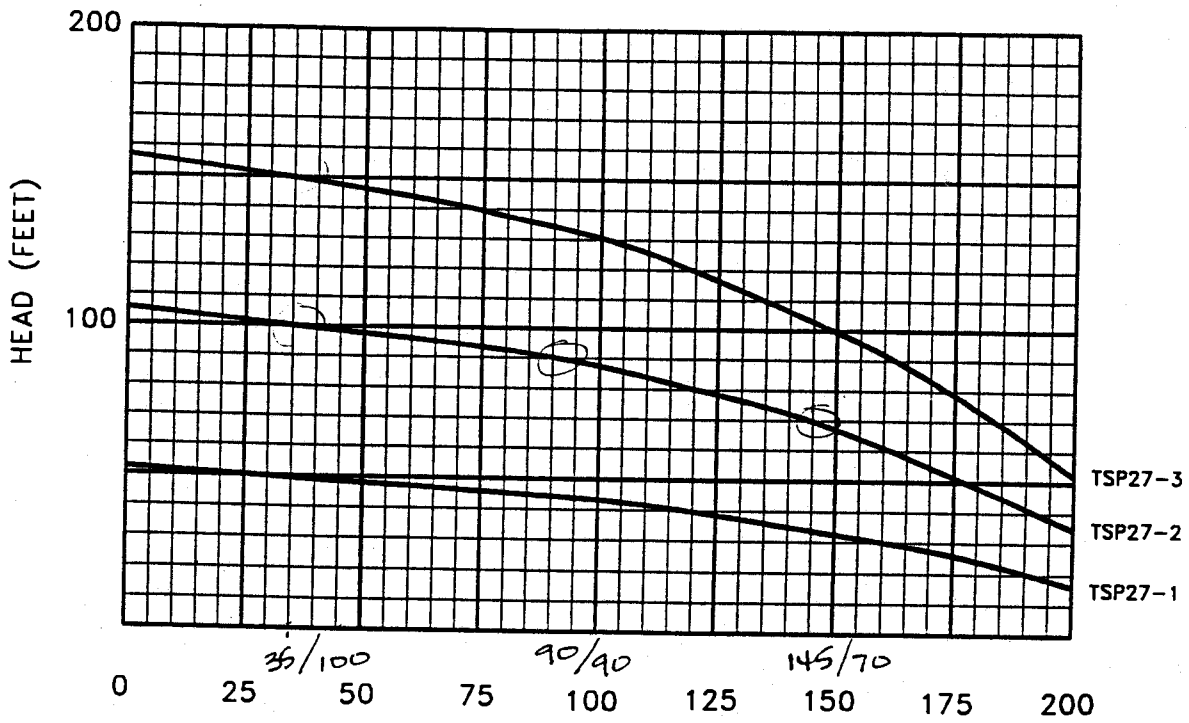
3. Pumps in B and C operating simultaneously

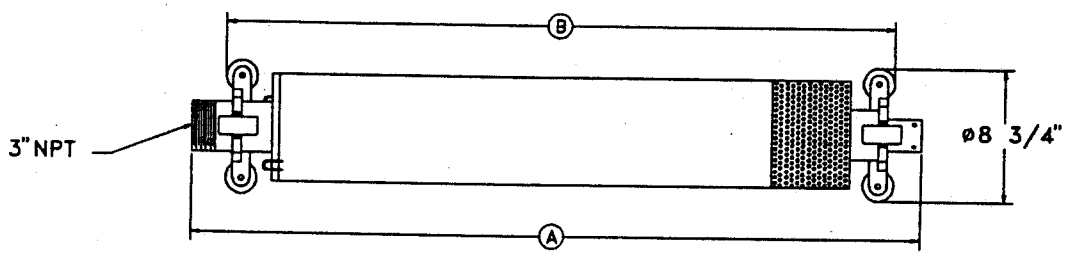
System curve shows force main from point where line from Sump C and B tie together (Manhole B).

Shown on System Curve are modified curves for pumps in Sump C and B, and the combined curve for B&C operating simultaneously.

TSP27 SurePump™
SIDE SLOPE RISER

892





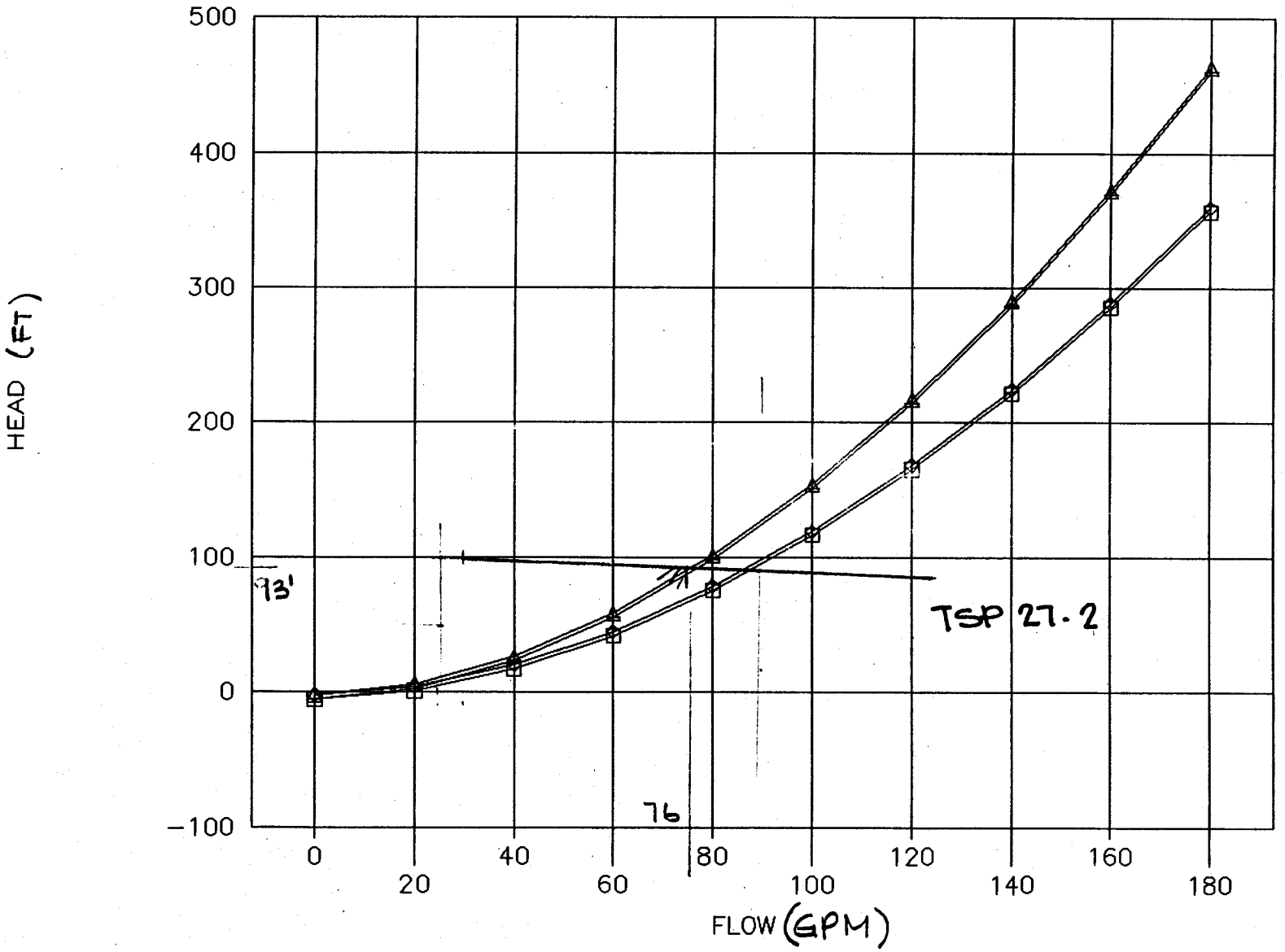
PUMP MODEL	SINGLE PHASE			THREE PHASE			SHIPPING WEIGHT (LBS)	
	MOTOR HP	A (in)	B (in)	MOTOR HP	A (in)	B (in)	1ø	3ø
TSP27-1	2.0	45.75	42.75	2.0	44.25	41.25	87.0	82.0
TSP27-2	5.0	63.50	60.50	5.0	57.50	54.50	140.2	120.5
TSP27-3	5.0	67.00	64.00	5.0	61.00	58.00	147.5	127.7

SEE 0579-2 FOR HIGH HEAD MODELS.

GRAPH #1

SYSTEM CURVE

New Sump C to Storage



SYSTEM HEAD CURVE

Client: Waste Management of North Carolina
Project: Piedmont Leachate New Sump C to Storage — Job No.: 87221.200
Date: 10/20/93 — Set up: Burgess

SYSTEM DATA

Friction Factors (C)		150	&	130	
Static Head:					
	Max Inlet El:	767		Min Inlet El:	764
	Min Outlet El:	762		Max Outlet El:	762
Flow rates in gpm:					
	Start	0			
	Increment	20			
	End	200	10 Data Points		
Force main length (Ft)		4,867		1	
Force main dia (in)		3		100	
FM Area (sq. ft.)		0.049		54.549	
Minor Losses:		18.8			

MINIMUM STATIC HEAD — AND C= 150 — AND C= 130

	Flows (GPM)	Static (ft)	Minor (ft)	Friction (ft)		Total (ft)	Minor (ft)	Friction (ft)		Total (ft)
1	0	−5	0.00	0	0	−5	0.00	0	0	−5
2	20	−5	0.24	6	0	1	0.24	8	0	3
3	40	−5	0.96	21	0	17	0.96	28	0	24
4	60	−5	2.16	45	0	42	2.16	58	0	56
5	80	−5	3.85	76	0	75	3.85	99	0	98
6	100	−5	6.01	115	0	116	6.01	150	0	151
7	120	−5	8.66	162	0	165	8.66	211	0	214
8	140	−5	11.79	215	0	222	11.79	280	0	287
9	160	−5	15.39	275	0	286	15.39	359	0	369
10	180	−5	19.48	342	0	357	19.48	446	0	460

MAXIMUM STATIC HEAD — AND C= 150 & 130

	Flows (GPM)	Static (ft)	Minor (ft)	Friction (ft)		Total (ft)	Minor (ft)	Friction (ft)		Total (ft)
1	0	−2	0.00	0	0	−2	0.00	0	0	−2
2	20	−2	0.24	6	0	4	0.24	8	0	6
3	40	−2	0.96	21	0	20	0.96	28	0	27
4	60	−2	2.16	45	0	45	2.16	58	0	59
5	80	−2	3.85	76	0	78	3.85	99	0	101
6	100	−2	6.01	115	0	119	6.01	150	0	154
7	120	−2	8.66	162	0	168	8.66	211	0	217
8	140	−2	11.79	215	0	225	11.79	280	0	290
9	160	−2	15.39	275	0	289	15.39	359	0	372
10	180	−2	19.48	342	0	360	19.48	446	0	463

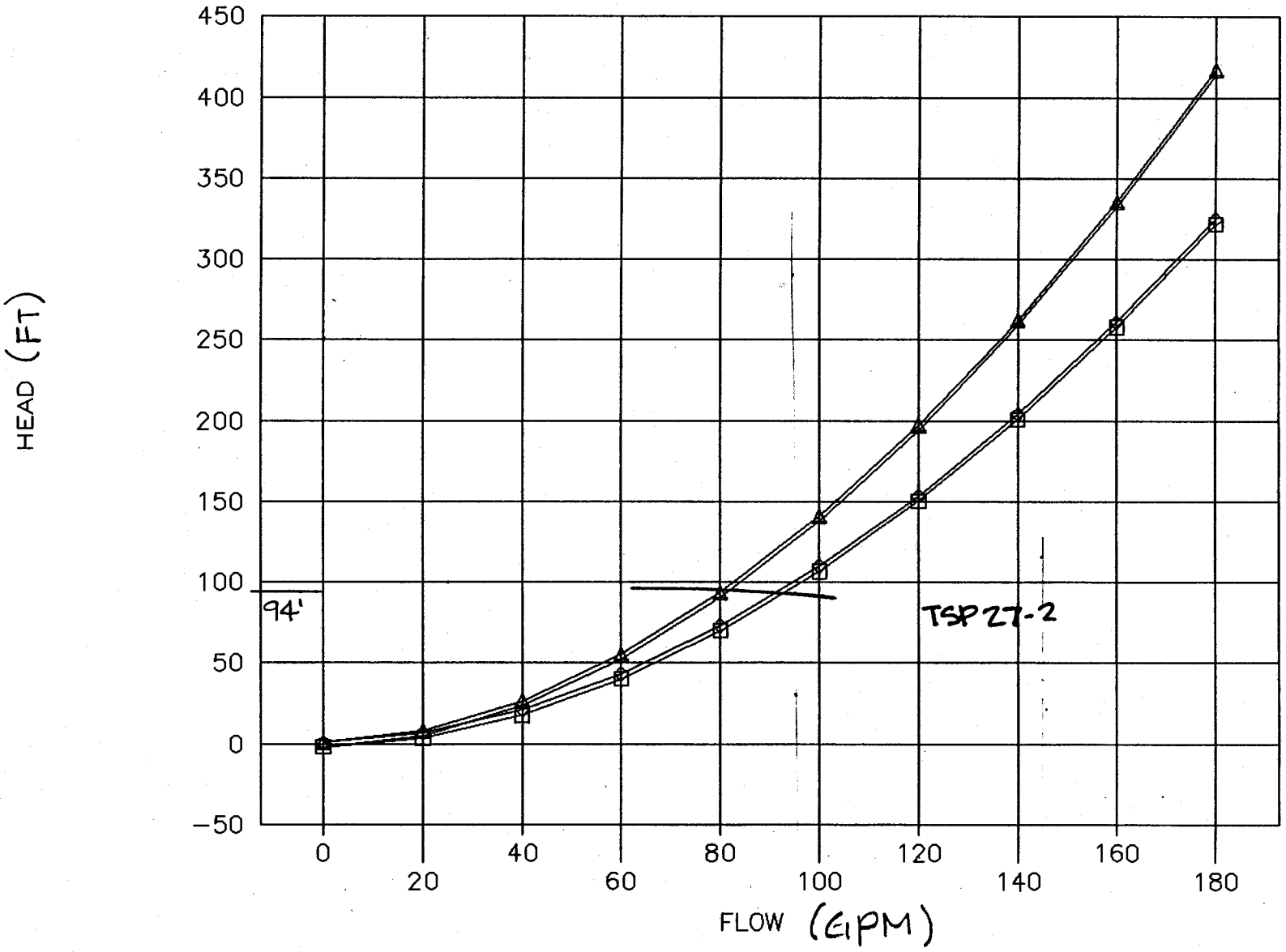
MINOR LOSS COEFFICIENTS

Valves (open)	Coef.	Quantity	Total
Check (swing)	2.5	1	2.5
Globe	10.0	0	0
Angle	5.0	0	0
Gate	0.2	8	1.6
Gate 3/4 open	1.0	0	0
Gate 1/2 open	5.6	0	0
Gate 1/4 open	24.0	0	0
Bends/Elbows			
Short rad.	0.9	0	0
Medium rad.	0.8	0	0
Long rad.	0.6	6	3.6
45 deg.	0.4	2	0.8
Closed return	2.2	0	0
Tee–through side	1.8	3	5.4
Tee–straight run	0.3	10	3
45 wye–side out	0.8	0	0
45 wye–straight run	0.3	0	0
Entrance			
square	0.5	0	0
bell	0.1	0	0
re–entrant	0.9	0	0
Exit	1.0	1	1
Coupling	0.3	3	0.9
Contractors	0.5	0	0
Others	0	0	0
Others	0	0	0
Total			18.8

GRAPH #2

SYSTEM CURVE

New Sump B to Storage



SYSTEM HEAD CURVE

Client: Waste Management of North Carolina
Project: Piedmont Leachate New Sump B to Storage — Job No.: 87221.200
Date: 10/20/93 — Set up: Burgess

SYSTEM DATA

Friction Factors (C)		150	&	130	
Static Head:					
	Max Inlet El:	764		Min Inlet El:	761
	Min Outlet El:	762		Max Outlet El:	762
Flow rates in gpm:					
	Start	0			
	Increment	20			
	End	200	10 Data Points		
Force main length (Ft)		4,339		1	
Force main dia (in)		3		100	
FM Area (sq. ft.)		0.049		54.549	
Minor Losses:		18.0			

MINIMUM STATIC HEAD — AND C= 150 — AND C= 130

	Flows (GPM)	Static (ft)	Minor (ft)	Friction (ft)		Total (ft)	Minor (ft)	Friction (ft)		Total (ft)
1	0	−2	0.00	0	0	−2	0.00	0	0	−2
2	20	−2	0.23	5	0	3	0.23	7	0	5
3	40	−2	0.92	19	0	18	0.92	25	0	24
4	60	−2	2.07	40	0	40	2.07	52	0	52
5	80	−2	3.68	68	0	70	3.68	89	0	90
6	100	−2	5.76	103	0	107	5.76	134	0	138
7	120	−2	8.29	144	0	150	8.29	188	0	194
8	140	−2	11.28	192	0	201	11.28	250	0	259
9	160	−2	14.74	245	0	258	14.74	320	0	332
10	180	−2	18.65	305	0	322	18.65	398	0	414

MAXIMUM STATIC HEAD — AND C= — 150 & 130

	Flows (GPM)	Static (ft)	Minor (ft)	Friction (ft)		Total (ft)	Minor (ft)	Friction (ft)		Total (ft)
1	0	1	0.00	0	0	1	0.00	0	0	1
2	20	1	0.23	5	0	6	0.23	7	0	8
3	40	1	0.92	19	0	21	0.92	25	0	27
4	60	1	2.07	40	0	43	2.07	52	0	55
5	80	1	3.68	68	0	73	3.68	89	0	93
6	100	1	5.76	103	0	110	5.76	134	0	141
7	120	1	8.29	144	0	153	8.29	188	0	197
8	140	1	11.28	192	0	204	11.28	250	0	262
9	160	1	14.74	245	0	261	14.74	320	0	335
10	180	1	18.65	305	0	325	18.65	398	0	417

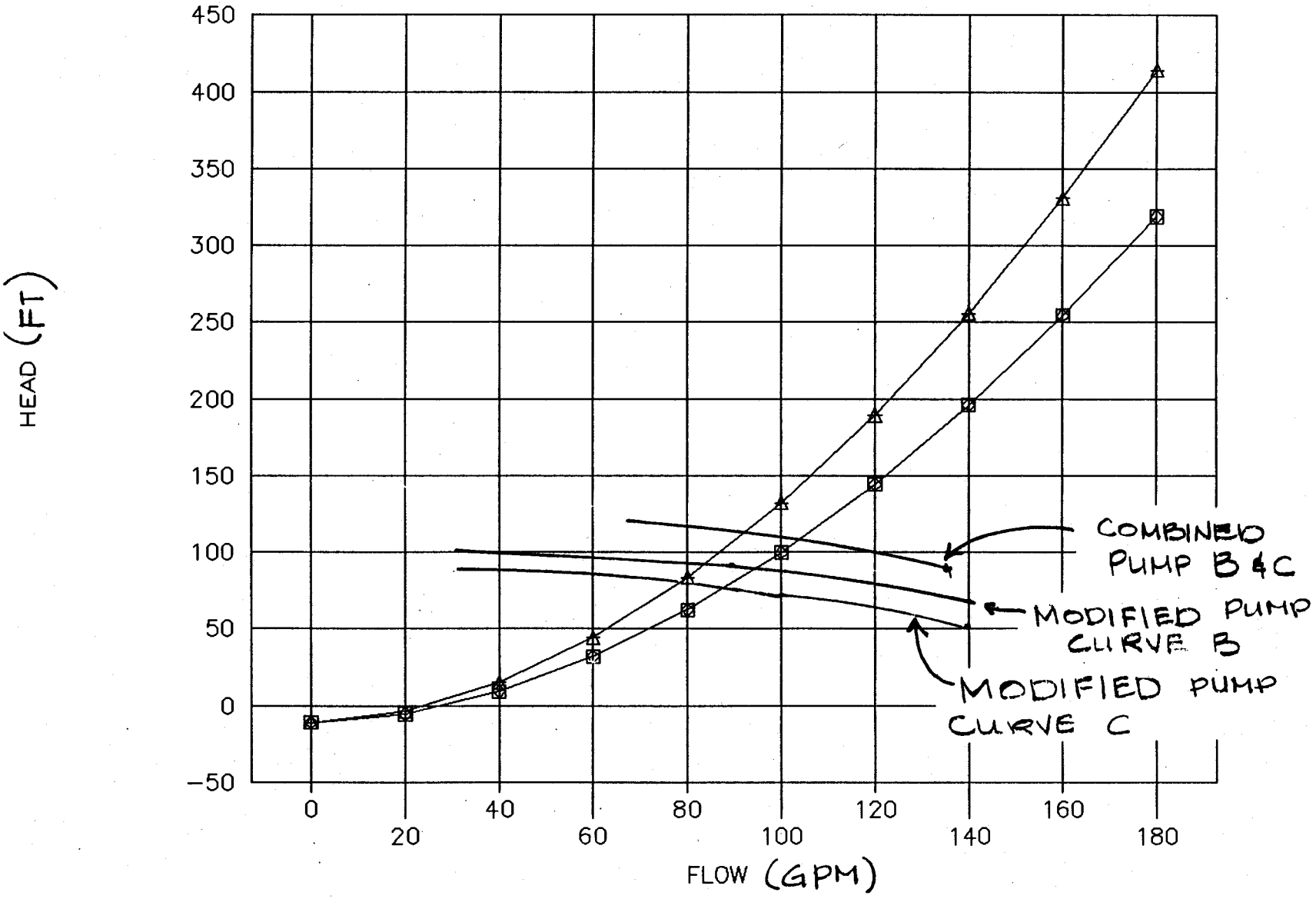
MINOR LOSS COEFFICIENTS

Valves (open)	Coef.	Quantity	Total
Check (swing)	2.5	1	2.5
Globe	10.0	0	0
Angle	5.0	0	0
Gate	0.2	7	1.4
Gate 3/4 open	1.0	0	0
Gate 1/2 open	5.6	0	0
Gate 1/4 open	24.0	0	0
Bends/Elbows			
Short rad.	0.9	0	0
Medium rad.	0.8	0	0
Long rad.	0.6	6	3.6
45 deg.	0.4	2	0.8
Closed return	2.2	0	0
Tee–through side	1.8	3	5.4
Tee–straight run	0.3	8	2.4
45 wye–side out	0.8	0	0
45 wye–straight run	0.3	0	0
Entrance			
square	0.5	0	0
bell	0.1	0	0
re–entrant	0.9	0	0
Exit	1.0	1	1
Coupling	0.3	3	0.9
Contractors	0.5	0	0
Others	0	0	0
Others	0	0	0
Total			18

GRAPH #3

SYSTEM CURVE

Manhole B to Storage



SYSTEM HEAD CURVE

Client: Waste Management of North Carolina
Project: Piedmont Leachate Manhole B to Storage Tanks — Job No.: 87221.200
Date: 10/20/93 — Set up: Burgess

SYSTEM DATA

Friction Factors (C)		150	&	130	
Static Head:					
	Max Inlet El:	773		Min Inlet El:	773
	Min Outlet El:	762		Max Outlet El:	762
Flow rates in gpm:					
	Start	0			
	Increment	20			
	End	200	10 Data Points		
Force main length (Ft)		4,470		1	
Force main dia (in)		3		100	
FM Area (sq. ft.)		0.049		54.549	
Minor Losses:		15.1			

MINIMUM STATIC HEAD — AND C= 150 — AND C= 130

	Flows (GPM)	Static (ft)	Minor (ft)	Friction (ft)		Total (ft)	Minor (ft)	Friction (ft)		Total (ft)
1	0	−11	0.00	0	0	−11	0.00	0	0	−11
2	20	−11	0.19	5	0	−5	0.19	7	0	−4
3	40	−11	0.77	19	0	9	0.77	25	0	15
4	60	−11	1.74	41	0	32	1.74	54	0	44
5	80	−11	3.09	70	0	62	3.09	91	0	83
6	100	−11	4.83	106	0	100	4.83	138	0	132
7	120	−11	6.95	148	0	144	6.95	193	0	189
8	140	−11	9.47	197	0	196	9.47	257	0	256
9	160	−11	12.36	253	0	254	12.36	329	0	331
10	180	−11	15.65	314	0	319	15.65	410	0	414

MAXIMUM STATIC HEAD — AND C= — 150 & 130

	Flows (GPM)	Static (ft)	Minor (ft)	Friction (ft)		Total (ft)	Minor (ft)	Friction (ft)		Total (ft)
1	0	−11	0.00	0	0	−11	0.00	0	0	−11
2	20	−11	0.19	5	0	−5	0.19	7	0	−4
3	40	−11	0.77	19	0	9	0.77	25	0	15
4	60	−11	1.74	41	0	32	1.74	54	0	44
5	80	−11	3.09	70	0	62	3.09	91	0	83
6	100	−11	4.83	106	0	100	4.83	138	0	132
7	120	−11	6.95	148	0	144	6.95	193	0	189
8	140	−11	9.47	197	0	196	9.47	257	0	256
9	160	−11	12.36	253	0	254	12.36	329	0	331
10	180	−11	15.65	314	0	319	15.65	410	0	414

MINOR LOSS COEFFICIENTS

Valves (open)	Coef.	Quantity	Total
Check (swing)	2.5	0	0
Globe	10.0	0	0
Angle	5.0	0	0
Gate	0.2	5	1
Gate 3/4 open	1.0	0	0
Gate 1/2 open	5.6	0	0
Gate 1/4 open	24.0	0	0
Bends/Elbows			
Short rad.	0.9	0	0
Medium rad.	0.8	0	0
Long rad.	0.6	10	6
45 deg.	0.4	2	0.8
Closed return	2.2	0	0
Tee–through side	1.8	1	1.8
Tee–straight run	0.3	12	3.6
45 wye–side out	0.8	0	0
45 wye–straight run	0.3	0	0
Entrance			
square	0.5	0	0
bell	0.1	0	0
re–entrant	0.9	0	0
Exit	1.0	1	1
Coupling	0.3	3	0.9
Contractors	0.5	0	0
Others	0	0	0
Others	0	0	0
Total			15.1

WASTE MANAGEMENT OF CAROLINAS, INC.

CONSTRUCTION QUALITY ASSURANCE PLAN

FOR

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, NORTH CAROLINA

DECEMBER 1993
REVISED APRIL 1994

Prepared By:
RUST ENVIRONMENT & INFRASTRUCTURE
GREENVILLE, SOUTH CAROLINA

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CONSTRUCTION QUALITY ASSURANCE PLAN
PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

1.0 INTRODUCTION

The North Carolina Department of Environment, Health and Natural Resources - Division of Solid Waste Management requires a program of Construction Quality Control Assurance (CQA) for various elements of the Piedmont Landfill and Recycling Center (Piedmont LRC) in Forsyth County, North Carolina. The purpose of this document is to define the scope, formal organization and procedures used to verify that work performed meets the project requirements and the CQA plan. This document also details the formal procedures and test methods utilized to ensure conformance of construction to plans, specifications or other criteria established for the project.

The CQA plan details the necessary information and guidance to verify that the components of liner, and leachate collection system for Phase 3 and final cover are constructed in compliance with approved plans and specifications. Modifications to this plan may be allowed upon Construction Quality Assurance Project Manager and Design Engineer approval. The specified quality assurance testing and measures, along with the required frequency of tests, are also provided herein. Detailed in this plan are guidelines for organization and minimum qualifications of the CQA personnel as well as the minimum standards for construction, testing and documentation to assure quality construction.

2.0 PROJECT QUALITY ASSURANCE/QUALITY CONTROL TEAM

2.1 PROJECT TEAM RESPONSIBILITIES

The project team responsible for implementing the CQA plan will be accountable to the site owner or designated owner representatives. The CQA project team will generally consist of a Construction Quality Assurance Project Manager (CQAPM), Certifying Engineer (CE), Senior Site Quality Assurance (SSQA) Technician, and an appropriate number of assistants to accomplish the work plan. The CQA team will use sound engineering judgement when determining what additional procedures, other than those outlined in this plan, may be required in order to further assure the construction quality. The CQA team will operate independently of the contractors' quality control team.

2.2 CONSTRUCTION QUALITY ASSURANCE PROJECT MANAGER (CQAPM)

The CQAPM will manage the earthwork and geosynthetics testing for CQA programs and be responsible for preparation of a CQA documentation report. The CQAPM will have knowledge of permit requirements, plans and specifications for the project. The CQAPM

is responsible that the flow of information between the SSQA Technician and CE is presented to the Contractor and Owner for problem resolutions.

The CQAPM will visit the site to inspect and document the work activities during the duration of the project. The CQAPM will be responsible for the overall administration of the quality assurance procedures and for the control of construction quality control documents. The CQAPM will be responsible for oversight and coordination of the CQA program (CE, SSQA Technician and CQA technicians), surveying and testing laboratories. The CQAPM, CE and SSQA Technician shall have daily dialogue on project status.

2.3 CERTIFYING ENGINEER (CE)

The project CE will be a professional engineer registered in the State of North Carolina. The CE shall have knowledge of plans and specifications for the project and is responsible (in conjunction with the CQAPM) for informing the contractor or landfill operator of any portion of the work or construction procedures that does not meet the design specifications.

The CE will visit the site to inspect and document the work activities during the clay liner, geomembrane liner, and leachate collection system installation to ensure that the project specifications are being met. The CE will also review all field and laboratory test data and all field reports to ensure design specification compliance. Any item requiring attention or approval will be attended to promptly by the CE and CQAPM. The CE will have the ultimate responsibility for approving and disapproving any element of the project based on the above mentioned factors and will certify the final construction is in conformance with the project plans, specifications, and the site specific CQA Plan.

2.4 SENIOR SITE QUALITY ASSURANCE TECHNICIAN (SSQA)

The SSQA Technician is the primary on-site representative of the CQA team. The SSQA Technician is responsible for performing or assuring performance of the sampling and testing required for implementation of this CQA plan. The SSQA Technician is accountable to the CQAPM. The SSQA Technician will be familiar with the tests, processes, and materials involved in the construction of compacted clay liners, leachate collection systems, geosynthetics, and any other elements involving landfill construction.

The SSQA Technician will be on site daily and monitor all appropriate construction operations. He will oversee all sampling and field testing. The SSQA Technician will log the construction activities on a daily basis. If necessary the CQAPM or CE will assume the duties and responsibilities of the SSQA Technician.

2.5 QUALITY ASSURANCE TECHNICIANS

Quality Assurance technicians will be required to assist the SSQA Technician in performance of his duties. CQA technician(s) will report directly to the SSQA Technician. All test results will be forwarded daily to the SSQA Technician to be included in the daily field report.

2.6 COMMUNICATION

To help ensure a high degree of quality during installation and assure a final product that meets all project specifications, clear, open channels of communication are essential between all parties. This section discusses appropriate lines of communication and describes all meetings that will be necessary to achieve project goals. The typical lines of communication necessary during a project will be coordinated through the project manager. Details of the communication process will be discussed at the preconstruction meeting.

A preconstruction meeting shall be held at the site prior to beginning of lining system installation. Typically, the meeting shall be attended by the CQA Project Manager, ~~r~~, Earthwork Contractor, Geosynthetic Installer, SSQA Technician and the Owner's technical representative. In addition, the Division will be invited to attend . Specific topics considered for this preconstruction meeting include review of the project CQA Plan for any problems or additions. The responsibilities of each party ~~should also~~ shall be ~~reviewed~~ established and understood clearly. During the preparation of and placement of the soil liner and the geosynthetic liner the CQA Certifying Engineer (CE) with support from the Owner or Owner's Representative (project manager) will have control of stop work due to conditions not meeting the requirements of the permit, specifications, drawings and or the site specific Construction Quality Assurance Plan. The Owner/Owner's Representative, CE, or SSQA Technician will notify the contractor of any conditions that does not meet requirements or material being installed (clay liner, geomembrane, etc.). The SSQA Technician shall keep the CE and the Owner's PM aware of site conditions on a daily basis. A recommended agenda with specific topics for the preconstruction meeting is presented in the attached example. The meeting shall be documented by a person designated at the beginning of the meeting, and minutes shall be transmitted to all parties.

A weekly progress meeting shall be held between the SSQA Technician, Earthwork Contractor's/Installer's Superintendent, Project Manager and any other concerned parties. this meeting shall discuss current progress, planned activities for the next week, issues requiring resolution, and any new business or revisions to the work. The SSQA Technician shall log any problems, decisions, or questions arising at this meeting in his weekly report. If any matter remains unresolved at the end of this meeting, the Project Manager shall be responsible for the resolution of the matter and the communication of the decision to the appropriate parties. The Project Manager may require daily progress meetings at his discretion.

3.0 SOIL TESTING

3.1 SOIL LINER SOURCE IDENTIFICATION

Soil liner material may be obtained from off-site borrow sources and/or on-site soils may be admixed with bentonite to achieve permeability requirements. Overburden soils not segregated for use in the soil subbase or soil liner will be stockpiled separately on the site and used as daily or intermediate cover. Soils will be distinguished by visual/ manual classification procedures (ASTM D-2488) and any other procedures that are appropriate.

3.2 PRE-CONSTRUCTION TESTING REQUIREMENTS

3.2.1 Soils

Soil material for use in compacted subbase and soil liner, which are stockpiled prior to construction of the landfill cells, will be tested at specified frequencies and whenever a change in material occurs. The test results will be evaluated for suitability prior to construction. The testing program shown in Table 1 for each material source will be utilized to define appropriate compaction specifications which will yield a high level of confidence that permeability requirements are met.

A soil liner test pad will be required to be constructed by the contractor using the same equipment intended for use during the construction of the actual soil subbase or clay liner in the landfill cell. This test strip will then be tested for actual field verification of the aforementioned soil properties by comparison with the laboratory test data. Correlations for the Atterberg limits, soil density, moisture content and permeability will be established. By compiling this data, the need for continuous testing is reduced. The established relationship can be used as a reference to compare the field results to predetermined moisture-density-permeability relationships. If materials change relative to the referenced lab data, a sample of the new material will be sent to the lab for evaluation and subsequently field verified using the previously described test strip method. The new test results will be added to the family of data thereby updating the reference.

3.2.2 Protective Cover Material

The protective cover materials will be placed and consist of material which will not cause damage to the underlying geosynthetic materials.

4.0 EARTHWORK

4.1 EXCAVATION

Excavation within the cell areas will proceed to the subgrade elevations provided in the permit drawings.

The SSQA Technician will verify that the excavated material is properly separated and stockpiled according to soil types. The SSQA Technician will inspect the excavation operation to verify that proper construction procedures are followed.

4.2 SUBBASE PREPARATION/CONSTRUCTION

The subbase will be constructed of suitable material as defined in the construction drawings and specifications. Unsuitable material such as organic, silt, or muck will be removed, replaced, and compacted with suitable soil material. Upon attainment of the final subbase beneath the soil liner, the entire subbase will be inspected, surveyed and approved by the SSQA Technician and CE.

4.3 CONSTRUCTION OBSERVATION

Observation of the soil liner construction will be coordinated with construction testing. At a minimum, the SSQA Technician will observe and record the following during the construction of soil
liners:

1. Moisture content and consistency of the soil during processing, placement, and compaction.

2. Type and level of compactive effort
   a. Roller type
   b. Roller weight
   c. Number of coverages

4.4 CONSTRUCTION TESTING

All construction quality assurance testing will be conducted in accordance with this CQA Plan. Field and laboratory tests will be conducted on samples taken from the soil liner materials during the course of the construction work. Testing and sampling procedures will be observed and documented by the SSQA Technician. Documentation and reporting of test results will be in accordance with the requirements identified in this CQA Plan.

4.4.1 Field Testing

The SSQA Technician will perform as a minimum, the following field tests on each lift of the compacted soil:

1. Field moisture content (ASTM D2216)
2. Field density (ASTM D2922, D1556, or D2167)
3. In-situ shear strength with pocket penetrometer.

Unless otherwise specified in the project specifications, these tests will be performed at a frequency of one per 10,000 ft^2 (1,000 m^2) area or less of each compacted lift. Sampling locations will be selected by the SSQA Technician.

Nuclear density tests (ASTM D2922) will be preferred for density (dry unit weight) testing for most projects. The location of routine in-place density tests can be determined using a non-biased sampling plan based on random selection of testing locations. Test penetrations will be backfilled with bentonite. Questions concerning the accuracy of any single test will be addressed by retesting in the same general location. Periodic checks using the Sand-Cone Method (ASTM D1556) or other standard techniques required in the project specifications may be performed to verify the nuclear density test results.

5.0 SOIL LINER/FINAL COVER

5.1 QUALITY CONTROL/QUALITY ASSURANCE PROCEDURES

Each soil liner lift will be compacted to a dry density of no less than the dry density required to attain acceptable permeability properties. Each lift thickness after compaction will no be greater than six (6) inches. Each compacted 6-inch thick lift will be tested for density and moisture content. The soil will be compacted within the specified range of water content. Loose thickness and initial (before compaction) water content will be tested at a frequency not less than one for every 10,000 sq. ft. of compacted clay area, but at least once per lift. The number and locations of the moisture-density tests will be at a frequency of one for every 10,000 sq. ft. and documented by the SSQA Technician.

The SSQA Technician may require additional testing in areas he believes may not meet specifications. Soil liner compaction should be monitored to locate and test any potential problem areas. If the compacted layer does not meet the required compactive effort, the SSQA Technician will perform additional tests, if necessary, to locate the extent of the area that needs to be recompacted and report it immediately to the CQAPM or CE and the Contractor. After corrective actions are taken, the SSQA Technician will perform additional tests to determine if the area has been brought up to design specifications.

5.2 TESTING AND ACCEPTANCE

A grab sample of soil used for the soil liner will be obtained for at least every 5,000 cy for laboratory permeability testing. In order to ensure that correct material is being used, one sample of soil will be obtained for every 5,000 yd^3 for density testing. Classification tests will be performed for every 3,000 yd^3 of clay material. The frequency of confirmatory Proctor testing may be relaxed once the CQAPM has established a family of Moisture-Density curves relating to Atterberg limits and percent fines and permeability.

The SSQA Technician will verify that the required thickness of compacted soil has been installed; this verification will be performed by surveying prior to and after placement of the compacted soil liner. The total compacted soil liner thickness will be a minimum of 24 inches with a maximum permeability of 1×10^{-7} cm/sec and a smooth rolled surface. Any core holes performed for sampling or any other purposes, will be completely plugged with bentonite pellets. Any disturbance to the surface during sampling or any other activity will also be smooth rolled. The CE will certify the completed soil liner. The approval will be accompanied by a report which includes "as-built" drawings, and a description of the testing program, test results and any other necessary documentation.

5.3 DEFECTS AND REPAIRS

5.3.1 Identification

Acceptability criteria for testing will be as identified in the CQA Plan. At locations where the testing indicates the requirements of the project specifications are not met, the SSQA Technician will determine the extent and the nature of the defect and report the findings to the CQAPM or CE.

The SSQA Technician will confirm that no stones are present which may damage overlying geosynthetic components. If the compacted soil liner has been subject to adverse weather conditions, the SSQA Technician will reexamine the soil for possible damage.

5.3.2 Notification

After determining the extent and nature of any defect, the SSQA Technician will promptly notify the Earthwork Contractor and the CQAPM or CE. A work deficiency meeting will be held as needed between the Earthwork Contractor, SSQA Technician, Designer, and other appropriate parties to assess the problem, review alternative solutions, and implement an action plan.

5.3.3 Repairs and Retesting

The Earthwork Contractor will correct all deficiencies to meet the project specifications.

The SSQA Technician will schedule appropriate retests when the work defect has been corrected. All retests by the SSQA Technician will verify that the defect has been corrected before any additional work is performed by the Earthwork Contractor in the area of the deficiency.

The SSQA Technician will observe any repair and report any noncompliance with the above requirements in writing to the CQAPM or CE.

5.4 SOIL LINER TEST PAD

The purpose of a test pad is to verify that the materials and installation requirements specified in the project plans and specifications to be used during construction of soil liners can meet the required performance standard. This test pad program will also allow the Earthwork Contractor, Designer, Owner, and CQAPM to identify appropriate placement and compaction procedures. Once the detailed construction procedures have been established during the test pad program, the Earthwork Contractor and the SSQA Technician will monitor the soil liner construction procedures as an indicator that the performance standards are being achieved. This test pad program may be modified based on site-specific design, and construction.

5.4.1 Construction Preparation

5.4.1.1 Test Pad Material

Source evaluation testing of the proposed soil liner material shall be performed prior to construction of the test pad. The material shall be evaluated in accordance with Section 5.4.3.1 of this document. These tests shall provide the basic relationship of hydraulic conductivity with varying water content, dry unit weight and compactive effort (Figure 1).

5.4.1.2 Construction Equipment

The equipment to be used for a test pad shall be proposed by the Earthwork Contractor and approved by the CQAPM, CE and SSQA Technician. The equipment to be used for the test pad shall be consistent with the equipment which will be used during liner construction.

5.4.1.3 Subgrade Preparation

The area within the limits of the test pad shall be cleared and grubbed of all trees, brushes, stumps, roots, debris, trash, and any other vegetation or objectionable material. Following clearing and grubbing, the area shall be stripped of topsoil.

The surface of the subgrade shall be proof-rolled so as to be free of soft zones, irregularities, loose earth, and abrupt changes in grade. The subgrade and test pad shall be sloped at an approximate 2 percent grade or as sloped in the cell. Line and grades shall be controlled by survey. No standing water or excessive moisture shall be allowed on the surface of the subgrade. The surface shall be observed by the SSQA Technician prior to beginning construction of the test pad.

5.4.2 Test Pad Construction

The test pad shall be a rectangle approximately 100 feet long by 50 feet wide (Figure 2). The test pad shall be constructed in uniform horizontal lifts with a total thickness consistent with the project plans and specifications. The construction procedures, which vary with the lift considered, are intended to allow determination of a relationship between soil measurements and compaction method parameters. Soil measurements include water content, dry unit weight, and hydraulic conductivity. Compaction method parameters includes compactor characteristics, thickness of compacted/uncompacted layers, number of compactor passes, and water content.

5.4.2.1 First Lift

A. Should the test pad be built outside the cell being constructed a geotextile or drainage layer shall be placed by the Earthwork Contractor over the subgrade area of the test pad. The geotextile or a drainage layer should extend beyond the edge of the test pad allowing an outlet for test pad infiltration.

B. The first lift of test pad material shall be placed to a thickness resulting in 6 inches after compaction or as required in the project specifications.

C. Soil water content shall be maintained in the Acceptable Zone as determined during source evaluation testing. The Earthwork Contractor shall adjust the water content as necessary to obtain the specified density criteria.

D. The test pad material shall be compacted with two one-way coverages using the previously agreed upon compaction equipment.

E. The Earthwork Contractor shall allow the SSQA Technician to perform in-place measurements of moisture content and density tests and collect soil samples.

F. Holes in each lift shall be repaired with a 50% soil and 50% bentonite mixture.

G. The test pad material shall be compacted a second time by applying two more one-way coverages with the selected compactor.

H. A second series of tests shall be taken near the original tests. Repeat Steps E and F.

I. The test pad material shall be compacted a third time by applying two more, one-way coverages.

J. A third series of tests shall be taken near the first and second tests. Repeat Steps E and F.

K. Steps I and J shall be repeated and continued until specified compaction criteria are obtained as identified by the SSQA Technician.

5.4.2.2 Second Lift

A. The loose thickness of the second lift shall be such that the thickness of the compacted lift will be 6 inches after compaction or as required by the project specifications.

B. A competent bond with the first lift shall be achieved by the Earthwork Contractor and observed by the SSQA Technician.

C. Steps C through K of Section 5.4.2.1shall be repeated.

5.4.2.3 Remaining Lifts

A. The loose thickness of the remaining lifts shall be such that the thickness of the lifts will be 6 inches after compaction or as required by the project specifications.

B. The procedures for compacting and testing the remaining lifts shall be those that have been tested and proven effective during the compaction of the second lift.

5.4.3 Testing and Observations

5.4.3.1 Construction Preparation

The SSQA Technician shall perform testing on the soil liner material prior to its use in the test pad. Source evaluation testing will include at a minimum the following unless otherwise specified in the project specifications:

- Water Content (ASTM D2216)
- Particle Size (ASTM D422, D1140)
- Atterberg Limits (ASTM D4318)

- Laboratory Compaction (ASTM D698 for standard Proctor or ASTM D1557 for modified Proctor)
- Laboratory Hydraulic Conductivity at a specified compaction (ASTM D5084)
- Soil Classification (ASTM D2487)

The SSQA Technician shall observe the prepared subgrade for firmness, smoothness, and absence of abrupt changes in grade. The subgrade shall be surveyed to serve as the origin for determining thicknesses.

5.4.3.2 Test Pad Construction

The SSQA Technician shall perform tests and make observations during the construction of the test pad to enable development of a curve providing in-place dry unit weight versus number of compactor coverages (Figure 3). Test sample locations shall be selected by the Soil QAC. An example of test sample locations and configuration is provided in Figure 4.

During the construction of the first and second lifts, the SSQA Technician shall perform the following activities:

- Estimate the thickness of the loose lifts

- Count the number of compactor coverages and observe compactor coverage of the test pad

- Perform a minimum of eight nuclear gauge in-place measurements of water content and dry unit weight (ASTM D2922) for every two compactor coverages, and a minimum of one in-place test using the sand-cone method (ASTM D1556) to verify the nuclear gauge readings; compute degree of compaction (in-place dry unit weight divided by the Standard Proctor maximum dry unit weight); collect four additional soil samples for moisture content determination (ASTM D2216)

- Observe the repair of holes left in the lift as a result of testing and soil sample collection

During construction of the remaining lifts, the Soil QAC shall perform the following activities:

- Verify that the thickness of the loose lift thickness determined from testing of the second lift

- Count the number of compactor coverages, determined from testing of the second lift, which are necessary to achieve the compaction in the Acceptable Zone and observe compactor coverage of the test pad

- Perform a minimum of eight tests nuclear gauge in-place measurements (ASTM D2922) and one sand-cone test (ASTM D1556) per lift to verify the adequacy of the construction procedures previously established

During the construction of the test pad, the SSQA Technician shall collect a minimum of six undisturbed Shelby tube samples from varying depths of the completed test pad. The samples shall be waxed or otherwise protected to retain natural water. The samples should be tested in the laboratory for the following:

- Water Content (ASTM D2216)
- Particle Size (ASTM D422)
- Atterberg Limits (ASTM D4318)
- Laboratory Hydraulic Conductivity (ASTM D5084)
- Soil Classification (ASTM D2487)

The SSQA Technician shall observe the test pad to verify the adequacy of the bonding between adjacent lifts. Such observation shall be exercised on the portion of the test pad which has been excavated to permit removal of undisturbed soil block samples and/or the sand-cone density testing.

5.4.3.3 Final Surface Preparation

The Soil QAC shall observe the prepared surface for firmness, smoothness, and absence of abrupt changes in grade. The final surface will be surveyed to verify the test pad thickness.

5.4.4 Documentation

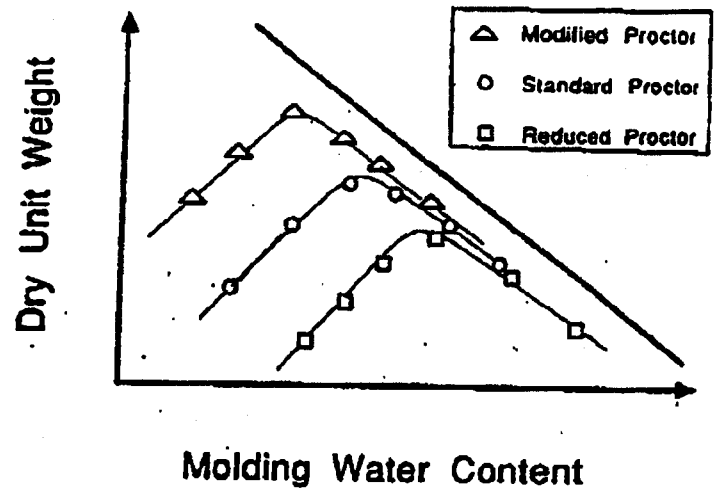
The SSQA Technician shall document activities associated with the preparation, construction, testing, and observation of the test pad and provide recommendations on placements and compaction procedures. A documentation summary report shall be prepared that contains information ont he test pad material, construction equipment, subgrade preparation, test pad construction, and testing and observations. The report shall also include daily reports of construction activities and oral communications with the Earthwork Contractor, CQAPM and CE.

The SSQA Technician shall provide a water content/dry unit weight relationship for the test pad material and other source material test results. The SSQA Technician shall document testing and observations on subgrade preparation, test pad construction, and final surface preparation, including but not limited to:

- Compactor type, configuration, and weight for sheepsfoot compactors, drum diameter and length, empty and ballasted weight, length and face area of feet, and yoking arrangement

- Thicknesses of lifts prior to and after compaction
- Dry unit weight versus number of compactor coverages for each lift thickness, as specified in Section 5.4.3.2
- Number of compactor coverages which will provide the specified compaction in the Acceptable Zone and hydraulic conductivity that meets the performance standard
- Procedure to bond lifts
- Recommended installation procedures
- Results of water content, in-place density and degree of compaction
- Description of repair of holes left in the lift as a result of density testing and soil sample collection, as specified in Section 5.4.3.2
- Results of laboratory hydraulic conductivity testing and other soil property tests performed on undisturbed soil samples
- Drawings of the test pad and locations of all test samples for each lift
- Cross sections of the test pad showing number of lifts and lift thickness
- Description of actual construction procedures
- Test pad excavation for removal of undisturbed soil samples and observations of layer bonding.

The SSQA Technician, CE and CQAPM shall submit the final documentation summary report to the Owner. The Owner shall submit the report to the NCDEHNR.



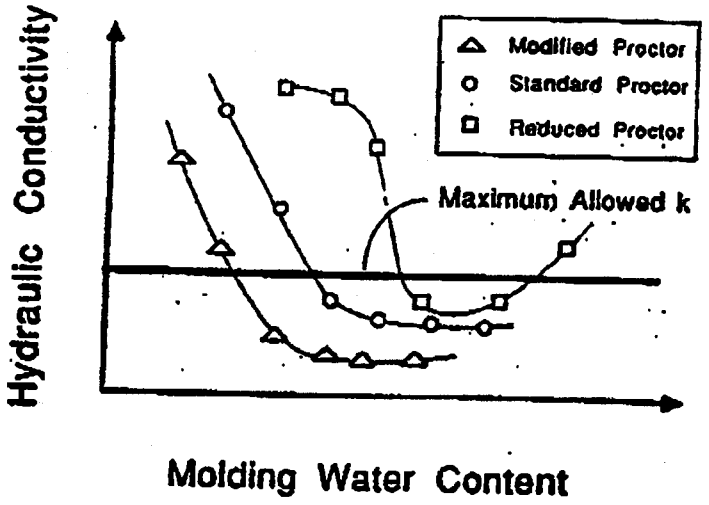


FIGURE 1 – COMPACTION CURVES AND HYDRAULIC CONDUCTIVITY CURVES

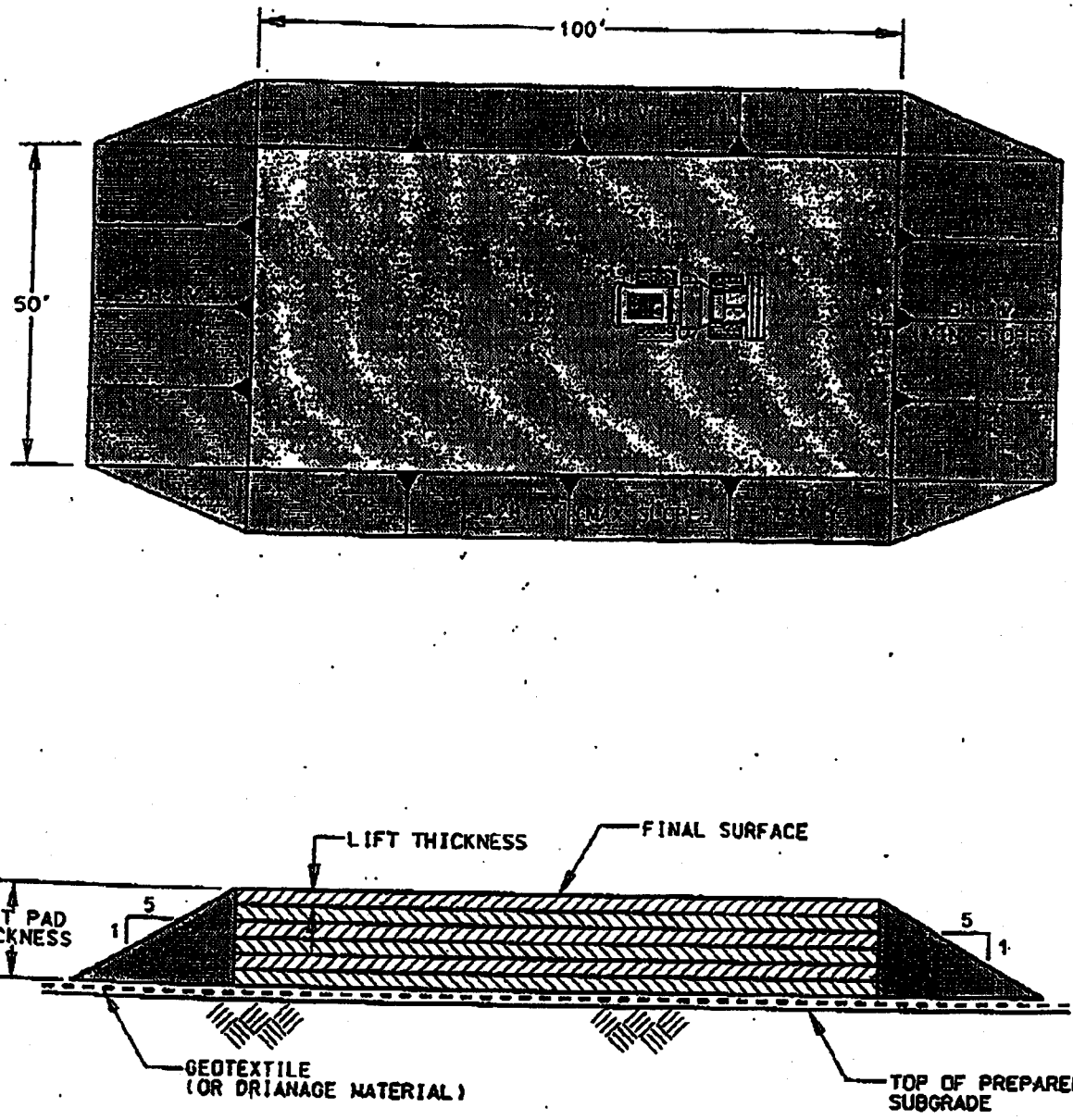


FIGURE 2 – TYPICAL TEST PAD CONFIGURATION

NTS

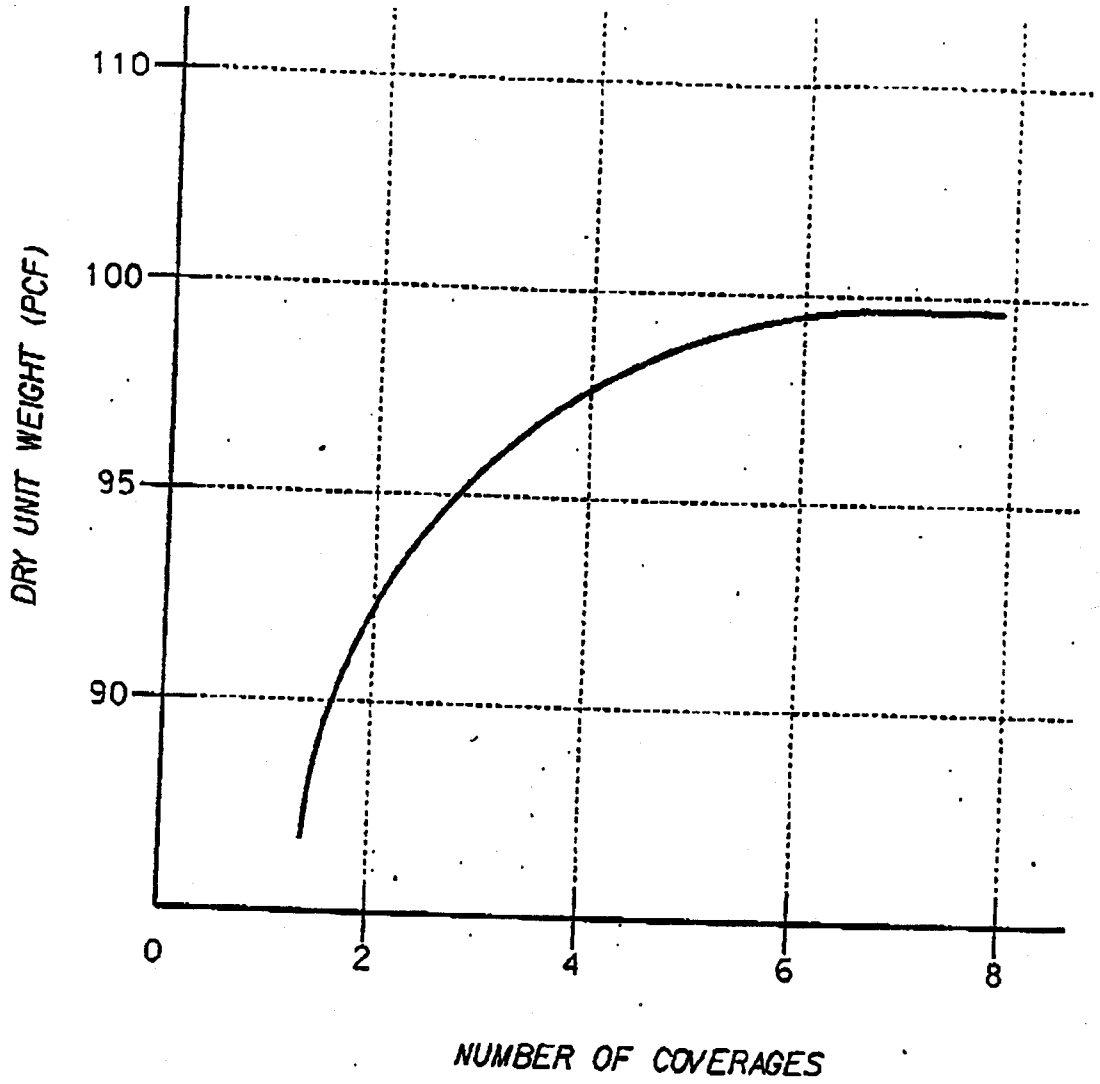
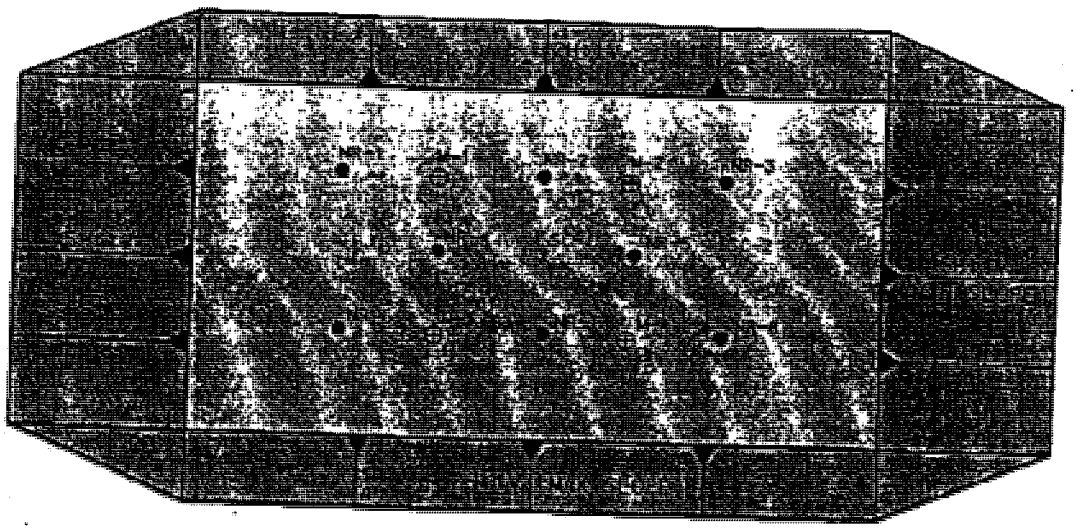
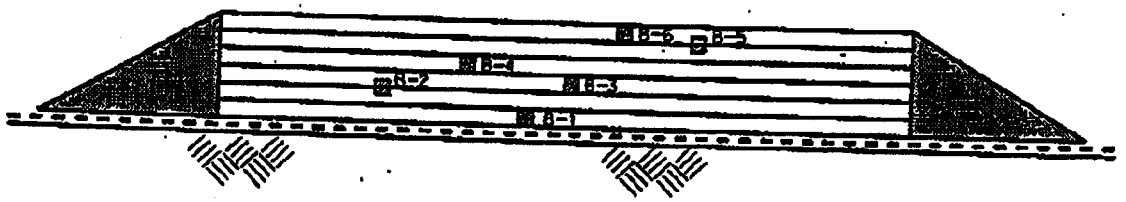


FIGURE 3 – EXAMPLE RELATIONSHIP BETWEEN DRY UNIT WEIGHT AND NUMBER OF COMPACTOR COVERAGES





LEGEND

- ● NR-1 — NUCLEAR GUAGE READING ON DENSITY AND MOISTURE (8 MIN/LIFT)
- x SC-1 — SAND CONE TEST (2 MIN/LIFT)
- ⊕ M-1 — MOISTURE CONTENT SAMPLE (4 MIN/LIFT))
- ■ B-1 — SHELBY TUBE OR BLOCK SAMPLE FOR LABORATORY TESTING (6 MIN PER TEST FILL AT VARYING DEPTHS)

FIGURE 4 – TYPICAL TEST SAMPLING CONFIGURATION
NTS

6.0 GEOMEMBRANE LINER (~~Primary~~ Liner/Final Closure Cap)

6.1 DESCRIPTION

A high density polyethylene (HDPE) liner is specified for this project.

6.2 QUALITY CONTROL DOCUMENTATION

Prior to the installation of any geomembrane, the Manufacturer or Installer will provide the Project Manager with the following information:

1. The origin (supplier's name and production plant) and identification (brand name and number) of the resin used to manufacture the geomembrane.
2. Copies of dated quality control certificates issued by the resin supplier.
3. Results of tests conducted by the Manufacturer to verify that the resin used to manufacture the geomembrane meets the project specifications.
4. A statement indicating that the amount of reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness.
5. A list of the materials which comprise the geomembrane, expressed in the following categories as percent by weight: polyethylene, carbon black, other additives.
6. A specification for the geomembrane which includes all properties contained in the project specifications measured using the appropriate test methods.
7. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.
8. Quality control certificates, signed by a responsible party employed by the Manufacturer. Each quality control certificate will include roll identification numbers, testing procedures, and results of quality control tests. At a minimum, results will be given for:

    a. Density (ASTM D1505)
    b. Carbon black content (ASTM D1603)
    c. Carbon black dispersion (ASTM D3015)
    d. Thickness (ASTM D751)
    e. Tensile properties (ASTM D638)
    f. Puncture resistance (ASTM D4833)

    These quality control tests will be performed in accordance with the test methods for every 50,000 ft^2 (5,000 m^2).

9. Results of environmental stress crack resistance tests (GM-5). At a minimum, tests will be performed once every resin lot.

The Manufacturer will identify all rolls of geomembranes with the following:

1. Manufacturer's name
2. Product identification
3. Thickness
4. Roll number
5. Roll dimensions

The CQAPM & CE will review these documents and will report any discrepancies.

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.
2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.
3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.
4. Rolls are appropriately labeled.
5. Certified minimum properties meet the project specifications.

6.3 CONFORMANCE TESTING

Upon delivery of the rolls of the geomembrane, conformance test samples will be obtained from the geomembrane. The geomembrane rolls to be sampled will be selected by the SSQA Technician. Samples will be taken across the entire width of the roll judged by the SSQA Technician not to be damaged. Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The SSQA Technician will mark the machine direction on the samples with an arrow.

A lot will be defined as a group of consecutively numbered rolls from the same manufacturing line. Alternatively, a lot may be designated by the SSQA Technician based on a review of all roll information including quality control documentation and manufacturing records.

Samples will be taken at a rate of one per lot and not less than one per 100,000 ft^2 (10,000 m^2) of geomembrane. These samples will be forwarded to the Geosynthetic Laboratory for testing.

The following conformance tests will be conducted:

1. Density (ASTM D1505)
2. Carbon black content (ASTM D1603)
3. Carbon black dispersion (ASTM D3015)
4. Thickness (ASTM D751)
5. Tensile properties (ASTM D638)

All conformance test results will be reviewed and accepted or rejected by the CE and SSQA Technician prior to the deployment of the geomembrane. The CE and SSQA Technician will examine all results from laboratory conformance testing. CE and SSQA Technician will be responsible for checking that all test results meet or exceed the property values listed for the project.

If a test result is in nonconformance, all material from the lot represented by the failing test will be considered out-of-specification and rejected. Additional conformance test samples may be taken to "bracket" the portion of the lot not meeting the project specification. This procedure is valid only when all rolls in the lot are consecutively produced and numbered from one manufacturing line. To isolate the out-of-specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) will be rejected. If one or both of the additional tests fail, then the entire lot will be rejected.

6.4 SOIL LINER PREPARATION

The Earthwork Contractor will be responsible for preparing the underlying soil liner prior to geomembrane placement.

Before the geomembrane installation begins, the SSQA Technician will verify that:

1. A licensed land surveyor has verified all lines and grades.
2. The CE has verified that the underlying soil meets the criteria specified in the project specifications.
3. The underlying soil liner surface to be lined has been rolled, compacted, or hand-worked so as to be free of irregularities, protrusions, loose soil, and abrupt changes in grade.
4. The surface of the underlying soil liner does not contain stones which may be damaging to the geomembrane.
5. There is no area excessively softened by high water content.
6. There is no area where the underlying soil surface contains desiccation cracks which may damage the geomembrane.

The Installer will certify in writing that the surface on which the geomembrane will be installed is acceptable. A certificate of acceptance will be given by the Installer to the SSQA Technician and the Earthwork Contractor prior to commencement of geomembrane deployment in the area under consideration.

After the underlying soil has been accepted by the Installer, it is the Installer's responsibility to indicate to the SSQA Technician any change in the underlying soil condition that requires repair work.

At any time before or during the geomembrane installation, the SSQA Technician will indicate to the Earthwork Contractor any locations which may not be adequately prepared for the geomembrane.

6.5 ANCHOR TRENCH

The SSQA Technician will verify:

1. The anchor trench has been constructed according to the project plans and specifications.
2. If the anchor trench is excavated in a clay material susceptible to desiccation, the amount of trench open at any time is minimized. The SSQA Technician will inform the Earthwork Contractor of any signs of significant desiccation associated with the anchor trench construction.
3. Rounded corners are provided in the trench so as to avoid sharp bends in the geomembrane.
4. Excessive amounts of loose soil are not allowed to underlie the geomembrane in the anchor trench.
5. The anchor trench is adequately drained to prevent ponding or softening of the adjacent soils while the trench is open.
6. The anchor trench is backfilled with cohesive soils and compacted as specified.

Care will be taken when backfilling the trenches to prevent any damage to the geosynthetic components. The SSQA Technician will observe the backfilling operation and advise the Earthwork Contractor. Any problems will be documented by the SSQA Technician in his daily report.

6.6 GEOMEMBRANE DEPLOYMENT

6.6.1 Panel Nomenclature

A field panel is defined as a unit of geomembrane which is to seamed in the field. A field panel is a roll or a portion of a roll cut in the field. The SSQA Technician will be responsible to ensure that each field panel is given an identification code (number or letter-number) consistent with the layout plan. This identification code will be as simple and logical as possible and will be agreed upon with the Installer.

In general, it is not appropriate to identify panels using roll numbers since roll numbers established in the manufacturing plant are usually cumbersome and are not related to location in the field. The SSQA Technician will establish a table or chart showing correspondence between roll numbers and field panel identification codes. The field panel identification code will be used for all quality assurance records.

6.6.2 Panel Deployment Procedure

The CE and SSQA Technician will review the panel deployment progress of the Installer as to compliance with the approved panel layout drawing. The CE and SSQA Technician will also review the panel deployment for suitability to actual field condition such as issues relating to wind, rain, soil liner desiccation and other site-specific conditions. Once approved, changes to the panel deployment procedure will not be taken unless changed in a formal meeting of all parties. The SSQA Technician will verify that the condition of the underlying soil does not change detrimentally during installation.

The SSQA Technician will verify that field panels are installed at the locations indicated on the Installer's layout plan. The SSQA Technician will record the identification code, location, and date of installation of each field panel.

6.6.3 Deployment Weather Conditions

Geomembrane deployment will not be undertaken if weather conditions will preclude material seaming following deployment.

The normal required weather conditions for seaming are as follows:

1. Ambient temperature between 32 °F (0 °C) and 104 °F (40 °C).
2. Dry conditions (no precipitation or other excessive moisture)
3. No excessive winds.

The CE or SSQA Technician will verify that these weather conditions are fulfilled and notify in writing if they are not. Ambient temperature will be measured by the SSQA Technician in the area in which the panels are to be placed.

The SSQA Technician will inform the Installer of any weather-related problems which may not allow geomembrane placement to proceed.

6.6.4 Method of Deployment

Before the geomembrane is handled on site, the CE and SSQA Technician will verify that handling equipment to be used on the site is adequate and does not pose risk of damage to the geomembrane. During handling, the SSQA Technician will observe and verify that the Installer's personnel handle the geomembrane with care.

The CE and SSQA Technician will verify the following:

1. Equipment used does not damage the geomembrane by handling.
2. The prepared surface underlying the geomembrane is acceptable immediately prior to geomembrane placement.

3. Geosynthetic elements immediately underlying the geomembrane are clean and free of debris.
4. Personnel do not smoke or wear damaging shoes while working on the geomembrane, or engage in other activities which could damage the geomembrane.
5. The method used to unroll the panels does not cause excessive scratches or crimps in the geomembrane and does not damage the supporting soil.
6. The method used to place the panels minimizes wrinkles, especially differential wrinkles between adjacent panels.
7. Adequate temporary loading and/or anchoring (such as sand bags or tires), not likely to damage the geomembrane, are placed to prevent uplift by wind. In case of high winds, continuous loading is recommended along edges of panels to minimize risk of wind flow under the panels.
8. Direct contact with the geomembrane is minimized, and the geomembrane is protected by geotextiles, extra geomembrane, or other suitable materials, in areas where excessive traffic may be expected.

Conditions differing from the above will be brought to the attention of the Installer or Earthwork Contractor.

6.6.5 Damage and Defects

Upon delivery to the site, the SSQA Technician will conduct a surface observation of all rolls for defects and for damage. This examination will be conducted without unrolling rolls unless defects or damages are found or suspected. The SSQA Technician will advise the Installer of any rolls or portions of rolls which should be rejected and removed from the site because they have severe flaws, and/or minor repairable flaws.

The SSQA Technician will examine each panel, after placement and prior to seaming, for damage and/or defects. The SSQA Technician will advise the Installer which panels, or portions of panels, should be rejected, repaired, or accepted. Damaged panels, or portions of damaged panels, which have been rejected will be marked and their removal from the work area recorded by the SSQA Technician.

6.6.6 Writing on the Liner

To avoid confusion, the Installer and the SSQA Technician will each use different colored markers that are readily visible for writing on the geomembrane. The markers used must be semi-permanent and compatible with the geomembrane. The Installer will use a white marker to write on the geomembrane while the SSQA Technician will use a yellow marker.

6.7 FIELD SEAMING

6.7.1 Seam Layout

Before installation begins, the Installer will provide the SSQA Technician with a panel layout drawing. This drawing will present all the proposed seams of the lining system at the facility. The CE and SSQA Technician will review the panel layout drawing and verify that it is consistent with accepted state of practice. In addition, panels not specifically shown on the panel layout drawing may not be used without prior approval.

In general, seams should be oriented parallel to the line of maximum slope, thus, oriented along, not across, the slope. In corners and odd-shaped geometric locations, the number of seams should be minimized. No horizontal seam should be less than 5 ft (1.5 m) from the toe or crest of the slope, or areas of potential stress concentrations, unless otherwise authorized by the Design Engineer.

A seam numbering system compatible with the panel numbering system will be used by the SSQA Technician.

6.7.2 Accepted Seaming Methods

Approved processes for field seaming are fusion welding and extrusion welding. Only apparatus which have been specifically approved by make and model will be used. Submit all documentation regarding seaming methods to be used to the CQAPM and CE.

6.7.2.1 Fusion Process

The SSQA Technician will log ambient, seaming apparatus, and geomembrane surface temperatures at appropriate intervals and report any noncompliances to the Installer.

The SSQA Technician will also verify that:

1. The Installer maintains on-site a minimum of 2 spare operable seaming apparatus.
2. Equipment used for seaming is not likely to damage the geomembrane.
3. The electric generator is placed on a smooth base such that no damage occurs to the geomembrane.
4. A smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage such that no damage occurs to the geomembrane.
5. A movable protective layer is used as required by the Installer directly below each overlap of geomembrane that is to be seamed to prevent buildup of moisture between the sheets and to prevent debris from collecting around the pressure rollers.

6. In general, the geomembrane panels are aligned to have an overlap of 4 to 6 in (100 mm to 150 mm) for fusion welding. In any event, the final overlap will be sufficient to allow peel tests to be performed on the seam.
7. No solvent or adhesive is used.
8. The geomembrane is protected from damage in heavy traffic areas.

6.7.2.2 Extrusion Process

The SSQA Technician will log ambient, seaming apparatus, and geomembrane surface temperatures at appropriate intervals.

The SSQA Technician will verify that:

1. The Installer maintains on-site a minimum of two spare operable seaming apparatus.
2. Equipment used for seaming is not likely to damage the geomembrane.
3. Prior to beginning a seam, the extruder is purged until all heat-degraded extrudate has been removed from the barrel.
4. Clean and dry welding rods or extrudate pellets are used.
5. The electric generator is placed on a smooth base such that no damage occurs to the geomembrane.
6. Grinding is completed no more than one hour prior to seaming.
7. A smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage such that no damage occurs.
8. The geomembrane is protected from damage in heavy traffic areas.
9. Exposed grinding marks adjacent to an extrusion weld will be minimized. In no instance will exposed grinding marks extend more than 1/4 in (6 mm) from the finished seamed area.
10. In general, the geomembrane panels are aligned to have a nominal overlap of 3 in (75 mm) for extrusion welding. In any event, the final overlap will be sufficient to allow peel tests to be performed on the seam.
11. No solvent or adhesive is used.
12. The procedure used to temporarily bond adjacent panels together does not damage the geomembrane; in particular, the temperature of hot air at the nozzle of any temporary welding apparatus is controlled such that the geomembrane is not damaged.

6.7.3 Seam Preparation

The SSQA Technician will verify that prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris or foreign material of any kind. If seam overlap grinding is required, SSQA Technician must ensure that the process is completed according to the Manufacturer's instructions within one hour of the seaming operation, and in a way that does not damage the geomembrane. The SSQA Technician will also verify that seams are aligned with the fewest possible number of wrinkles and "fishmouths."

6.7.4 Trial Seams

Trial seams will be made on fragment pieces of geomembrane liner to verify that conditions are adequate for production seaming. Such trial seams will be made at the beginning of each seaming period, and at least once each five hours, for each production seaming apparatus used that day. Trial seams will be made under the same conditions as production seams.

The trial seam sample will be at least 5 ft (1.6 m) long by 1 ft (0.3 m) wide (after seaming) with the seam centered lengthwise. Seam overlap will be as indicated in Section 6.7.2. Two specimens will be cut from the sample with a 1 in (25 mm) wide die. The specimens will be cut by the Installer at locations selected randomly along the trial seam sample by the SSQA Technician.

The specimens will be tested in peel using a field tensiometer. The tensiometer will be capable of maintaining a constant jaw separation rate of two inches per minute. If a specimen fails, the entire trial seam operation will be repeated. If the additional specimen fails, the seaming apparatus and seamer will not be accepted and will not be used for seaming until the deficiencies are corrected and two consecutive successful trial seams are achieved. The SSQA Technician will observe all trial seam procedures.

The remainder of the successful trial seam sample will be retained in the Owner archives for possible laboratory testing. Each sample will be assigned a number and marked accordingly by the SSQA Technician, who will also log the date, hour, ambient temperature, number of seaming unit, name of seamer, and pass or fail description.

If a trial seam sample fails a test conducted by the SSQA Technician a destructive seam test sample will be taken from each of the seams completed by the seamer during the shift related to the subject trial seam. These samples will be forwarded to the Geosynthetic Laboratory and, if they fail the tests, the procedure indicated in Section 6.9.7 will apply. The conditions of this paragraph will be considered satisfied for a given seam if a destructive seam test sample has already been taken.

6.7.5 General Seaming Procedures

During general seaming, the SSQA Technician will ensure the following:

1. Fishmouths or wrinkles at the seam overlaps will be cut along the ridge of the wrinkle in order to achieve a flat overlap. The cut fishmouths or wrinkles will be seamed and any portion where the overlap is inadequate will then be patched with an oval or round patch of the same geomembrane extending a minimum of 6 in (150 mm) beyond the cut in all directions.
2. Seaming will extend to the outside edge of panels placed in the anchor trench.

3. All cross seam tees should be extrusion welded to a minimum distance of 4 in (100 mm) on each side of the tee.
4. No field seaming will take place without the Master Seamer being present.
5. A firm substrate may be required to be provided by using a flat board, a conveyor belt, or similar hard surface directly under the seam overlap to achieve proper support.

The SSQA Technician will verify that the above seaming procedures are followed, and will report any nonconformance.

6.7.6 Seaming Weather Conditions

At ambient temperatures above 104 °F, no seaming of the geomembrane will be permitted unless the Installer can demonstrate to the satisfaction of the CE and SSQA Technician that geomembrane seam quality is not compromised. Trial seaming, as described in Section 6.7.4 will be conducted under the same ambient temperature conditions as the production seams. At the option of the SSQA Technician, additional destructive tests may be required for any suspect areas.

6.8 NONDESTRUCTIVE SEAM TESTING

6.8.1 Concept

The Installer will nondestructively test all field seams over their full length using an air pressure test (for double fusion seams only), a vacuum test or other approved method. Air pressure testing and vacuum testing are described in Sections 6.8.2 and 6.8.3, respectively. The purpose of nondestructive tests is to check the continuity of seams. It does not provide quantitative information on seam strength. Nondestructive testing will be carried out as the seaming work progresses, not at the completion of all field seaming.

For all seams, the SSQA Technician:

1. Observe nondestructive testing procedures.
2. Record location, data, test unit number, name of tester, and outcome of all testing.
3. Inform the Installer of any required repairs.

Any seams that cannot be nondestructively tested will be cap-stripped with the same geomembrane. The cap-stripping operations will be observed by the SSQA Technician and Installer for uniformity and completeness.

6.8.2 Air Pressure Testing

Air pressure testing is applicable to double fusion welding which produces a double seam with an enclosed space.

1. The equipment for air pressure testing will consist of the following:

   a. An air pump (manual or motor driven), equipped with pressure gauge and capable of generating and sustaining a pressure between 24 and 35 psi (160 and 240 kPa) and mounted on a cushion to protect the geomembrane.
   b. A rubber hose with fittings and connections.
   c. A sharp hollow needle, or other pressure feed device.

2. The following procedures will be followed:

   a. Seal both ends of the seam to be tested.
   b. Insert needle or other approved pressure feed device into the air channel created by the fusion weld.
   c. Insert a protective cushion between the air pump and the geomembrane.
   d. Pressurize the air channel to a pressure of approximately 30 psi (200 kPa). Close valve, allow 2 minutes for pressure to stabilize, and sustain pressure for at least 5 minutes.
   e. If loss of pressure exceeds 3 psi, or does not stabilize, locate faulty area and repair in accordance with Section 6.10.3.
   f. Cut opposite end of tested seam area once testing is completed to verify continuity of the air channel. If air does not escape, locate blockage and retest unpressurized area. Seam the cut end of the air channel.
   g. Remove needle or other approved pressure feed device and seal the hole in the geomembrane.

6.8.3 Vacuum Testing

Vacuum testing is applicable to extrusion welding.

1. The equipment will consist of the following:

   a. A vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, a porthole or valve assembly, and a vacuum gauge.
   b. A pump assembly equipped with a pressure controller and pipe connections.
   c. A rubber pressure/vacuum hose with fittings and connections.
   d. A soapy solution.
   e. A bucket and side paint brush, or other means of applying the soapy solution.

2. The following procedures will be followed:

   a. Wet a strip of geomembrane approximately 12 in x 48 in (0.3 m x 1.2 m) with the soapy solution.
   b. Place the box over the wetted area.

c. Close the bleed valve and open the vacuum valve.
d. Ensure that a leak-tight seal is created.
e. Energize the vacuum pump and reduce the applied pressure to approximately 5 psi (10 in of Hg/35 kPa) gauge.
f. For a minimum of 10 seconds, apply vacuum with the box placed and maintaining a seal, examine the geomembrane through the viewing window for the presence of soap bubbles.
g. If no bubble appears after 10 seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3 in (75 mm) overlap, and repeat the process.
h. All areas where soap bubbles appear will be marked and repaired in accordance with Section 6.10.3.

6.8.4 Test Failure Procedures

The Installer will complete any required repairs in accordance with Section 6.10. For repairs, the SSQA Technician will:

1. Observe the repair and testing of the repair.
2. Mark on the geomembrane that the repair has been made.
3. Document the repair procedures and test results.

6.9 DESTRUCTIVE SEAM TESTING

6.9.1 Concept

The purpose of destructive tests is to evaluate seam strength. Destructive seam tests will be performed at selected locations. Seam strength testing will be done as the seaming work progresses, not at the completion of all field seaming.

6.9.2 Location and Frequency

The SSQA Technician will select where seam samples will be cut out for laboratory testing. The frequency and locations will be established as follows:

1. A minimum frequency of one test location per 500 ft (150 m) of seam length performed by each welding machine. This frequency is to be determined as an average taken throughout the entire facility.
2. Test locations will be determined during seaming at the SSQA Technician's discretion. Special consideration will be given to locations where the potential for imperfect welding, such as overheating, contamination, offset welds exists.

The Installer will not be informed in advance of the locations where the seam samples will be taken.

6.9.3 Sampling Procedures

Samples will be cut by the Installer at locations chosen by the SSQA Technician as the seaming progresses so that laboratory test results are available before the geomembrane is covered by another material. The SSQA Technician will:

1. Observe sample cutting.
2. Assign a number to each sample, and mark it accordingly.
3. Record sample location on layout drawing.
4. Record reason for taking the sample at this location (e.g., statistical routine, suspicious feature of the geomembrane).

All holes in the geomembrane resulting from destructive seam sampling will be repaired in accordance with repair procedures described in Section 6.10.3immediately following receipt of successful test results. The continuity of the new seams in the repaired area will be tested according to Section 6.8.3.

6.9.4 Sample Dimensions

At each sampling location, two types of samples will be taken by the Installer. First, two specimens for field testing should be taken. Each of these samples will be cut with a 1 in (25 mm) wide die, with the seam centered parallel to the width. The distance between these two samples will be 42 in (1.1 m). If both samples pass the field test described in Section 6.9.5,a sample for laboratory testing will be taken.

The sample for laboratory testing will be located between the samples for field testing. The sample for laboratory testing will be 12 in (0.3 m) wide by 42 in (1.1 m) long with the seam centered lengthwise. The sample will be cut into three parts and distributed as follows:

1. One 12 in wide x 18 in long (0.3 m x 0.5 m) portion for Geosynthetic Laboratory for testing.
2. One 12 in wide x 12 in long (0.3 m x 0.3 m) portion to the Installer for optional laboratory testing.
3. One 12 in wide x 12 in long (0.3 m x 0.3 m) portion to the Owner for archive storage.

6.9.5 Field Testing

The two 1 in (25 mm) wide specimens mentioned in Section 6.7.4and Section 6.9.4will be tested in the field using a tensiometer for peel adhesion and will not fail according to the criteria in Table 3B. The tensiometer will be capable of maintaining a constant jaw separation rate of two inches per minute. If it fails, the seam should be repaired in accordance with Section 6.9.7. Final judgements regarding seam acceptability, based on the failure criteria provided in the Table 3B, rests with the SSQA Technician.

The SSQA Technician will witness all field tests and mark all samples and portions with their number. The SSQA Technician will also log the date and time, ambient temperature, number of seaming unit, name of seamer, welding apparatus temperatures and pressures, and pass or fail description, and attach a copy to each sample portion.

6.9.6 Laboratory Testing

Destructive test samples will be packaged and shipped, to the Geosynthetic Laboratory in a manner which will not damage the test sample. The sample will be shipped as soon as possible to expedite laboratory testing. The SSQA Technician will collect and turnover to the Owner archive samples.

Testing will include seam strength and peel adhesion (ASTM D4437). The minimum acceptable values to be obtained in these tests will be per Table 3B. At least 5 specimens will be tested, each in both shear and peel. Specimens will be selected alternately by test from the samples (i.e., peel, shear, peel, shear). A passing test will meet the minimum acceptable values in at least 4 of the 5 specimens tested for each method.

The Geosynthetic Laboratory will provide test results within 24 hours of receiving the samples. The SSQA Technician will review laboratory test results as soon as they become available, and make appropriate recommendations to the CE and Owner.

6.9.7 Destructive Test Failure

When a sample fails a destructive test, whether that test is conducted by the Geosynthetic Laboratory or by field tensiometer, the Installer has two options:

1. The Installer can repair the seam between any two passing destructive test locations.
2. The Installer can trace the welding path to an intermediate location 10 ft (3 m) minimum from the point of the failed test in each direction and take a sample with a 1 in (25 mm) wide die for an additional field test at each location. If these additional samples pass the test, then full laboratory samples are taken. If these laboratory samples pass the tests, then the seam is repaired between these locations. If either sample fails, then the process is repeated to establish the zone in which the seam should be repaired.

All acceptable repaired seams will be bound by two locations from which samples passing laboratory destructive tests have been taken. Passing laboratory destructive tests of trial seam samples taken as indicated in Section 6.7.4 may be used as a boundary for the failing seam. In cases exceeding 150 ft (50 m) of repaired seam, a sample taken from the zone in which the seam has been repaired must pass destructive testing. Repairs will be made in accordance with Section 6.10.

The SSQA Technician will document all actions taken in conjunction with destructive test failures.

6.10 DEFECTS AND REPAIRS

6.10.1 Identification

All seams and non-seam areas of the geomembrane will be examined by the SSQA Technician for identification of defects, holes, blister, undispersed raw materials, large wrinkles and any sign of contamination by foreign matter. The geomembrane surface will be cleaned by the Installer prior to examination if the SSQA Technician determines that the amount of dust, water, or mud inhibits examination.

6.10.2 Evaluation

Each suspect location both in seam and non-seam areas will be nondestructively tested using the methods described in Section 6.8. Each location which fails the nondestructive testing will be marked by the SSQA Technician and repaired by the Installer. Work will not proceed with any materials which will cover locations which have been repaired until successful nondestructive and/or laboratory tests are obtained.

When seaming of the geomembrane is completed, and prior to placing overlying materials, the SSQA Technician will indicate to the Installer any large wrinkles which should be cut and reseamed. The number of wrinkles to be repaired should be kept to an absolute minimum. Therefore, wrinkles should be located during the coldest part of the installation period, while keeping in mind the forecasted weather to which the uncovered geomembrane may be exposed. Wrinkles are considered to be large when the geomembrane can be folded over on to itself which is generally a wrinkle that extends 12 in (0.3 m) from the subgrade. Seams produced while repairing wrinkles will be nondestructively tested.

When placing overlying material on the geomembrane, every effort must be made to minimize wrinkle development. If possible, cover should be placed during the coolest weather. In addition, small wrinkles should be isolated and covered as quickly as possible to prevent their growth. The placement of cover materials will be observed by the SSQA Technician to ensure that wrinkle formation is minimized and that, in all cases, the geomembrane is not folded over on itself.

6.10.3 Repair Procedures

Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, will be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure will be agreed upon between the Installer, Designer, and CE or SSQA Technician.

1. The repair procedures available include:

   a. Patching, used to repair holes, tears, undispersed raw materials, and contamination by foreign matter.
   b. Spot welding used to repair pinholes, or other minor, localized flaws.
   c. Capping, used to repair large lengths of failed seams.
   d. Extrusion welding the flap, used to repair areas of inadequate fusion seams which have an exposed edge. Repairs of this type will be approved by the SSQA Technician and will not exceed 100 ft (30 m) in total length.
   e. Removing bad seam and replacing with a strip of new material welded into place.

2. For any repair method, the following provisions will be satisfied:

   a. Surfaces of the geomembrane which are to be repaired using extrusion methods will be ground no more than one hour prior to the repair.
   b. All surfaces will be clean and dry at the time of the repair.
   c. All seaming equipment used in repairing procedures will meet the requirements of the project specifications.
   d. Patches or caps will extend at least 6 in (150 mm) beyond the edge of the defect, and all corners of patches will be rounded with a radius of approximately 3 in (75 mm).

6.10.4 Repair Verification

The SSQA Technician will observe all nondestructive testing of repairs and will record the number of each repair, date and test outcome. Each repair will be nondestructively tested using the methods described in Section 6.8 as appropriate. Repairs which pass the nondestructive test will be taken as an indication of an adequate repair. Repairs more than 150 ft (50 m) long require destructive test sampling. Failed tests require that the repair will be redone and retested until a passing test results.

When placing overlying material on the geomembrane, every effort must be made to minimize wrinkle development. If possible, cover should be placed during the coolest weather available. In addition, small wrinkles should be isolated and covered as quickly as possible to prevent their growth. The placement of cover materials will be observed by the SSQA Technician to ensure that wrinkle formation is minimized and that, in all cases, the geomembrane is not folded over on itself.

6.11 GEOMEMBRANE PROTECTION

The quality assurance procedures indicated in this Section are intended only to assure that the installation of adjacent materials does not damage the geomembrane. The quality assurance of the adjacent materials themselves are covered in separate Sections.

6.11.1 Soils

A copy of the project specifications prepared by the Designer for placement of soils will be given to the CE or SSQA Technician. The CE and SSQA Technician will verify that these project specifications are consistent with geosynthetic state-of-practice such as:

1. Placement of soils on the geomembrane will not proceed at an ambient temperature below 32 °F (0 °C) nor above 104 °F (40 °C) unless otherwise specified.
2. Placement of soil on the geomembrane should be done during the coolest part of the day to minimize the development of wrinkles in the geomembrane.
3. Equipment used for placing soil will not be driven directly on the geomembrane.
4. A minimum thickness of 1 ft (0.3 m) of soil is specified between a light dozer, ground pressure of 5 psi (35 kPa) or lighter, and the geomembrane.
5. In any areas traversed by heavy construction, any vehicles other than low ground pressure vehicles approved by the SSQA Technician, the soil layer will have a minimum thickness of 3 ft (0.9 m). This requirement may be waived if provisions are made to protect the geomembrane through an engineered design. Drivers will proceed with caution when on the overlying soil and prevent spinning of tires or sharp turns.

The SSQA Technician will measure soil thickness and verify that the required thickness is present. The SSQA Technician must also verify that final thickness is consistent with the design and verify that placement of the soil is done in such a manner that geomembrane damage is unlikely.

6.11.2 Sumps and Appurtenances

A copy of the plans and project specifications prepared by the Designer for sumps and appurtenances will be given to the CQAPM and CE. The CQAPM and CE will review these plans and verify that:

1. Installation of the geomembrane in sump and appurtenant areas, and connection of geomembrane to sumps and appurtenances have been made according to project specifications.
2. Extreme care is taken while welding around appurtenances since neither non-destructive nor destructive testing may be feasible in these areas.
3. The geomembrane has not been visibly damaged while making connections to sumps and appurtenances.
4. A representative of the SSQA Technician will be present at all times when the Installer is welding geomembrane to appurtenant structures.

7.0 GEOTEXTILES

7.1 DESCRIPTION

Geotextiles are used in protection and filtering applications in this lining system.

7.2 QUALITY CONTROL DOCUMENTATION

Prior to the installation of any geotextile, the Manufacturer or Installer will provide the Project Manager with the following information:

1. The origin (resin supplier's name and resin production plant) and identification (brand name and number) of the resin used to manufacture the geotextile.
2. Copies of dated quality control certificates issued by the resin supplier.
3. Results of tests conducted by the Manufacturer to verify that resin used to manufacture the geotextile meets the project specifications.
4. Results of quality control tests conducted by the Manufacturer to verify that the geotextile manufactured for the project meets the project specifications.
5. A statement indicating that the reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness.
6. A list of the materials which comprise the geotextile, expressed in the following categories as percent by weight: base polymer, carbon black, other additives.
7. A specification for the geotextile which includes all properties published by the Manufacturer, measured using the appropriate test methods.
8. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.
9. Written certification that the Manufacturer has continuously inspected the geotextile for the presence of needles and found the geotextile to be needle-free.
10. Quality control certificates will include roll identification numbers, testing procedures and results of quality control tests. At a minimum, results will be given for:

    a. Mass per unit area (ASTM D3776)
    b. Grab strength (ASTM D4632)
    c. Trapezoidal tear strength (ASTM D4533)
    d. Burst strength (ASTM D3786)
    e. Puncture strength (ASTM D4833)

    These quality control tests will be performed in accordance with the test methods for at least every 100,000 ft^2 (10,000 m^2) of geotextile produced.

The Manufacturer will identify all rolls of geotextiles with the following:

1. Manufacturer's name
2. Product identification

3. Roll number
4. Roll dimensions

The CQAPM and CE will review these documents and will report any discrepancies. The CQAPM and CE will verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.
2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.
3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.
4. Roll packages are appropriately labeled.
5. Certified minimum roll properties meet the project specifications.

7.3 CONFORMANCE TESTING

Upon delivery of the rolls of geotextiles, conformance test samples will be obtained for the geotextile. The rolls to be sampled will be selected by the SSQA Technician. Samples will be taken from any portion of a roll which has not been deemed damaged by the SSQA Technician. Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The SSQA Technician will mark the machine direction on the samples with an arrow. All lots of material and the particular test sample that represents each lot should be defined before the samples are taken.

A lot will be defined as a group of consecutively numbered rolls from the same manufacturing line. Alternatively, a lot may be designated by the SSQA Technician based on a review of all roll information including quality control documentation and manufacturing records.

Samples will be taken at a rate of one per lot and not less than one per 100,000 ft^2 (10,000 m^2) of geotextile. These samples will then be forwarded to the Geosynthetic Laboratory for testing.

At a minimum, the following conformance tests will be performed on geotextiles:

1. Mass per unit area (ASTM D3776)
2. Grab strength (ASTM D4632)
3. Trapezoidal tear strength (ASTM D4533)
4. Burst strength (ASTM D4833)
5. Puncture strength (ASTM D4833)

All conformance test results will be reviewed and accepted or rejected by the SSQA Technician and CE prior to the deployment of the geotextile. The CE and SSQA

Technician will examine all results from laboratory conformance testing. The CE and SSQA Technician will be responsible for checking that all test results meet or exceed the property values listed for the project.

If a test result is in nonconformance, all material from the lot represented by the failing test should be considered out-of-specification and rejected. Additional conformance test samples may be taken to "bracket" the portion of the lot not meeting project specifications. To isolate the out-of-specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) will be rejected. If one of the additional tests fail, then the entire lot will be rejected or the procedure repeated with two additional tests that bracket greater number of rolls within the lot.

7.4 GEOTEXTILE DEPLOYMENT

During shipment and storage, the geotextile will be protected from ultraviolet light exposure, moisture, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions. Geotextile rolls will be shipped and stored in relatively opaque and watertight wrappings. Wrappings will not be removed until shortly before deployment.

The SSQA Technician will observe rolls upon delivery at the site.

The Installer will ensure that geotextiles are not damaged during handling. The geotextile will be deployed as described below:

1. On slopes, the geotextiles will be securely anchored and then rolled down the slope in such a manner as to continually keep the geotextile sheet in tension.
2. In the presence of wind, all geotextiles will be weighted with sandbags or the equivalent. Such sand bags will be installed during deployment and will remain until replaced with cover material.
3. Geotextiles will be cut using a geotextile cutter (hook blade) only. If in place, special care will be taken to protect other materials from damage which could be caused by the cutting of the geotextiles.
4. The Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geotextile.
5. During placement of geotextiles, care will be taken not to entrap, in or beneath the geotextile, stones, excessive dust, or moisture that could damage the geomembrane, cause clogging of drains or filters, or hamper subsequent seaming.
6. A visual examination of the geotextile will be carried out over the entire surface, after installation, to ensure that no potentially harmful foreign objects, such as needles, are present.

Conditions differing from the above will be brought to the attention of the Installer.

7.5 SEAMING PROCEDURES

Geotextiles will be overlapped a minimum of 3 in (75 mm) prior to seaming. In general, no horizontal seams will be allowed on sideslopes (seams along, not across, the slope) except as part of a patch. When horizontal seams are necessary, adjacent seams will be staggered horizontally.

On slopes steeper than 10:1 (horizontal:vertical), all geotextiles will be continuously sewn. Spot sewing is not allowed. On bottoms and slopes shallower than 10:1, geotextiles will be continually sewn or thermally bonded with the written approval of the Designer.

Any sewing will be done using polymeric thread with chemical and ultraviolet light resistance properties equal to or exceeding those of the geotextile. The color of the sewing thread will contrast the background color of the geotextile. Sewing will be done using machinery and stitch types agreed upon by the Installer, SSQA Technician and CE.

7.6 DEFECTS AND REPAIRS

7.6.1 Identification

If a defect is identified in the geotextile, the SSQA Technician will determine the extent and nature of the defect. If the defect is indicated by unsatisfactory test result, the SSQA Technician and CE will determine the extent of the deficient area by additional tests, observations, a review of records and other means that the SSQA Technician deems appropriate.

7.6.2 Notification

After determining the extent and nature of the defect, the SSQA Technician will promptly notify the CE, Installer and Project Manager. A work deficiency meeting will be held as required between the Installer, CE or SSQA Technician, Designer, and Project Manager to assess the problem, review alternative solutions, and implement an action plan.

7.6.3 Repair Procedures

The final decision as to the appropriate repair will be agreed upon between the Project Manager, Installer, Designer, and CE.

Any holes or tears in the geotextile will be repaired using the following two procedures.

On sideslopes, a patch made from the same geotextile will be thermally bonded or sewn into place in accordance with the project specifications. Should any tear exceed 10% of the width of the roll, that roll will be removed from the slope and replaced.

On non-sideslope areas, a patch made from the same geotextile will be thermally bonded or sewn into place with a minimum of 12-inch overlap in all directions. Care will be taken to remove any soil or other material which may have penetrated the torn geotextile.

The SSQA Technician will observe any repair.

7.7 GEOTEXTILE PROTECTIONS

All soil materials located on top of a geotextile will be deployed in such a manner as to ensure:

1. The geotextile and underlying lining materials are not damaged.
2. Minimal slippage of the geotextile on underlying layers occurs.
3. No excess tensile stresses occur in the geotextile.

8.0 GEONETS

8.1 DESCRIPTION

Geonets are geosynthetic nets used as a drainage medium in lining systems.

8.2 QUALITY CONTROL DOCUMENTATION

Prior to the installation of any geotextile, the Manufacturer or Installer will provide the Project Manager with the following information:

1. The origin (supplier's name and production plant) and identification (brand name and number) of the resin.
2. Copies of dated quality control certificates issued by the resin supplier.
3. Results of tests conducted by the Manufacturer to verify that resin used to manufacture the geonet meets the project specifications.
4. A statement indicating that the amount of reclaimed polymer added to the resin during manufacturing was done with appropriate cleanliness and for geonet used for leachate collection applications.
5. A list of the materials which comprise the geonet, expressed in the following categories as percent by weight: polyethylene, carbon black, other additives.
6. A specification for the geonet which includes all contained in the project specifications measured using the appropriate test methods.
7. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.
9. Quality control certificates, signed by a responsible party employed by the Manufacturer. The quality control certificates will include roll identification numbers, sampling procedures and results of quality control tests. At a minimum, results will be given for:

a. Density (ASTM D1505)
b. Mass per unit area (ASTM D3776)
c. Thickness (ASTM D751)
d. Carbon black content (ASTM D1603)

Quality control tests will be performed in accordance with the test methods for every 40,000 ft^2 (4,000 m^2) of geonet produced.

The Manufacturer will identify all rolls of geonets with the following:

1. Manufacturer's name
2. Product identification
3. Roll number
4. Roll dimensions

The CQAPM and CE will review these documents and will report any discrepancies. The CQAPM and CE will verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.
2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.
3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.
4. Roll packages are appropriately labeled.
5. Certified minimum roll properties meet the project specifications.

8.3 CONFORMANCE TESTING

Upon delivery of the rolls of geonet, the SSQA Technician will ensure that conformance test samples are obtained for the geonet. The rolls to be sampled will be selected by the SSQA Technician. Samples will be taken from any portion of a roll which has not been deemed damaged by the SSQA Technician. Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The SSQA Technician will mark the machine direction on the samples with an arrow.

A lot will be defined as a group of consecutively numbered rolls from the same manufacturing line. Alternatively, a lot may be designated by the SSQA Technician based on a review of all roll information including quality control documentation and manufacturing records.

Samples will be taken at a rate of one per lot and not less than one per 100,000 ft^2 (10,000 m^2) of geonet. These samples will then be forwarded to the Geosynthetic Laboratory for testing.

At a minimum, the following conformance tests will be performed:

1. Density (ASTM D1505)
2. Mass per unit area (ASTM D3776)
3. Thickness (ASTM D751)

All conformance test results will be reviewed and accepted or rejected by the SSQA Technician and CE prior to the deployment of the geonet. The CE and SSQA Technician will examine all results from laboratory conformance testing. The CE and SSQA Technician will be responsible for checking that all test results meet or exceed the property values listed for the project.

If a test result is in nonconformance, all material from the lot represented by the failing test should be considered out-of-specification and rejected. Additional conformance test samples may be taken to "bracket" the portion of the lot not meeting project specifications. To isolate the out-of-specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the (next larger roll number) will be rejected. If one of the additional tests fail, then the entire lot will be rejected or the procedure repeated with two additional tests that bracket a greater number of rolls within the lot.

8.4 GEONET DEPLOYMENT

The SSQA Technician will examine rolls upon delivery.

The geonet rolls should be protected against dust and dirt during shipment and storage since geonet cleanliness is essential to its performance. The geonet rolls will be delivered wrapped in plastic to protect the geonet from contamination. The SSQA Technician will verify that the geonet is free of dirt and dust prior to installation. The SSQA Technician will report any rolls judged dirty to the Installer. The Installer will clean geonet prior to installation. Cleaning operations will be observed by the SSQA Technician.

The Installer will handle all geonet in such a manner as to ensure that it is not damaged in any way, and the following will be complied with:

1. On slopes, the geonet will be secured and rolled down the slope in such a manner as to continually keep the geonet sheet in tension. If necessary, the geonet will be positioned by hand being unrolled to minimize wrinkles.
2. In the presence of wind, all geonet will be weighted with sandbags or the equivalent. Such sand bags will be installed during deployment and will remain until replaced with cover material.
3. Unless otherwise specified, geonet will not be welded to geomembrane.

4. Geonet will be cut using scissors or other cutting tools approved by the Project Manager that will not damage the underlying geosynthetics. Care will be taken not to leave tools in the geonet.
5. The Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geonet.
6. During placement of geonet, care will be taken not to entrap dirt, excessive dust, or fugitive bentonite clay in the geonet that could cause clogging of the drainage system, and/or stones that could damage the adjacent geomembrane. If dirt or excessive dust is entrapped in the geonet, it should be cleaned prior to placement of the next material on top of it. In this regard, care will be taken with the handling of sandbags, to prevent puncturing the sand bag.

Conditions differing from the above will be brought to the attention of the Installer.

8.5 SEAMING AND OVERLAPS

At a minimum, the following requirements for joining the adjacent geonet will be met:

1. Adjacent rolls will be overlapped by at least 4 in (100 mm).
2. The geonet overlaps will be tied with plastic fasteners. Tying devices will be white or yellow for easy inspection. Metallic devices are not allowed.
3. Tying will be every 5 ft (1.5 m) along the length at the adjacent rolls, every 6 in (0.15 m) in the anchor trench and every 6 in (0.15 m) along end-to-end seams.
4. In general, no horizontal seams will be allowed on sideslopes.
5. In the corners of the sideslopes of rectangular landfills, where overlaps between perpendicular geonet strips are required, an extra layer of geonet will be unrolled along the slope, on top of the previously installed geonet, from top to bottom of the slope.
6. When more than one layer of geonet is installed, joints will be staggered.
7. When several layers of geonet are stacked, rolls will be deployed in the same direction to prevent strands of one layer from penetrating the channels of the adjacent layer.

8.6 DEFECTS AND REPAIRS

8.6.1 Identification

If a defect is identified in the geonet, the SSQA Technician will determine the extent and nature of the defect. If the defect is indicated by unsatisfactory test result, the SSQA Technician will determine the extent of the deficient area by additional tests, observations, a review of records and other means that the SSQA Technician deems appropriate.

8.6.2 Notification

After determining the extent and nature of the defect, the SSQA Technician will promptly notify the CE, Installer and Project Manager. A work deficiency meeting will be held as required between the Installer, CE or SSQA Technician, Designer, and Project Manager to assess the problem, review alternative solutions, and implement an action plan.

8.6.3 Repair Procedures

The final decision as to the appropriate repair will be agreed upon between the Project Manager, Installer, Designer, and CE.

If the hole or tear width is less than 50% of the width of the roll, the damaged area will be repaired as follows: procedures.

1. A patch will be placed extending 1 ft (0.3 m) beyond the edges of the hole or tear.
2. The patch will be secured to the original geonet by tying every 6 in (0.15 m). Tying devices will be as indicated in Section 8.6.

If the hole or tear width across the roll is equal to or more than 50% of the width of the roll, the damaged area will be repaired as follows:

1. On the base of the landfill, the damaged area will be cut out and the two portions of the geonet will be joined as indicated in Section 8.6.

The SSQA Technician will observe any repair.

8.7 GEONET PROTECTIONS

Soil should never be placed in direct contact with geonet. Soil materials near the geonet will be placed in such a manner as to ensure:

1. The geonet and underlying lining materials are not damaged.
2. Minimal slippage of the geonet on underlying layers occurs.
3. No excess tensile stresses occur in the geonet.

9.0 GEOTEXTILE/GEONET COMPOSITE

9.1 DEFINITION AND APPLICABILITY

Geotextile/geonet composites are geocomposites used as a filter and drainage media in lining systems. This Section is applicable to drainage geocomposites made of polyester or polypropylene nonwoven geotextiles and high density polyethylene (HDPE) geonet. The

geotextiles may be bonded to one side or both sides of the geonet. This Section is not applicable to geocomposites made with other material or components. The specific type of geocomposite shall be specified in the project specifications.

9.2 MANUFACTURING PLANT INSPECTION

The Owner or appropriate representative will conduct a periodic inspection of the geocomposite Manufacturer's plant. In addition, the Project Manager or his designated representative, may visit the manufacturing plant for a project-specific inspection if deemed necessary. If possible, the project-specific inspection shall be prior to or during the manufacturing of the geotextile/geonet composite rolls for that particular project. The purpose of the plant inspection is to review the manufacturing process and quality control procedures.

The manufacturing plant inspection shall include:

1. Verification that the proper quality control documentation has been received by the Manufacturer from the component manufacturers.
2. Verification that properties guaranteed by the Manufacturer are met and meet all project specifications.
3. Verification that the measurement of properties by the Manufacturer is properly documented and test methods used are acceptable.
4. Spot inspection of the rolls and verification that they are free of imperfections or contamination by foreign matter.
5. Review of packaging, handling, storage, and transportation procedures and verification that these procedures will not damage the geocomposite.
6. Verification that roll packages have a label indicating the name of the manufacturer, type of geocomposite, roll number, and roll dimensions.

A report describing the inspection will be retained by the Owner for periodic inspections and by the Project Manager for project-specific inspections.

9.3 QUALITY CONTROL DOCUMENTATION

Prior to the installation of any geocomposite, the geocomposite Manufacturer or Installer shall provide the Project Manager with the following information:

1. The origin (supplier's name and production plant) and identification (brand name and number) of the geotextile and geonet used to fabricate the geocomposite.
2. Copies of dated quality control certificates issued by the geotextile and geonet supplier. These certificates shall contain the results of the quality control tests performed on the geocomposite components.

3. A specification for the geocomposite which includes all properties published by the Manufacturer measured using the appropriate test methods.
4. Written certification that minimum values given in the specification are guaranteed by the Manufacturer.
5. Quality control certificates for the geocomposite, signed by a responsible party employed by the Manufacturer. The quality control certificates shall include roll identification numbers, testing procedures and results of quality control tests. At a minimum, results shall be given for:

    a. Mass per unit area (ASTM D3776)
    b. Thickness (ASTM D1777)
    c. Geotextile-geonet adhesion (ASTM D413)

    Quality control tests shall be performed in accordance with the test methods for at least every 40,000 ft^2 (4,000 m^2) of geocomposite produced.

The Manufacturer shall identify all rolls of geocomposite with the following:

1. Manufacturer's name
2. Product identification
3. Roll number
4. Roll dimensions

The SSQA Technician shall review these documents and shall report any discrepancies with the above requirements to the Project Manager. The SSQA Technician shall verify that:

1. Property values certified by the Manufacturer meet all of its guaranteed specifications.
2. Measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.
3. Quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it.
4. Roll packages are appropriately labeled.
5. Certified minimum roll properties meet the project specifications.
6. Project specifications and the CQA plan were submitted by the Project Manager to the Installer.

9.4 CONFORMANCE TESTING

9.4.1 Sampling Procedures

Upon delivery of the rolls of geocomposite, the SSQA Technician shall ensure that conformance test samples are obtained for the geocomposite.

The rolls to be sampled shall be selected by the SSQA Technician. Samples shall not be taken from any portion of a roll which has been damaged. Unless otherwise specified, samples shall be 3 ft (1 m) long by the roll width. The SSQA Technician shall mark the machine direction on the samples with an arrow. All lots of material and the particular test sample that represents each lot should be defined before the samples are taken.

A lot shall be defined as a group of consecutively numbered rolls from the same manufacturing line. Alternatively, a lot may be designated by the SSQA Technician based on a review of all roll information including quality control documentation and manufacturing records.

Unless otherwise specified, samples shall be taken at a rate of one per lot, not to be less than one per 100,000 ft^2 (10,000 m^2) of geocomposite. These samples shall then be forwarded to the SSQA Technician for testing to ensure conformance with the project specifications.

9.4.2 Conformance Tests

At a minimum, the following conformance tests shall be performed on the geocomposite as a unit:

1. Mass per unit area (ASTM D3776)
2. Geotextile-geonet adhesion (ASTM D413)

Additional conformance tests may be required by the project specifications.

9.4.3 Test Results

All conformance tests results shall be reviewed and accepted or rejected by the SSQA Technician prior to the deployment of the geocomposite. The SSQA Technician shall examine all results from laboratory conformance testing and shall report any nonconformance to the Project Manager. The SSQA Technician shall be responsible for checking that all test results meet or exceed the property values listed in the project specifications.

If the Manufacturer has reason to believe that failing tests may be the result of the SSQA Technician incorrectly conducting the tests, the Manufacturer may request that the sample in question be retested by the SSQA Technician with a technical representative of the Manufacturer present during the testing. Alternatively, the Manufacturer may have the sample retested at two different Owner-approved labs at the expense of the Manufacturer. If both laboratories produce passing results, the material shall be accepted. If both laboratories do not produce passing results, then the original SSQA Technician's test results shall be accepted. The use of these procedures for dealing with failed test results is subject to the approval of the Project Manager.

If a test result is in noncomformance, all material from the lot represented by the failing test should be considered out-of-specification and rejected. Alternatively, at the option of the Project Manager, additional conformance test samples may be taken to "bracket" the portion of the lot not meeting specification (note that this procedure is valid only when all rolls in the lot are consecutively produced and numbered from one manufacturing line). To isolate the out-of-specification material, additional samples must be taken from rolls that have roll numbers immediately adjacent to the roll that was sampled and failed. If both additional tests pass, the roll that represents the initial failed test and the roll manufactured immediately after that roll (next larger roll number) shall be rejected. If one or both of the additional tests fail, then the entire lot shall be rejected or the procedure repeated with two additional tests that bracket a greater number of rolls within the lot.

9.5 GEOCOMPOSITE DEPLOYMENT

During shipment and storage, the geocomposite shall be protected from ultraviolet light exposure, moisture, mud, dirt, dust, puncture, cutting, or any other damaging conditions. Geocomposite rolls shall be shipped and stored in relatively opaque and watertight wrappings. The roll wrappings shall be removed shortly before deployment.

For one-sided geocomposite, the SSQA Technician shall verify that the geonet is free of dirt and dust prior to installation. The SSQA Technician shall identify any dirty rolls and report them to the Project Manager. If the geonet is judged to be dirty or dusty by the SSQA Technician, it shall be cleaned by the Installer prior to installation. Washing operation shall be observed by the SSQA Technician and improper washing operations shall be reported to the Project Manager.

The SSQA Technician shall observe rolls upon delivery at the site and any deviation from the above requirements shall be reported to the Project Manager.

The Installer shall handle all geocomposite in such a manner as to ensure they are not damaged and the following shall be compiled with:

1. On slopes, the geocomposite shall be securely anchored and then rolled down the slope in such a manner as to continually keep the geocomposite sheet in tension. If necessary, the geocomposite shall be positioned by hand after being unrolled to minimize wrinkles.
2. In the presence of wind, all geocomposites shall be weighted with sandbags of the equivalent. Sandbags shall be installed during deployment and shall remain until replaced with cover material.
3. Unless otherwise specified, single-sided geocomposite shall not be welded to the geomembrane.
4. Geocomposites shall be cut using a hook blade or other tool approved by the Project Manager. If in place, special care shall be taken to protect underlying geosynthetics

from damage which could be caused by the cutting of the geocomposite. Care shall be taken not to leave the tools in the composite.

5. The Installer shall take any necessary precautions to prevent damage to underlying layers during placement of the geocomposite.
6. During placement of geocomposite, care shall be taken not to entrap in or beneath the geocomposite, stones, or dirt that could damage the geomembrane, cause clogging of drains or filters, or hamper subsequent seaming. If dirt or excess dust is entrapped in the geonet of single-sided geocomposite, it should be washed clean prior to placement of the next material on top of it. In this regard, care shall be taken with the handling of sandbags, to prevent puncturing the sandbag.
7. A visual examination of the geotextile component of the geocomposite shall be carried out over the entire surface, after installation to ensure that no potentially harmful foreign objects are present.

The SSQA Technician shall note any noncompliance and report it to the Project Manger.

9.6 SEAMING PROCEDURES

In general, no horizontal seams shall be allowed on sideslopes thus seams shall be along, not across, the slope, except as part of a patch. If horizontal seams are required, offset adjacent horizontal seams.

At a minimum, the following requirements shall be met:

1. Adjacent geocomposite shall be overlapped so that the geonet overlaps by at least 4 in (100 mm) and geotextile overlap by at least 3 in (75 mm).
2. If two sided geocomposite or the geotextile is on bottom, seam geotextile together.
3. The geonet overlaps shall be tied with plastic fasteners. Tying devices shall be white or yellow for easy inspection. Metallic devices are not allowed.
4. Tying shall be every 5 ft (1.5 m) along the slope, every 6 in (150 mm) in the anchor trench, and every 6 in (150 mm) along end-to-end seams on the base of the landfill.
5. In the corners of the sideslopes where overlaps between perpendicular strips are required, an extra layer shall be unrolled along the slope, on top of the previously installed geocomposite, from top to bottom of the slope.
6. When more than one layer of geocomposite is installed, joints shall be staggered.
7. Once geonet is tied, the geotextile of the geocomposite shall be seamed. On slopes steeper than 10:1 (horizontal:vertical), all geotextiles shall be continuously sewn. Spot sewing is not allowed. On bottoms and slopes shallower than 10:1, geotextiles shall be sewn (preferred), or thermally bonded with the written approval of the Project Manager. The Installer shall pay particular attention to seams to ensure that no earth cover material could be inadvertently inserted beneath the geotextile if applicable.

8. Any sewing shall be done using polymeric thread with chemical and ultraviolet light resistance properties equal to or exceeding those of the geotextile. Sewing shall be done using machinery and stitch types specified int he project specifications or as approved in writing by the Project Manager and the SSQA Technician.

The SSQA Technician shall note any noncompliance and report it to the Project Manager.

9.7 DEFECTS AND REPAIRS

9.7.1 Identification

If a defect is identified in the geotextile/geonet composite, the SSQA Technician shall determine the extent and nature of the defect. If the defect is indicated by unsatisfactory test result, the SSQA Technician shall determine the extent of the deficient area by additional tests, observations, a review of records and other means that the SSQA Technician deems appropriate.

9.7.2 Notification

After determining the extent and nature of the defect, the SSQA Technician shall promptly notify the Installer and Project Manager. A work deficiency meeting shall be held as required between the Installer, SSQA Technician, Project Manager and any other necessary parties to assess the problem, review alternative solutions, and implement an action plan.

9.7.3 Repair Procedures

The final decision as to the appropriate repair shall be agreed upon between the Project Manager, Installer, and SSQA Technician. Prior to acceptance of the geocomposite, the Installer shall locate and repair all damaged areas as directed by the SSQA Technician. Care shall be taken to remove any soil or other material which may have penetrated the torn geotextile. The SSQA Technician shall observe any repair and report any noncompliance with the following requirements in writing to the Project Manager.

If in the SSQA Technician's judgement, the defect is determined to be small, typically smaller than 3 by 3 feet (1 m by 1 m), the geocomposite shall be repaired as follows:

1. If the geonet is judged to be undamaged but the geotextile is damaged, a patch of geotextile shall be placed. The geotextile patch shall be thermally bonded in place with a minimum of 12 inch (0.3 m) overlap in all directions.
2. If the geonet is judged to be damaged, the damaged geonet shall be removed. A section of geonet shall be cut to replace the removed section. The geonet shall be tied to the existing geonet using white plastic fasteners placed at least every 6 inches (150 mm) overlap. A geotextile patch shall be placed over the repaired geonet

section. The geotextile patch shall be thermally bonded in place with a minimum of 12 inch (0.3 m) overlap in all directions.

If in the SSQA Technician's judgement, the defect is determined to be large, typically larger than 3 by 3 feet (1 m by 1 m), the geocomposite shall be replaced.

The SSQA Technician shall observe any repair and report any noncompliance with the above requirements in writing to the Project Manager.

9.7.4 Geocomposite Protection

For single-sided geocomposites, soils should never be placed in direct contact with geonet. All soil materials located on top of the geocomposite shall be deployed in such a manner as to ensure:

1. The geocomposite and underlying lining materials are not damaged.
2. Minimal slippage of the geocomposite on underlying layers occurs.
3. No excess tensile stresses occur in the geocomposite.

Any noncompliance with these guidelines or the project specifications shall be noted by the SSQA Technician and reported to the Project Manager.

10.0 LEACHATE COLLECTION SYSTEM

10.1 LEACHATE COLLECTION NETWORK

The leachate collection network is to be placed in the cell after the liner section has been installed and before the protective and drainage layer has been placed. The pipes will conform to all pertinent ASTM requirements and dimensions outlined in project plans and specifications. The pipes will be surrounded by non-carbonate stone which is to be wrapped in geotextile drainage fabric as shown in the project drawings. The SSQA Technician will observe the process of placement to ensure that reasonable precautions are taken so as to avoid any damage to the geomembrane liner during the placement of the drainage layer. Prior to covering the system with the protective and drainage layer, the pipe diameter, length and slope of each lateral will be confirmed by the SSQA Technician. Any and all discrepancies will be immediately brought to the attention of the SSQA Technician and landfill manager. Resolution of the discrepancy must be obtained and corrections documented prior to placing the protective cover.

10.2 DRAINAGE LAYER/LINER PROTECTIVE COVER CONSTRUCTION

The liner protective cover, which is intended to also serve as a protective buffer between the waste and the geosynthetic base liner components, as well as a drainage layer for the

leachate collection system, will be installed above the geomembrane liner and around the collection system after completion of the liner system. Placement technique will be submitted to and approved by CE or SSQA Technician prior to construction. In general, the protective cover will be placed in a single 2 foot lift to minimize possible liner damage.

10.3 QUALITY ASSURANCE TESTING

The SSQA Technician will collect representative samples of granular layer for every 3,000 yd^3 placed. Grain size distribution (through #200 sieve) test will be performed to confirm material compliance with project specifications. Testing to determine grain size distribution will be performed to allow certification of material compliance before the liner protective cover and drainage media is placed over the leachate collection system. Testing to determine permeability of granular layers will be made for every source of material. The CE or CQAPM will determine the appropriate sample density and confining pressure. Documentation of testing will be submitted with the certification as support data. Documentation will include comparison of grain size distribution with permeability test results.

11.0 DOCUMENTATION

11.1 DAILY FIELD REPORTS AND TEST RESULTS

The SSQA Technician will prepare daily reports which provide detailed descriptions of the construction operations for each day. The daily field reports will include but not be limited to the following:

a. Date, project name, location and other identifications and data on weather conditions.
b. Reports of any meeting(s) held and any pertinent results.
c. Description of construction operations, locations and equipment working on site during the time frame of the daily report.
d. Description of location of CQA testing including other testing operations, sampling locations, material used and quality verification documentation.
e. Decisions with regard to approval of units, material or work and/or corrective actions to be taken where substandard quality is identified.

The daily field reports should also include general comments and observations and will be signed by the SSQA Technician. A copy of the daily report will be kept on site and made available to the landfill manager, CQA personnel, Contractor, and ~~EPD~~ NCDEHNR personnel if so requested.

All field and laboratory test results will be reported on standard sheets and will, at a minimum, include the following:

a. Test title, sampling/test date, exact location of field test or sampling, personnel and any comments.
b. Type of testing activity and procedural references (standard methods when applicable).
c. Results of testing activity and comparison with specifications along with signature of performer and the SSQA Technician.

11.2 APPROVAL DOCUMENTATION

All corrective measures taken to bring unsuitable work up to design specifications must be documented. This documentation must describe what is in fault and the exact location, test designation that proves the work is unsuitable, the corrective measures agreed upon to bring it up to design specifications, the dates that corrective work was done, and the test designation that proves the work is now acceptable. All non-conformance reports will be brought to the attention of the Owner or Owner's representative by the SSQA Technician and will be documented in the CQA files. All work will be documented as to quality and attested to by the CE, including his signature of approval of "as-built" drawings.

The QA documentation will be organized and indexed to enable easy access and retrieval of original inspection and testing data sheets and all reports. During the construction period, originals of the documents will be maintained by the SSQA Technician and copies will be kept by the Owner's representative. At the completion of the project, a final certification report will be issued by the CE and transmitted to the ~~Georgia EPD~~ NCDEHNR and the Owner and Owner's representative. This document will include, at a minimum, the following information:

a. Scope of work, all daily field reports, laboratory and field test results.
b. Test methods and evaluation of test results with respect to project specifications.
c. Any non-conformance reports and corrective action reports.
d. Personnel involved with project and their qualifications.
e. As-built drawings and survey notes.
f. Certification of final construction as meeting or exceeding construction specifications. This certification should be signed and stamped by a registered professional geotechnical engineer (CE).

Once the construction quality has been certified by the CE and has been accepted by the Owner or Owner's representative, originals of the CQA documentation will be stored by the Owner and maintained.

To facilitate the review process by North Carolina DEHNR, the certification report may be issued as each phase of construction is completed. If this is done, it will be accomplished in such as way that when the last certification report is submitted North Carolina DEHNR will have a completed document, thus allowing sufficient time for review as well as insuring that the project is progressing in accordance with the permit.

TABLE 1

COMPACTED CLAY BASE LINER/FINAL COVER

		FREQUENCY		
Property	**Test Method**	**Conformance**	**Field**	**Acceptance Criteria**
Moisture Content	ASTM D2216, D3017	*5,000 cy	1 per 10,000 sf/lift	Acceptable Zone
Particle Size	ASTM D1140, D422	*3,000 cy	NA	
Atterburg Limits	ASTM D4318	5,000 cy	NA	LL ≥30 PI ≥15
Lab compaction	ASTM D1557 modified D698 standard	5,000 cy	NA	
Lab Hydraulic Conductivity	ASTM D5084	5,000 cy	1 per acre/lift	K ≤1x10^{-7} CM/SEC Clay Base K ≤1x10^{-5} CM/SEC Final Cover
Density	ASTM D1556, D2922, D2167	NA	1 per 10,000 SF/lift	Acceptable Zone
In-Situ Shear	ASTM D4767	1 per source	Pocket pentrometer 1 per 10,000 SF/lift	

*Source quality control one per 20,000 cy.

TABLE 2

PROTECTIVE COVER MATERIAL

Property	Test Method	Minimum Frequency	Acceptance Criteria
Particle Size	ASTM D422, D1140	1 per 3,000 cy	Within the Specified Gradation
Permeability	ASTM D2434	1 per material change	Curve K $\geq 1 \times 10^{-4}$ CM/SEC

TABLE 3A

POLYETHYLENE GEOMEMBRANE (Smooth)

PROPERTY	METHOD	VALUE	
Thickness	ASTM D751*	60 mils	min. average
Thickness	ASTM D751*	~~60~~ 54 mils	min. reading
Density (geomembrane)	ASTM D1505	0.940 g/cc	min.
Melt Index (resin)	ASTM D1238	1.0 g/10	min. max.
Tensile Properties: (each direction) 1. Yield strength 2. Break strength 3. Elongation at yield 4. Elongation at break	ASTM D638*	 2200 psi 3800 psi 12% ~~600~~ 700%	 min. min. min. min.
Tear Strength	ASTM D1004	~~700~~ 750 lb/in	min.
Puncture Resistance	ASTM D4833	1800 lb/in	min.
Low Temperature	ASTM D746	-60°C	max.
Carbon Black Content	ASTM D1603	2.0 to 3.0%	
Carbon Black Dispersion	ASTM D3015 NSF Modified	A-1 or A-2	
Dimensional Stability (each direction)	ASTM D1204*	2.0%	max. change
Environmental Stress Crack	GRI GM-5b	200 hours	min.
Multi-axial Elongation	GRI GM-4	20%	min.

*Test Methods as modified in Table ~~7~~ 5.

TABLE 3B

60 MIL HDPE GEOMEMBRANE SEAMS

PROPERTY	METHOD	VALUE	
Bonded Seam Strength	ASTM D4437	132 lb/in	min.
Peel Adhesion: Fusion Extrusion	 ASTM D4437 ASTM D4437	 90 lb/in 78 lb/in	 min. min.

TABLE 3C

POLYETHYLENE GEOMEMBRANE (Textured)

PROPERTY	METHOD	VALUE	
Thickness	ASTM D751*	60 mils	min. average
Thickness	ASTM D751*	54 mils	min. reading
Density (geomembrane)	ASTM D1505	0.940 g/cc	min.
Melt Index (resin)	ASTM D1238	1.0 g/10	min. max.
Tensile Properties: (each direction) 1. Yield strength 2. Break strength 3. Elongation at yield 4. Elongation at break	ASTM D638*	 2200 psi 2200 psi 12% 200%	 min. min. min. min.
Tear Strength	ASTM D1004	750 lb/in	min.
Puncture Resistance	ASTM D4833	1800 lb/in	min.
Low Temperature	ASTM D746	-60℃	max.
Carbon Black Content	ASTM D1603	2.0 to 3.0%	
Carbon Black Dispersion	ASTM D3015 NSF Modified	A-1 or A-2	
Dimensional Stability (each direction)	ASTM D1204*	2.0%	max. change
Environmental Stress Crack	GRI GM-5b	200 hours	min.
Multi-axial Elongation	GRI GM-4	15%	min.
Index Friction	GRI GS-7	40°	Min Avg.

*Test Methods as modified in Table 5.

WASTE MANAGEMENT OF CAROLINAS, INC.

OPERATIONS PLAN

FOR

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, NORTH CAROLINA

Prepared By:
RUST ENVIRONMENT & INFRASTRUCTURE
GREENVILLE, SOUTH CAROLINA
NAPERVILLE, ILLINOIS

DECEMBER 1993

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1.0 INTRODUCTION

1.1 PURPOSE

The Piedmont Landfill and Recycling Center (PLRC) is presenting this Operating Plan as part of a Transition Application for the existing portion of the landfill. It is also submitted as part of a One-Time Permit Renewal for the currently permitted but not yet constructed portion of the landfill. This operations manual should be read in conjunction with the following documents:

- North Carolina Solid Waste Management Regulations
- Design drawings entitled "Site Development Plan for Piedmont Landfill and Recycling Center, Forsyth County, North Carolina," prepared for Waste Management of Carolinas, Inc., by Rust Environment & Infrastructure, Solid Waste Division, November 1993 (Vol. 3 of this submittal).

The objective of this Operating Plan is to describe the daily activities involved in the operation and development of the entire PLRC. This plan will also describe how the facility will comply with various operational requirements pursuant to Rule .1625 and .1626 of the Solid Waste Management Regulations.

Information provided in this plan from the aforementioned supporting documentation is incorporated by reference to eliminate any variations or conflicts. This plan supersedes all previously permitted operating plans for PLRC.

1.2 SITE DESCRIPTION

The PLRC is located on approximately 108 acres of property in Forsyth County, North Carolina, approximately 10 miles north of the city of Kernersville. The landfill occupies an approximate foot print of 68 acres. Waste Management of Carolinas, Inc., a subsidiary of Waste Management, Inc. - South, owns and operates the facility. Currently, approximately 30 acres of the landfill have been developed for landfilling operations. A composite liner system on the landfill base consisting of 18 inches of compacted cohesive soil with a maximum hydraulic conductivity of 1×10^{-7} cm/sec overlain by a 60 mil HDPE geomembrane exists in Phase I, Modules 1 and 2. A double synthetic with primary and secondary leachate collection systems is incorporated in existing Phase I, Modules 3, 5 and 6 and Phase II, Modules 1 and 2. The future fill areas will incorporate a composite liner system which meets the requirements of the Solid Waste Management Regulations. The location and layout of the site is detailed on the design drawings.

2.0 GENERAL FACILITY INFORMATION

2.1 PERSONNEL

The daily operations at Piedmont Landfill and Recycling Center are performed under the direction of the Landfill Manager. The Landfill Manager is responsible for all operations at the site.

In addition to the Landfill Manager, site personnel, including equipment operators, mechanics, laborers, and others will assist in performing daily operations. The numbers of each type of employee may vary. However, sufficient staff will be maintained to operate the landfill in a safe and efficient manner. These personnel will also be trained in safe and proper landfill operation procedures.

2.2 EQUIPMENT

The following equipment is currently maintained on-site and is utilized as necessary for landfill operations:

- Waste compactor(s)
- Bulldozer(s)
- Scraper(s)
- Backhoe(s)
- Dump Truck(s)
- Water Truck
- Grader

Equipment substitutions, deletions and/or additions may occur, but the site will maintain sufficient numbers of equipment which will perform comparably to the equipment listed above.

2.3 OPERATING SCHEDULE

The normal operating hours at the PLRC are from 7 a.m. to 5 p.m. on Monday through Friday, and Saturdays from 7 a.m. to 1 p.m.. The hours of operation may be adjusted if conditions warrant. A sign indicating the hours of normal operation and the permit number is posted at the entrance of the landfill. Site operations may extend beyond the gate hours to allow for site preparation, construction activities, maintenance, and daily covering. In addition, access to the site may exceed posted hours on special occasions and during emergencies.

2.4 ON-SITE STRUCTURES AND UTILITIES

The PLRC maintains on-site structures necessary for daily operations. These include an administrative building/scale house, and maintenance garage.

Existing buildings on the landfill site may be demolished or relocated as necessary as landfill operations progress across the site property. These buildings will be replaced as needed to ensure proper operations. Existing on-site drinking water wells will be abandoned per North Carolina regulations.

2.5 HAUL ROADS

Haul roads for landfill construction vehicles will be constructed as needed to provide access to serve fill and borrow areas. The access roads constructed will be of all-weather construction and maintained to allow year-round use. Traffic will be directed by landfill personnel, signs, barricades, and/or pylons as needed.

2.6 SIGNS POSTED

Signs are posted at the site entrance indicating the name of the facility, name of the operating entity, state permit number, hours and days of operation. In addition, a sign which clearly states "NO HAZARDOUS WASTES ACCEPTED" is located at the entrance to the landfill. Traffic control and safety requirement signs are located at and near the entrance to the facility as needed.

2.7 LANDFILL CAPACITY (Rule .1625 (b)(3)(D) and (E)

~~The remaining waste capacity of the PLRC as of October 9, 1993 has been calculated to be 1,140,000 cubic yards.~~ The remaining waste capacity of the PLRC as of January 31, 1994 has been calculated to be 5,240,023 cubic yards. This value represents a net waste volume (i.e., excluding daily, intermediate and final cover soils).

Based on the aforementioned net waste volume, the remaining site life of the PLRC has been calculated to be approximately ~~4.4~~ 4.5 years. This life was calculated assuming a constant waste receipt rate of ~~2200~~ 2,600 tons/day, an in-place density of 1200 lbs/cu yd, and days of operation as mentioned in Section 2.3 of this Operations Plan.

None of the existing operating units have been closed. The anticipated completion of filling existing units is approximately August, 1994.

3.0 OPERATION DRAWINGS

3.1 EXISTING CONDITIONS (Rule .1625 (b)(1)(A))

All existing conditions, including the known limits of existing disposal areas, are illustrated on the design drawings. This drawing provides an aerial topographic survey, as performed July 1, 1993 by Air Survey, Inc., of Virginia.

3.2 LANDFILL DEVELOPMENT SEQUENCE (Rule .1625 (b)(1)(B))

Landfill development will involve construction of areas of the landfill on a cell-by-cell basis, as current fill areas begin approaching fill limits. Each cell will be constructed individually to planned base grades. The designed liner and leachate collection system will be installed, and temporary and permanent berms will be placed around each cell to prevent water run-off from entering fill areas. The locations and details of the phase separation berms, as well as liner and leachate collection systems, are shown on the design drawings. Deviations may be made to the phasing plans as necessary.

The landfill is divided into three phases, with each phase divided into cells and subcells. The orientation and designation of landfill cells and phases are presented on the Base Grade Plan, in the design drawings. The first cell constructed was Phase I-Module I, located at the southwest corner of the site. Construction of Phase 3-Cell I, Subcells 1 and 2, is anticipated to occur in 1994.

3.3 PROGRESSION OF LANDFILL OPERATIONS (Rule .1625 (b)(1)(C))

Landfilling activities will proceed on a cell-by-cell basis as discussed in the previous section. As landfilling activities proceed within one cell, the next cell to be filled will be prepared to accept waste.

~~To reduce the potential for damage to the liner and leachate collection system, only moderate compaction will be applied to the first lift of waste. The initial lift of waste will be approximately five feet thick and will be placed across the entire newly constructed cell. Following this initial lift, normal operations will proceed.~~

~~Compacted refuse will be placed in lifts approximately 15 to 20 feet thick but may depend upon conditions.~~

~~Landfill compaction equipment will make an appropriate number of passes over each layer to provide an adequate compaction and reduce the long term settlement of the landfill. Large or bulky objects may be separated from refuse at the working face and thoroughly crushed by compaction equipment to prevent bridging and localized subsidence at a later time.~~

~~For a detailed illustration of the final contours for the PLRC, see the design drawings.~~

Waste placement will occur in general accordance with Section 4.1.6Waste Placement. As the landfill progresses towards unused, constructed subcells cut off by stormwater flaps, the flaps will be properly abandoned prior to the placement of waste in that new subcell. See Detail 4 on Drawing No. 11 for the specific abandonment procedure. Once the stormwater flap has been properly abandoned, any stormwater entering the area will be handled as leachate.

3.4 LEACHATE CONTROL (Rule .1625 (b)(1)(D))

To collect and store any leachate that infiltrates through the landfill, a leachate collection system has been designed for the PLRC. This system is shown in detail in the Leachate Collection System Plan and Detail sheets of the site design plans. The system includes a geonet drainage layer overlain with a drainage/protective cover layer. Included is a network of perforated 6 inch diameter Schedule 80 PVC or SDR 17 HDPE plastic pipes in stone-filled trenches within the base drainage layer of the landfill. These pipes drain at a minimum of one percent grade towards one of five leachate collection sumps.

Additional leachate collection pipes are located around the perimeter of the landfill base. The base of the landfill is designed to slope at a minimum of 2% post-settlement grade.

The leachate collection sumps are designed to collect leachate and allow for removal of the leachate by pumping when a pre-determined level of leachate is reached. Each sump consists of a minimum of 2'-6"feet deep trough with a 10 ft X 12 ft base and 3 horizontal:1 vertical side slopes (see collection pump detail in the design plans). The sump is lined with the composite system as well as additional geomembrane and geotextile for added protection. The sump's purpose is to provide enough liquid to operate pumps efficiently.

Leachate is pumped from the collection sumps via a perimeter force main system to a leachate storage area. Leachate is then removed from the storage facilities and disposed of at an approved disposal facility (e.g., wwtp, potw, etc.). Refer to the design drawings for further detail.

3.5 STORMWATER CONTROL (Rule .1625 (b)(1)(D) and .1625 (b)(3)(B))

A sedimentation and erosion control plan has been developed and is provided on the design drawings. The plan incorporates a series of landfill benches or terraces, perimeter channels, and three sedimentation ponds. The landfill benches are spaced at approximate 40 foot vertical intervals to collect and direct surface water run-off from the final cover to the perimeter channels. The perimeter channels then convey the run-off to the sedimentation ponds. The run-off will be detained in one of the three sedimentation ponds to allow adequate settling of the sediment particles, prior to discharge off-site. These structures were designed to manage run-off from a minimum 25-year, 24-year storm event. In addition, temporary erosion control devices will be installed as needed to minimize sediment discharge off-site from unvegetated areas of the facility.

Refer to Section 4.8 of this Operations Plan for further information on the surface water management system for the facility.

3.6 WASTE ACCEPTANCE (Rule .1625 (b)(1)(E))

There will be no special waste areas within the landfill. Special wastes will be commingled with other wastes. See Appendix A at the end of this section for a detailed description of the special waste acceptance program.

3.7 BUFFER ZONES (Rule .1625 (b)(1)(F))

All buffer zones and restricted use areas at the PLRC are indicated on the design drawings.

3.8 STOCKPILE AND BORROW OPERATIONS (Rule .1625 (b)(1)(G))

Stockpile areas at the PLRC are indicated on Sheet 21 of the design drawings. Soil for use as cover and liner will be obtained from excavating the Phase 3 area. Liner may be admixed with bentonite or obtained from off-site sources. See Appendix B of the Engineering Plan for soil balance calculations.

3.9 PHASING PLANS (Rule .1625 (b)(3)(C)

Section 3.2 of this Operations Plan describes the sequence of the development and phasing at the PLRC. The facility anticipates achieving final elevations during the latter stages of development, and therefore the Final Development Plan presented in the design drawings represents conditions at the end of the phase. No additional phasing plans have been prepared. Subcell transition contours have been developed for each construction sequence. Cell 1, Subcells 1 and 2 will be developed initially, followed by Cell 2, Subcells 1 and 2. Transition contours for these subcell developments have been included as Drawings 4A and 4B of the design drawings. Cell 1 - Subcell 3 is the final area to be developed, and its transition contours are displayed on the Final Development Plan.

4.0 OPERATIONAL REQUIREMENTS

4.1 WASTE ACCEPTANCE AND DISPOSAL REQUIREMENTS (Rule .1626 (1) (a)

The PLRC is permitted as a non-hazardous municipal solid waste disposal facility and will only accept those solid wastes which it has been permitted to receive. The types of waste accepted at the facility include non-hazardous residential, industrial, and commercial wastes, demolition and construction debris, asbestos, waste water sludge and various other miscellaneous waste types.

~~The Division has developed a "Procedure and Criteria for Waste Determination". Presently this procedure requires the generator of the special waste to seek Division approval for each individual waste stream. this application requests that the Division provide for an addendum to the above referenced "Procedure" specifically for the PLRC. Essentially, this addendum would allow the piedmont LRC to utilize its own Special Waste approval process which is identified below. Please understand that the PLRC WOULD NOT APPROVE OF ANY SPECIAL WASTE STREAM THAT IT IS NOT PERMITTED TO RECEIVE. PLRC's Special Waste Guidance document is included in Appendix A at the end of this section.~~

4.1.1 Hazardous and Liquid Waste Disposal(Rule .1626(1)(b) and (f))

The PLRC will not accept any of the following wastes:

1. Hazardous waste as defined within 15A NCAC 13A, also including hazardous waste from conditionally exempt small quantity generators.

2. Polychlorinated Biphenyls (PCB) wastes as defined in 40 CFR 761.

3. Liquid wastes unless they are managed in accordance with Rule .1626 (9) of the North Carolina Solid Waste Regulations. Refer to Section 4.10 of this Operations Plan for further information on liquid waste disposal restrictions.

The PLRC currently incorporates a program for detecting and preventing the disposal of hazardous and liquid wastes at the facility. The first line of defense to reduce the risk of accepting unauthorized waste is the PLRC's special waste approval program. This program utilizes a system by which the generator of a special waste stream provides the appropriate information related to a particular waste stream (e.g. process generating waste, analytical data, amount, etc.). The information is then approved/disapproved by qualified in-house personnel. This decision process is appropriately documented in writing, with all files kept at the site. For further information on this program, see Appendix A.

If the special waste stream is approved, the paperwork is placed in the active, on-site files. All incoming special waste is then manifested by the generator and, while at the gate house, the manifest is cross checked against the approved waste stream. Should the cross check prove that the manifested load is approved, it is then sent to the working face for disposal, provided there is no need for verifying some of the parameters on the approved analytical data such as testing for free liquids (paint filter test). The PLRC has constructed a sampling tower should this need arise to verify the load. Should the manifested load not cross check with an approved special waste file, the truck is pulled to the side and a further investigation will begin. Typically, on the same day of attempted disposal the PLRC will be able to resolve the issue. If the load is not authorized for disposal, the load is sent back to the generator and the DSWM notified within 24 hrs of any attempted unauthorized disposal.

Another program that has been instituted at the PLRC is the random waste inspection program. This program consists of ~~includes~~ random inspections of incoming waste loads with corresponding documentation of each event. This inspection occurs near the working face whereby the load is dumped and inspected for unauthorized waste. All unauthorized waste is returned to the generator .

Facility personnel are trained to recognize hazardous and liquid wastes. Contingency plans also have been developed to properly manage any identified hazardous and liquid wastes which might be encountered during a random waste load inspection. The plan addresses identification, removal, storage, and final disposal of the waste.

4.1.2 Asbestos Waste Disposal (Rule .1626(1)(d))

Asbestos waste shall be managed in accordance with 40 CFR 61, including all subsequent additions. ~~The asbestos will be covered immediately upon disposal. In addition, asbestos waste will be disposed of separate from other solid wastes, at the bottom of the working face or in areas not contiguous with other disposal areas. Asbestos disposal areas will be clearly designated on a map for future reference.~~ Specifically, asbestos management will be as follow:

- Will place friable asbestos waste in a disposal area separate from the active working face. Asbestos disposal will not require a separate cell. The asbestos will not be compacted. It will then be covered immediately with additional soil or refuse. A receipt log containing the following information will be maintained: generator name and address, date, volume, transporter name and address and disposal location coordinates.

- A topographic map or alternative system which can reasonably identify the disposal location of friable asbestos waste will be used. Disposal locations will be recorded on a log and this topographic map.

- All asbestos waste will have Waste Shipment Record (WSR).

- Will reconcile quantity discrepancies as well as any improperly enclosed or contained waste with generator within 15 days of receipt; if cannot reconcile, will report discrepancy and reconciliation attempts to NCDEHNR in writing with copy of WSR.

- Will list on WSR any rejected materials and indicate the ultimate destination of those materials.

- Will send completed copy of WSR to generator within 30 days of waste receipt

- Will notify NCDEHNR in writing at least 45 days prior to excavating or disturbing waste deposited and covered. If excavation begins on date other than one in original notice, new start date will be provided to NCDEHNR at least 10 working days before excavation begins.

  Notice will include:

  - Scheduled start and completion dates
  - Procedures to control emission during excavation, storage, transport and ultimate disposal
  - Location of any temporary storage site and final disposal site

- Will retain WSRs for at least 2 years

- Upon site closure, submit to Federal or State Agency Administrator copy of records of disposal location and quantities

- Within 60 days of site closure, a notification on the deed to facility property to notify any potential purchaser that the land had been used for disposal of asbestos containing waste. A survey plot and locations will be filed with the NCDEHNR.

4.1.3 Wastewater Treatment Sludge Disposal (Rule .1626(1)(e))

Because the PLRC meets the design requirements of Rule .1624, wastewater treatment sludge may be co-disposed at the facility. In addition, the sludge may be used as a soil conditioner and incorporated into the vegetative growth layer in a thickness not to exceed six inches.

4.1.4 Miscellaneous Waste Disposal (Rule .1626 (1)(c))

Spoiled foods, animal carcasses, abattoir waste, hatchery waste, and other animal wastes accepted by the facility for disposal will be covered immediately by cover soils or additional waste.

4.1.5 Special Waste Disposal (see Appendix A)

4.1.6 Waste Placement (Rule .1626 (1)(g))

Waste disposal at the PLRC will be performed in a manner which promotes efficient compaction and waste stability. Waste placement will be maintained within the permitted vertical and horizontal waste limits. Refer to the facility Base Grade and Final Development Plans for information concerning these waste limits. The existing landfill area has been and the future development areas will be constructed with a base liner.

The landfill will be constructed using the area method. Normally, compacted refuse will be placed in lifts approximately 15 feet in thickness. The exception will be when initially covering a newly constructed cell. This initial placement of this waste will happen in the following manner:

> Special care and precaution will be taken when placing and compacting the first lift of refuse over the leachate collection/protective layer and liner. The first lift over newly lined areas will consist of only select solid waste devoid of materials that could damage or penetrate the leachate collection system and/or liner. To further reduce the potential for damage to the liner/collection system, only moderate compaction (approximately two passes of the compactor) will be applied to the first lift. Operating equipment will be restricted to travel on the waste layer only.

During normal landfilling operations, refuse will be spread and compacted in thin layers at the working face and compacted on a suitable slope with a landfill compactor. Landfill compaction equipment will make approximately three to five passes over each layer. This will provide adequate compaction and reduce the long-term settlement of the landfill. The advancing layer of compacted refuse will be covered at the end of each working day. The working face will be kept to a size which is easily compacted and covered daily. Under normal operating conditions, it is expected that the working face will not be any larger than 300 feet by 300 feet.

It is expected that no special provisions will be necessary for wet weather operations at the PLRC. The on-site entrance is designed as a paved all-weather road. Haul roads leading to the active working face will be constructed of crushed stone on an as-needed basis. In all cases, the roads will be maintained for all-weather use. Drainage controls will be maintained so that disposal operations can continue during periods of wet weather. These measures include use of temporary berms to drain direct precipitation and run-off within the active area away from the working face.

As waste placement reaches the crest of the cell/subcell slopes, placement will cease when the waste reaches approximately 12 feet horizontally from the inside crest to allow for the placement of intermediate and final cover (see Detail No. 4 on Drawing No. 12 specifying the transition contours for waste placement). Waste and intermediate cover will then be placed at a 4:1 slope, as per the final development plan. Also, as waste placement approaches unconstructed, future cells/subcells and/or stormwater flap berms, placement will cease at the base of the berm to allow for the placement of intermediate cover (see Detail Nos. 3, 4, and 6 and Drawing No. 11 for location of these berms). This intermediate cover will be installed so that stormwater can be shed away from the constructed modules and be pumped off as stormwater. It is expected that interim slopes, not subject to final cover grade requirements, will be no steeper than 2:1.

4.2 COVER MATERIAL PROVISIONS (Rule .1626 (2))

When additional refuse will not be placed over a landfill surface within 12 months, the refuse surface shall be covered with a minimum of 12 inches of intermediate soil cover. ~~As new daily cover alternatives become available, such as portable synthetic materials, the PLRC plans to utilize these methods on a trial basis, and requests generic approval from the Division to use them on a long-term basis. Landfilling will proceed in this manner until final elevations, as shown on the Final Development Plan (Sheet 5), are achieved.~~ Daily cover will consist of soil, and an alternate cover material. The purpose of this daily cover is to control various nuisances. The alternative daily cover expected to be used will be similar to the type presently approved by the DSWM, e.g., coated tarps sewn together and draped over the waste at the end of each day. See Appendix C for an explanation of how these tarps are placed in operation.

The final cover at the PLRC consists of the following components in ascending order: 18 inches of compacted cohesive soil with a maximum hydraulic conductivity of 1×10^{-5} cm/sec, a 40 mil coextruded VLDPE and HDPE geomembrane, an HDPE geonet/geotextile

composite, and two feet of protective soil. The top six inches of protective soil will be capable of supporting vegetation. This final cover meets the requirements of Rule .1627. Final cover will be constructed as areas of the landfill achieve final permitted elevations.

4.3 DISEASE VECTOR CONTROL (Rule .1626 (3))

Proper application of cover at the PLRC will be performed to minimize the presence of potential disease vectors, such as rodents, birds, and insects by limiting their access to buried wastes.

4.4 NUISANCE CONTROL

4.4.1 Odor Control

The potential for odors at the PLRC would be the direct result of landfill gas generated from the decomposition of the disposed waste. Odors will be minimized through the application of daily, intermediate and final cover of the waste. In addition, as landfill development proceeds and landfill gas generation increases, the Landfill Gas Management System presented on the design drawings will be gradually incorporated. This system represents a proactive approach to minimizing potential gas migration and odor problems.

4.4.2 Dust Control

Dust generation at the PLRC can result from operations such as cover material stripping and placement, refuse compaction and vehicular travel on access roads. Dust raised from operations will be controlled by wetting source areas with water or other dust control measures.

4.4.3 Noise Control

Noise from facility equipment will be minimized by use of appropriate mufflers. The equipment will be maintained in good condition to ensure its proper operation. In addition, the landfill buffers should provide an adequate distance between property boundaries and residences such that noise levels off-site are minimized.

4.4.4 Litter Control

The PLRC employs various practices to minimize litter away from the landfill working face. Litter control at the working face is performed by minimizing the size of the working face. Temporary fencing can be strategically placed at the working face to prevent winds from blowing the waste away. Given potentially difficult wind conditions, the working face can be oriented accordingly to avoid problems.

Facility personnel are directed to collect litter which may have escaped the working face. In addition, Freeman Road near the landfill is inspected to ensure litter which may have collected along the roadside is removed and properly disposed.

4.5 EXPLOSIVE GASES CONTROL (Rule .1626 (4))

This plan's purpose is to provide guidance to ensure compliance with applicable regulatory and safety guidelines regarding the control of landfill gas. This plan establishes a program to monitor for the presence of landfill gas along site property lines, adjacent to any off-site structures, and to monitor the potential for gas accumulation within on-site structures, at the Piedmont Landfill and Recycling Center. If monitoring indicates the possibility of off-site migration, additional monitoring will be performed to determine the extent of migration beyond the property line and also the potential for accumulation within off-site structures (i.e.,buildings, sub-grade utilities). A final objective is to ensure that the site remains within applicable regulatory and safety guidelines regarding control of landfill gas migration.

This plan provides site specific information on the gas monitoring system, including locations of monitoring points, methods and procedures of monitoring, equipment, monitoring frequency and report distribution.

4.5.1 Monitoring Frequency and Location

Monitoring for combustible gas (methane) concentrations will be performed by PLRC on a quarterly basis throughout waste disposal operations and the 30-year post-closure care period. Personnel designated to perform the monitoring will be trained in the operation, maintenance and calibration of the monitoring equipment. Barhole probe, structure, and permanent probe locations will be used for explosive gas migration monitoring. Monitoring may be modified as determined by the approved plan, regulations, or as monitoring indicates.

Structure monitoring will be performed as appropriate in on-site structures to monitor for any potential gas accumulation. Structures will be monitored to ensure that the concentration of methane does not exceed 25% of the lower explosive limit of methane (or 1.25%). This monitoring will be performed in confined areas of the structure (drains, toilets, vents, corners) where air movement may be restricted.

Barhole probing will be performed along the boundary of the landfill on a quarterly basis. Barhole probe locations will be modified and expanded to include remaining phases as filling progresses. Additional barhole probing may be performed to evaluate the concentration of methane in areas of stressed vegetation, or assess the potential for gas migration.

~~There are currently five (5) permanent gas monitoring probes installed along the boundary of the waste disposal area. These gas monitoring probes will be monitored to ensure that the concentration of methane does not exceed the lower explosive limit of methane (or 5%). Additional permanent gas monitoring probes will be established and installed along the perimeter of the landfill as filling progresses.~~

There are currently five (5) permanent gas monitoring probes installed along the boundary of the current waste disposal area. These probes are designated as GP-1 through GP-5. At a minimum, an additional six (6) permanent gas monitoring probes are proposed along the boundary of the future waste disposal areas within Phase 3, Cells 1 and 2. These proposed probes will be designated GP-6 through GP-11. The gas monitoring probes will be monitored to ensure that the concentration of methane does not exceed the lower explosive limit of methane (or 5%) in air. Refer to the Gas Management System Plan on Drawing No. 17 of 21 of the design drawings for the locations of the existing and proposed permanent gas monitoring probes.

Gas monitoring probes GP-6 through GP-11 are spaced at minimum intervals of 750 feet around the future waste boundary. Probe spacings of 750 feet to 1,000 feet are commonly incorporated at other landfill facilities, and have provided for effective gas migration monitoring. In addition, various features of the PLRC will aid in minimizing the potential of gas migration, thereby ensuring that the proposed spacing is adequate. These features include the presence of a continuous landfill base liner and final cover, with geosynthetic components, to contain gases within the landfill. These features also include the proposed incorporation of an active landfill gas extraction/management system. The proposed system represents a proactive approach to gas migration control.

At a minimum, the depths of the probes will extend to the lowest elevation to which waste has been deposited. The specific geology and hydrogeology beneath the facility will be considered when selecting the slotting of the permanent gas monitoring probes. In addition, the location of structures within and outside of the facility property boundary will be considered.

It should be noted that the exact locations of permanent gas monitoring probes may be modified, and the number of probes may be increased, from what is proposed in the design drawings as determined necessary at the time of installation.

4.5.2 Data Evaluation and Reporting

Should the methane gas concentration be determined to equal or exceed 5% by volume at any gas monitoring probe location, or 1.25% by volume within a structure, an evaluation will be made to determine whether the frequency of monitoring should be increased and locations expanded.

All methane gas monitoring measurements and observations shall be tabulated and a report generated.

4.5.3 Methane Gas Monitoring Procedures

Methane gas monitoring will be performed with a dual range, natural gas indicator that measures concentration as percent by volume of methane equivalent. Two ranges of detection should be available; 0 to 5%, and 0 to 100% methane by volume. A device such as the Gas-Tech Model NP-204 will be used. The combustible gas indicator will be

equipped with a flexible extension hose and rigid metal or fiberglass 30-inch long probe. At a minimum, the combustible gas indicator will be calibrated on a monthly basis in accordance with the manufacturer's specifications. The amount of monitoring and the handling of the meter will dictate if the calibration frequency should be increased.

A barhole probe monitoring procedure will be used at the designated gas monitoring probe locations. When barhole monitoring yields a high concentration of combustible gas, additional barhole probing is required. Barholes are punched and sampled at "as directed" intervals, radiating out in various directions from the original barhole of concern, until readings of zero are obtained. By employing a radial or grid-type pattern, a graph depicting the apparent movement of gas based on concentration can be plotted.

Barhole probing will be done at varying depths from the ground surface to a depth of about three feet. Shallower barhole probing on the order of 6 inches to 12 inches would be done when investigating for cover soil gas concentrations in vegetation stressed areas.

It should be noted that there are a number of seasonal variables which may affect the overall effectiveness of the barhole probe method. The moisture content of the ground may greatly affect the concentration of gas detected because combustible gas migrates readily through dry soil materials, as opposed to wet or saturated materials. Additionally, water saturated conditions may prohibit the use of the barhole probe method. Under these conditions, water infiltrates the barhole and can be sucked into the instrument during sampling.

The monitoring of structures can present a sensitive situation due to the possibility/proximity of human inhabitants. Combustible gas accumulation should be monitored within confined areas of the structure such as corners, along baseboards, crawl spaces, attics, underground facilities, drainage structures (drains, toilets, sumps, etc.) or any area where air movement is restricted.

4.5.4 Structure and Confined Space Monitoring

In addition to routine sampling, buildings may be monitored by permanently installed gas detectors. As continuous monitoring devices, these detectors have a built-in alarm system which sounds if gas concentrations exceed a predetermined value.

4.6 AIR AND FIRE PROTECTION CRITERIA (Rule .1626 (5))

The PLRC will be in compliance with any applicable requirements developed under a state implementation plan (SIP) approved or promulgated by the U.S. EPA administration pursuant to Section 110 of the Clean Air Act, as amended.

No open burning of solid waste, except for the infrequent burning of land clearing debris generated on-site or debris from emergency clean-up operations, will be permitted at the PLRC.

Fire protection of landfill equipment and vehicles will be provided by portable fire extinguishers. The gatehouse and maintenance facility will be equipped with suitable fire extinguishers for suppressions of any minor fires and for safety of personnel. Any minor fires that occur on the landfill will be extinguished by landfill personnel using soil to smother the fire or other appropriate methods. Additional fire protection coverage of this facility will be contracted with the local fire district.

The chance of fire occurring at a properly run landfill is minimal. However, the site will take several fire protection measures. "Hot loads" arriving on site will be directed to a designated area away from the active face and will be completely extinguished before being placed in an active cell. Soil stockpiles will be kept in the vicinity of the working face for use in extinguishing fires that may ignite during filling. In addition, site personnel are to be routinely instructed in firefighting procedures.

Should a fire occur at the landfill, additional compacted soil cover will be applied to cut off the flow of oxygen into the burning area. If this is not adequate to contain and extinguish the fire, the affected area is to be wetted thoroughly, excavated, and wetted down again before the affected cells are reconstructed.

The Division will be verbally notified within 24 hours in the event that a fire has occurred at the PLRC. Written notification will be submitted to the Division within 15 days.

4.7 ACCESS SAFETY PROVISIONS (Rule .1626 (6))

4.7.1 Access Provisions

The PLRC is accessible via Freeman Road. A sign is posted at the site entrance providing information on name, hours of operation, state permit number, and other pertinent information. A sign is also posted stating that regulated hazardous and liquid wastes are prohibited from being disposed in the landfill. The site entrance road is paved to provide year-round, all-weather access. In addition, on-site access roads to the landfill disposal areas are constructed of all-weather materials to facilitate year-round use. The access roads are maintained to ensure that operations are not impeded.

During normal operating hours, an attendant is on duty at the scales/office to monitor access. Access to the entrance of the site is restricted outside of normal operating hours by locked gates. A chain link fence is located around the perimeter of the facility property to serve as an additional access restriction. In addition, no trespassing signs are along the facility perimeter.

4.7.2 Safety Provisions

Traffic in and out of the PLRC will be directed through the use of traffic signs and markers. These signs will provide such information as direction to the disposal area, and other pertinent information for ensuring safe traffic patterns.

Scavenging from the landfill working face is not permitted. Disposal of barrels and drums will be prohibited unless they are RCRA empty and perforated sufficiently to ensure no liquid or hazardous waste is contained. Fiber drums containing asbestos are permitted for disposal.

The PLRC has developed an extensive program of safety and accident prevention at its waste disposal facilities. As part of this program, employees are routinely trained in proper operating and emergency procedures. In addition, telephone, radio communication, and first aid equipment are provided at the facility. See Appendix D for the agenda presently taught at the routine safety meetings. This agenda will change as the need arises.

4.8 EROSION AND SEDIMENTATION CONTROL PROVISIONS (Rule .1626 (7))

Surface water at the PLRC is managed through a series of landfill benches, perimeter channels, and three sedimentation ponds. Refer to Sheet 6 of the design drawings for the plan view of the Surface Water Management System. At a minimum, the landfill benches and perimeter channels were designed to convey the run off from the 100 year, 24-hour storm without overtopping. The sedimentation ponds were designed to detain run off from the 25-year, 24-hour storm with one foot of freeboard to the emergency spillway crest, and the 100-year, 24-hour storm with one foot of freeboard to the pond crest.

Vegetation will be established within 30 working days of completion of construction of that component. Other types of erosion control devices (such as riprap or gabions) will be constructed within the same time frame, as necessary. Temporary erosion control devices (such as silt fencing) will be installed as needed to minimize sediment discharge from unvegetated areas.

4.9 DRAINAGE CONTROL AND WATER PROTECTION PROVISIONS (Rule .1625 (8))

In conjunction with the erosion and sedimentation control provisions discussed in Section 4.8, surface water will be diverted from the landfill area by a series of terrace and drainage ditches. The surface water will then be detained in sedimentation ponds surrounding the landfill prior to discharge off-site. Surface water which comes in contact with waste will be contained in the landfill and treated as leachate. This should only occur at the working face of the landfill. Leachate will be stored on-site prior to off-site treatment. Leachate will not be discharged to surface waters.

The PLRC will also provide additional water protection provisions in order to comply with certain Division and Clean Water Act requirements.

4.10 LIQUID WASTE RESTRICTIONS (Rule .1626 (9))

The PLRC will not accept bulk or non-containerized liquid waste for disposal. Exceptions to this include household wastes and leachate and gas condensate derived from the landfill. Containerized waste will not be placed in the landfill unless the container is similar in size

to that normally found in household waste, the container is designed to hold liquids for use other than storage, or the container is household waste.

Liquid wastes are defined as waste materials determined to contain "free liquids" as defined by Method 9095 (Paint Filter Liquids Test), S.W. 846.

4.11 RECORDKEEPING PROVISIONS (Rule .1626 (10))

The PLRC manager will record and retain at the landfill an operating record at the facility which contains the following information as it becomes available:

1. Inspection records, waste determination records, and employee training procedures.

2. Amounts, by weight, of solid waste received at the facility including sources of generation, broken down by County.

3. Gas monitoring results and any remediation plans, if necessary.

4. Demonstration, certification, finding, monitoring, testing, or analytical data required by Rule .1630 through .1637 of the regulations.

5. Monitoring, testing, and analytical data as required by Rule .1627.

6. Cost estimates and financial assurance documentation as required by Rule .1628.

In addition, all information contained in the operating record will be furnished upon request to the Division or be made available at all reasonable times for inspection by the Division.

4.12 SPREADING AND COMPACTING PROVISIONS (Rule .1626 (11))

~~The PLRC will compact all solid waste to a maximum practical density, minimize void space, and increase stability. The daily working area will be restricted to the smallest area feasible for efficient operations.~~

See Section 4.1.6 for a discussion on spreading and compacting provisions.

4.13 LEACHATE MANAGEMENT PLAN (Rule .1626 (12))

The proposed leachate management system for the PLRC is provided on the design drawings. A Leachate Management Plan is included in Appendix B.

5.0 ENVIRONMENTAL MONITORING

5.1 LEACHATE MONITORING

See attached Appendix B for the Leachate Management Plan.

5.2 GROUNDWATER MONITORING

A Water Quality Monitoring Plan has been prepared for the PLRC and is provided. Refer to this plan for detailed information concerning groundwater monitoring at the facility.

5.3 SURFACE WATER MONITORING

A discussion of Surface Water Monitoring is included in the Water Quality Monitoring Plan.

OPERATIONS PLAN

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, NORTH CAROLINA

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APPENDIX C

Alternative Daily Cover

APPENDIX C

ALTERNATIVE DAILY COVER (ADC)

ADC MATERIAL

The ADC will consist of a series of waterproof panels placed over the working face in a shingle like fashion. The material typically used is a strong crosswoven fabric, laminated with plastic. It is a material that resists the propagation of small tears and holes. The material is thin and relatively lightweight. While site specifics will dictate the most appropriate panel size, actual panel sizes are typically rectangular, ranging from 25' X 25' to 100' X 100' in plan dimension.

ADC METHODOLOGY

Each panel is applied such that its adjacent lower and side panels or adjacent soil cover are overlapped. If a problem with high winds is expected, the joints between panels can be secured if necessary by the use of weights such as sandbags, tires, chunks of concrete, etc. or by the placement of conventional cover material along the top edge of each panel. The ADC may be applied by hand or machine. If by hand, at least two people are needed to handle the material, though three would be desirable. In the case of hand placement, the panels are pulled back in the morning and replaced in the evening so that an area is exposed sufficiently large for that day's waste. If the ADC is maneuvered with a machine, it will be outfitted with a spreaderbar or similar device to support and place the panels. In this case, the panel edges are outfitted with grommets. A spreaderbar with hooks that can fit into these grommets is fixed to a front-end loader, backhoe, crane or similar machine. At night, the panels are "hooked onto the machine by hand and then are walked to the area to be covered. The panels are lowered and unhooked. The removal in the morning is just the reverse.

The panels are typically placed on the surface of the daily working face. The ADC sheds water in the same manner as water is normally drained from the landfill. Surfaces of a working face are normally supplied with a soil daily cover if it is in the area that requires truck trafficability. However, in areas where trucks do not run, the ADC may be used in accordance with this plan. In no event are trucks allowed to travel on top of the ADC panels. Placing ADC panels on horizontal portions of the working face do not alter the current drainage plan from the site.

Minor rips or penetration of the panels can be expected to occur and are acceptable. Any significant damage to the panels, however, must be repaired or the panel must be removed from service. As a contingency for those times when the ADC panels cannot be used, there will always be sufficient soil to cover the waste.

ADC ADVANTAGES

There have been a number of obvious advantages realized by using an ADC. There is the immediate benefit of soil cover savings for sites that are short of soil. The "other end" of that benefit is air space savings in that more refuse can be filled for a given volume. The environmental benefits is having successive daily cells and lifts interconnected. This interconnection allows for less obstructed downward flow of leachate to the collection system, and upwards/lateral migration of gas to the active venting and flare system. The objectives or goals of a daily cover are met as well.

1. Control of Odors

The use of ADC in conjunction with the occasional use of soil cover has not appreciably changed the presence of odors. The most likely reason for this is because the refuse left from one day is covered with "new" refuse in less than 48 hours. In other words, the refuse is not exposed to the air long enough to start producing significant odors. More importantly, however, is the degree to which the refuse is compacted. The "tighter" the working face, the slower and more controlled will be the generation of odors. There is a direct relationship between exposed surface area and odor.

2. Control of Vectors

It has been clearly demonstrated that as important as cover is to meeting this objective, proper compaction is a major factor. No increase in flies or other insects have been noticed when the ADC has been used.

3. Control of Blowing Debris

The objective to control blowing debris is easily met by the ADC. However, precautions must be taken to insure that ADC doesn't blow away. The material is heavy enough that it generally takes a 25-30 mph wind before the application of weights around the edges is necessary. Generally, there is no more blowing debris when using the ADC vs. soil cover.

4. Control of Water Infiltration

The material used for ADC cannot absorb nor allow the infiltration of waste by water at the working face. Typically, the use of ADC is more effective at meeting this objective than soil cover.

5. Control of Fires

The chance of fires in a properly maintained operated sanitary landfill, with good compaction, waste screening, and operational controls is not dependent on the use of soil cover. The use of ADC does not preclude the need for adequate stockpiles of soil near any working face, nor the proper and complete fire fighting equipment, training, etc..

6. Aesthetics

Since this is a portable item and one that is not a permanent part of the site, the ADC panels do not in any way detract from the overall site aesthetics. The ADC panels are advantageous in delineating/defining that area of the landfill which is being worked. Considering the limited number of people who actually see the working face, this is not a major concern.

APPENDIX D

Safety Provisions

APPENDIX D

A typical agenda presently taught at routine safety meetings held at the PLRC include, in addition to good/safe operating practices/procedures:

- Special waste recognition and procedures for handling.
- Asbestos training.
- Right-to Know training.
- Emergency response training.
- Spill prevention.
- Ethics in the workplace
- Lock out/tag out.

WASTE MANAGEMENT OF CAROLINAS, INC.

CLOSURE AND POST-CLOSURE CARE PLAN

FOR

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, NORTH CAROLINA

Prepared By:
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TABLE 1 SUMMARY OF POST-CLOSURE CARE ACTIVITIES

1.0 INTRODUCTION

1.1 PURPOSE

The Piedmont Landfill and Recycling Center (PLRC) is presenting this Closure and Post-Closure Care Plan as part of a Transition Application for the existing portion of the landfill. It is also submitted as part of a Permit Renewal for the currently permitted but not yet constructed portion of the landfill. These permit applications will be submitted to the North Carolina Department of Environment, Health and Natural Resources - Division of Solid Waste Management (Division).

Under the requirements of Rule .1629 of the North Carolina Solid Waste Management Regulations, the facility must submit a written Closure and Post-Closure Care Plan to the Division. The plan must present information concerning landfill closure activities, post-closure care activities, and financial assurance for these activities, pursuant to Rules .1627 and .1629. This plan contains the necessary information to meet the above-referenced requirements for the PLRC.

This Closure and Post-Closure Care Plan should be read in conjunction with the following documents:

- The North Carolina Solid Waste Management Regulations
- Design drawings entitled "Site Development Plan for Piedmont Landfill and Recycling Center, Forsyth County, North Carolina," prepared for Waste Management of Carolina's, Inc., by RUST Environment and Infrastructure, Inc., Solid Waste Division, October 1993 (Volume 2 of this submittal).

1.2 SITE DESCRIPTION

The PLRC is located on approximately 108 acres of property in Forsyth County, North Carolina, approximately 10 miles north of the city of Kernersville. The landfill disposal area occupies an approximate foot print of 68$\pm$ acres. Waste Management of Carolinas, Inc., a subsidiary of Waste Management, Inc., owns and operates the facility. Currently, approximately 30 acres of the landfill have been developed for landfilling operations. A composite liner system on the landfill base consisting of 18 inches compacted cohesive soil with a maximum hydraulic conductivity of $1x10^{-7}$ cm/sec overlain by a 60 mil HDPE geomembrane exists in Phase I, Modules 1 and 2. A double synthetic with primary and secondary leachate collection systems is incorporated in existing Phase I, Modules 3, 5 and 6 and Phase II, Modules 1 and 2. The future fill areas will incorporate a composite liner system which meets the requirements of the Solid Waste Management Regulations. The location and layout of the site is detailed on the design drawings.

2.0 CLOSURE PLAN (Rule .1629(b))

Pursuant to Rule .1629 of the North Carolina Solid Waste Management Regulations, the PLRC will prepare a written closure plan describing the activities involved in closing the landfill facility. The following information describes the planned activities involved in closing the facility pursuant to Rules .1629 (b)(1) and .1627 (c).

2.1 DIVISION NOTIFICATION OF CLOSURE (Rule .1627 (c)(4) and (5))

The PLRC will notify the Division of the intent to close the entire landfill facility prior to the beginning of closure activities. The notice of intent to close the facility will also be placed in the facility operating record. This notification of the intent to close the landfill will mark the initiation of closure activities and will begin no later than 30 days after the receipt of the final volume of waste.

2.2 ON-SITE WASTE INVENTORY (Rule .1629(b)(1)(c))

~~The PLRC does not store wastes on the facility property outside of the landfill disposal area. A citizen's drop off facility exists at the facility and is collected on an as needed basis. The drop off facility contains minimal amounts of waste and is not used for long term storage. As a result, there is not anticipated to be an on site waste inventory during the course of landfill operations.~~

The estimate of the maximum inventory of waste ever on site over the active life of the landfill facility is 6,213,322 gate tons as of January 31, 1994. See Appendix B of the Engineering Plan.

2.3 EQUIPMENT DECONTAMINATION

Decontamination of operation equipment following determination of facility closure will involve the removal of refuse from all equipment surfaces and cleaning of residues from these surfaces. Removal of refuse from equipment surfaces will include manually dislodging the refuse from equipment tracks and undercarriages, and any other areas where loose refuse may have been lodged. Refuse residues can be manually washed from the equipment surfaces. All refuse collected from the equipment decontamination process will be placed in the facility fill area prior to final closure. Decontamination of the equipment involved in the disposal of the final loads of refuse will occur at an approved equipment washing location. Equipment which has not come into contact with refuse, such as earth moving equipment, will not require decontamination.

2.4 FINAL COVER DESCRIPTION (Rule .1627(c)(1) and .1629 (b)(A))

The landfill final cover for the PLRC consists of the following components in ascending order: an 18 inch thick compacted cohesive soil layer with a maximum saturated hydraulic conductivity of 1×10^{-5} cm/sec, a 40 mil coextruded VLDPE and HDPE geomembrane, an HDPE geotextile/geonet composite drainage layer, and a two foot thick protective soil layer.

At minimum, the top six inches of the protective soil layer will be capable of supporting vegetation.

Upon completion of the placement of final cover materials, seeding will be performed to establish vegetation. Vegetation will be used on all areas of the facility requiring vegetation. Fertilizer may also be applied to the soil if determined necessary. Mulch may be applied to minimize soil erosion on sloped areas prior to the establishment of vegetation.

2.5 ALTERNATIVE FINAL COVER (Rule .1627 (c)(2))

The final cover previously described meets the requirements pursuant to Rule .1627 (c)(1). No alternative final cover type is currently proposed to the Division for approval.

2.6 FINAL COVER CONSTRUCTION (Rule .1627 (c)(3) and .1629 (b)(1)(A) and (B))

The landfill final cover at the PLRC will be constructed pursuant to the requirements specified under Rule .1624 (b)(8), (9) and (15) pertaining to compacted clay liners. Refer to the Engineering Report for information regarding the construction techniques performed in the preparation of the compacted clay liners.

Based on the anticipated fill rate at the facility, the entire facility may be final covered at one time. However, total cover requirements may be less in the event of premature closure of the facility as well as routine final closure of portions of the facility at any time during its active life. The closure cost estimate of Section 4.1 of this plan assumes the facility will be filled to maximum capacity.

The final grades at the PLRC have been designed with maximum slopes of 25 percent and minimum slopes of six percent. In the event of differential settlement of the landfill surface, the grades will be maintained to ensure a minimum slope of five percent by backfilling settled areas with soil, grading to provide proper slope and revegetating. A gas venting layer is not proposed as a component of the final cover. However, an active landfill gas extraction/collection system is proposed for the facility. This system represents a proactive approach to managing landfill gas to avoid potential gas problems.

The type vegetation/fertilizer/mulch that will be used on the final cover will be in accordance with the existing State approved Sediment and Erosion Control Permit. See Appendix C for the specific vegetation/fertilizer/mulch requirements.

Refer to the Landfill Gas Management System Plan in the design drawings for further information on landfill slopes and gas management system components.

2.7 SURFACE WATER MANAGEMENT PREPARATION

The surface water management system for the PLRC will be continually developed as needed during the active life of the facility as areas of the landfill become developed. Landfill benches and perimeter channels will be constructed to manage surface water run-off

originating from the landfill. Sedimentation ponds and some perimeter ditches currently exist at the facility. Temporary erosion control measures (such as silt fencing and hay bales) will be situated to minimize sediment discharge from areas which have yet to establish vegetation.

Following determination of the necessity of final closure, components of the surface water management system not yet constructed will be completed. Refer to the Surface Water Management Plan in the design drawings for locations of system components. In addition, necessary earthwork will be performed to promote effective surface water management from other areas of the facility property which may have been disturbed by operations. Grading of these areas will be performed to promote positive drainage toward surface water management features. Vegetation will be established as needed. Every effort will be made to prevent both point and non-point discharges of sediment from the facility.

2.8 LANDFILL GAS AND LEACHATE MANAGEMENT

Following determination of the necessity of final closure at the PLRC, the landfill gas and leachate management systems will be completed. Refer to the Leachate Management and Landfill Gas Management System Plans in the design drawings for further information. It is assumed that construction of the leachate management system will be complete at this time and that minor effort will be expended to ensure that the system is prepared for operation throughout the post-closure care period.

Construction of the active landfill gas management system will begin at the west portion of the facility where operations began. The system will be continually expanded to the east as landfill development advances and gas generation increases. The gas management system will not be completed during the closure process because gas generation will not warrant additional wells at that time. However, additional wells will be added to the system during the post-closure care period as determined necessary.

2.9 REMOVAL OF ON-SITE STRUCTURES

Upon completion of closure activities at the PLRC, equipment and structures unnecessary for post-closure landfill care will be removed. These include, but are not limited to, the waste compaction equipment, earthmoving equipment, and the facility scales. Areas of the facility which are disturbed by structure removal will be repaired and landscaped to blend with the surroundings. Structures and equipment required for post-closure care will remain on-site.

2.10 CLOSURE CERTIFICATION (Rule .1627 (c)(7))

A registered professional engineer will certify that closure has been completed in accordance with the appropriate regulations and this Closure Plan. The Division will be provided with the appropriate certification of closure. In addition, the certification of closure will be placed in the facility operating record.

2.11 DEED RECORDING (Rule .1627 (c)(8) and (9))

Following completion of closure activities, a notation will be placed on the property deed, or some other instrument normally examined during a title search, indicating that the property has been used as a landfill facility. The notice on the deed will indicate in perpetuity that buried wastes are present on the property and that property use is restricted under this Closure and Post-Closure Care Plan. A copy of the deed with the notation will be kept in the facility operating record.

2.12 CLOSURE SCHEDULE (Rule .1627 (c)(5) and (6) and .1629 (b)(1)(D))

Completion of final landfill closure tasks are extremely time and weather dependent. The following time frame assumes normal weather conditions, with the acceptance of the final waste volume occurring early in the construction season. The closure process will be initiated on the date the Division has been notified of the intent to close the facility. This initiation process will begin no later than 30 days following the acceptance of the final waste volume.

In the event the final waste volume is received after or near the end of the construction season, and poor weather hampers closure activities, an intermediate cover will be placed and maintained over the waste. Final cover will then be installed as weather permits. As scheduled, closure activities will be completed within 180 days following the initiation of closure activities. If poor weather or other factors threaten completion within the 180 day period, an extension will be requested of the Division.

The following table presents a schedule to complete facility closure.

PIEDMONT LANDFILL & RECYCLING CENTER FACILITY CLOSURE SCHEDULE													
	TIME PROJECTION (Weeks)*												
TASK	2	4	6	8	10	12	14	16	18	20	22	24	26
1. Notify Agency & Initiate Closure													
2. Equipment Decontamination													
3. Barrier Soil Layer Placement													
4. Geosynthetics Placement													
5. Protective Soil Layer Placement													
6. Surface Water Management													
7. Vegetation													
8. Gas Control													
9. Removal of On-Site Structures													
10. Certification of Closure													

*NOTE: The above task schedule is approximate and is subject to weather conditions.

3.0 POST-CLOSURE CARE PLAN (Rule .1629(c))

Pursuant to Rule .1629 of the North Carolina Solid Waste Management Regulations, the PLRC will prepare a written post-closure care plan describing the activities involved in the post-closure care of the landfill facility. The following information describes the planned activities involved in providing care for the facility during the post-closure period pursuant to Rules .1629 (c)(1) - (3) and .1627 (d). Post-closure care will be provided for a period of 30 years.

3.1 INSPECTION, MAINTENANCE AND MONITORING (Rule .1627 (d)(1) and .1629 (c)(1))

Activities performed during the post-closure care period include routine scheduled inspection and maintenance programs at the facility to ensure the integrity of facility components. These components include, but are not limited to, the landfill final cover, leachate management system, groundwater monitoring system and landfill gas management system. In addition, routine monitoring for leachate levels, groundwater quality, surface water quality, and gas migration will continue. Table 1, provided at the end of this plan, provides a summary of the post-closure care inspection, maintenance and monitoring activities along with the frequencies the activities will be performed.

3.1.1 Final Cover (Rule .1627 (d)(1)(A))

The landfill final cover will be inspected quarterly during the first five years of the post-closure care period. The cover will be inspected for signs of erosion, differential settlement, vegetation stress, desiccation, or other defects which may sacrifice its integrity. After these five years have passed, the quarterly inspections may be reduced as vegetation becomes well established and the rate of differential settlement lessons. This reduction in inspection frequency will require justification in the form of reduced maintenance requirements.

The final cover will be repaired in areas where significant rills, gullies, and crevices have been identified. Areas of final cover which are identified as being particularly susceptible to soil erosion or desiccation will be repaired. Also, holes or depressions which had been created by settling and may promote ponding of surface water will be repaired. The landfill final slope on the plateau will be maintained at a minimum of five percent, pursuant to Rule .1627 (c)(3)(A). This will be accomplished by backfilling settled and defective final cover areas with soil and regrading to ensure the minimum slope requirements are met and the final cover integrity is ensured.

Areas of final cover which have been repaired due to erosion, desiccation, settlement or other causes will be immediately vegetated. As you can see from the schedule in Appendix C, it is expected that refertilization will occur in the second year. Mowing should not be necessary.

3.1.2 Leachate Management System (Rule .1627 (d)(1)(B))

A leachate management system will be installed within the PLRC prior to the beginning of the post-closure care period. It is anticipated that the completed leachate management system will be sufficient to manage the leachate generated during the post-closure period and no further modifications to the system are necessary. This system is shown in detail on the Leachate Management System Plan and Detail sheets in the design drawings. Also, refer to Section 2.8 of this Closure and Post-Closure Care Plan for further description of the system.

The leachate force main system incorporated at the facility will automatically discharge leachate from the collection sumps when liquid levels reach a predetermined elevation. The leachate will then be pumped to on-site storage, prior to discharge off-site. Manual monitoring of leachate levels will be performed routinely in the event of force main system shutdown. Routine maintenance on the system components will be performed according to manufacturer's specifications. In addition, inspections during the first 3 years of the operating period of the system components for potential defects will be performed on a quarterly basis.

The quantities of leachate generated by the facility will be recorded and maintained in the facility operating record.

Refer to the Operations Plan of the PLRC for further information concerning leachate management at the facility.

3.1.3 Groundwater Monitoring System (Rule .1627 (d)(1)(C))

The PLRC has prepared a groundwater monitoring system in accordance with the requirements of Rules .1631 through .1637. The plan is provided under a separate cover. The plan describes the procedures to be performed in the sampling and analysis of groundwater quality. In addition, the plan describes routine inspection and maintenance procedures to be performed on the system components. Refer to this plan for detailed information on the operation of the groundwater monitoring system.

3.1.4 Surface Water Management System (Rule .1627 (d)(1)(C))

The PLRC will perform surface water monitoring as required by Rule .0602. Surface water discharge from the three sedimentation ponds will be monitored in accordance with the facility National Pollutant Discharge Elimination System Permit.

The facility will perform quarterly surface water management system inspection and maintenance during the first 5 years of the operating period. As with the final cover inspections, the frequency of the surface water management system inspections may be

reduced after five years, as vegetation becomes well established. Drainage channels which have become significantly scoured and eroded will be regraded to maintain design drainage capacity. Areas which have developed sediment buildup will be cleaned and restored to the proper capacity and dimensions.

Areas of the surface water management system which have been repaired due to erosion, scouring, regrading, or other causes will be immediately revegetated to minimize additional erosion and sediment discharge.

3.1.5 Landfill Gas Management System (Rule .1627 (d)(1)(D))

A Landfill Gas Management System will be incorporated at the PLRC, as shown in the design drawings. Routine maintenance will be performed on the system components in accordance with manufacturer's specifications. In addition, the system will be inspected quarterly for the first 5 years of the operating period to ensure the integrity of the system components.

A landfill gas monitoring plan has been prepared for the facility and is included in the facility Operations Plan. Monitoring for landfill gas migration and recording of the results of testing will be performed on quarterly basis. The results of the monitoring will be presented to the Division. Refer to the Operations Plan for further information on landfill gas monitoring at the PLRC.

3.1.6 Miscellaneous Features

In addition to performing inspections and maintenance on the aforementioned landfill features, general facility inspections will be performed for the entire PLRC. These inspections will be performed quarterly at minimum during the first 5 years of the operating period and will include inspecting facility security features, on-site structures, roads and other features. The inspections will involve touring the property and visually observing the area for indications of damage to components. Damaged items will be repaired or replaced.

3.2 FACILITY CONTACT (Rule .1629 (c)(2))

During the post-closure care period, all issues concerning the PLRC should be directed to:

Waste Management Inc.
2600 Delk Road
Marietta, Georgia
Telephone Number: (404) 859-0800

3.3 FACILITY END USE PLAN (Rule .1629 (c)(3))

During the post closure period, the PLRC property will be maintained and operated solely as a restricted landfill facility. The Landfill Gas Management System in the design drawings represents the final configuration of the facility. Post-closure use of the property will not disturb the integrity of the cap system, base liner system, or any other component of the containment and monitoring systems. Post-closure operations of the facility will conform to the provision described in this Closure and Post-Closure Care Plan.

3.4 ADJUSTMENT OF THE POST-CLOSURE CARE PERIOD (Rule .1627 (d)(2))

As mentioned in Section 3.0 of this plan, the post-closure care period at the PLRC will last 30 years. The length of the post-closure care period may be reduced by the Division if the facility demonstrates that human health and the environment will continue to be protected during this reduced period.

3.5 POST-CLOSURE CARE COMPLETION CERTIFICATION (Rule .1627 (d)(3))

Following completion of the post-closure care period at the PLRC, certification will be prepared to demonstrate post-closure care was completed in accordance with this plan. The certification will be prepared by a registered professional engineer, and will be provided to the Division. A copy of the certification will be placed in the facility operating record.

4.0 COST ESTIMATES AND FINANCIAL ASSURANCE (Rule .1629 (b)(3) and .1629(c)(4))

The following information provides a cost estimate for closure of the PLRC at a time in its active life when closure costs would be greatest. In addition, a cost estimate for post-closure care of the facility over the 30 year post-closure care period is provided.

4.1 CLOSURE COST ESTIMATE (Rule .1629 (b)(3))

See Appendix A for closure cost estimate and supporting information.

4.2 POST-CLOSURE CARE COST ESTIMATE (Rule .1629 (c)(4))

See Appendix B for post-closure cost estimate and supporting information.

TABLE 1 - SUMMARY OF POST-CLOSURE CARE ACTIVITIES	
ACTIVITY TO BE PERFORMED	**FREQUENCY**
Final Cover Inspection/Maintenance	Quarterly during the first five years of post-closure care. May be reduced after this if justified by reduced maintenance requirements.
Leachate Forcemain System Inspection/Maintenance	Routine maintenance according to manufacturer's specifications. Inspections during the first three years of the operating period of new system components.
Groundwater Monitoring	Refer to the site-specific groundwater monitoring plan.
Groundwater Monitoring System Inspection/Maintenance	Refer to the site-specific groundwater monitoring plan.
Surface Water Monitoring	Refer to the site-specific NPDES Permit requirements.
Surface Water Management System Inspection/Maintenance	Quarterly during the first five years of post-closure care. May be reduced after this if justified by reduced maintenance requirements.
Landfill Gas Migration Monitoring	Quarterly throughout the post-closure care period.
Landfill Gas Management System Inspection/Maintenance	Quarterly inspections during first five years of post-closure care. May be reduced after this if justified by reduced non-routine maintenance requirements. Routine maintenance per manufacturer's recommendations.
General Facility Inspection/Maintenance	Quarterly inspections during the first five years of post-closure care of entire facility. May be reduced after this if justified by reduced maintenance requirements.

CLOSURE AND POST CLOSURE PLAN

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, NORTH CAROLINA

List of Appendices

Closure Cost Estimate	A
Post Closure Cost Estimate	B
Vegetation Plan	C

APPENDIX A

Closure Cost Estimate

COST ASSUMPTIONS FOR CLOSURE/POST CLOSURE ESTIMATES:

Site Name: Piedmont Landfill & Recycling Ctr
Site Location: Kernersville, NC
Division President: William Lewis, P.E.
Prepared By: William Lewis, Ed Gibson
Date Prepared: March 1994

Projected Closure Date: 1999
Area Requiring Closure: 68
Area Landfilled: 30
Period of Post Closure Care (Years): 30
Basis for PC Period: North Carolina Solid Waste Regulations

ALL CONSTRUCTION COSTS BASED UPON WASTE MANAGEMENT, INC. EXPERIENCE AT THE PIEDMONT LANDFILL, OTHER SIMILAR LANDFILLS, AND PRESENT DAY CONTRACTOR EQUIPMENT COSTS.

FINAL COVER

Fine Graded Waste

Cohesive Soil Thickness (in):	18	Evidence indicates that on-site soil can be 1X10-5 cm/sec
Geomembrane:	40 mil VLDPE	
Drainage Layer:	Texnet	
Protective Soil Thickness (in):	24	Assume upper 6 inches will be prepared with fertilizer, lime; topsoil unnecessary.
Shrinkage Factor for Soils:	25%	Conservative for silty sands
Clay Soil Placement Rate (cy/hr):	160	
Cost per CY to Import Clay Soils:	$0.00	
Acres of Synthetics Required:	68	
Synthetic Placement Rate (ac/day):	3	
Cover Soil Placement Rate (cy/hr):	160	
Cost per CY to Import Topsoil:	$0.00	
Cost per acre for seeding, fertilizing:	$1,100	

ASSUMPTIONS (Continued)

POST CLOSURE:

Sediment Ponds Currently Permitted:	3	
# of G/W Mon Wells Currently Permitted	6	
Capital Cost per G/W Well:	$8,500	
# of Gas Probes Currently Constructed	5	
Capital Cost per Gas Probe:	$1,500	
Leachate Sumps Currently Permitted:	8	
Leachate Pump Capital Cost:	$7,500	
Capital - Leachate Tank #1:	$7,000	
Capital - Leachate Tank #2:	$7,000	
Capital - Leachate Tank #3:	$0	
Capital - Leachate Tank #4:	$0	
Misc Capital - Leachate System:		
Capital in Gas Mgmt System to date:	$0	
G/W Monitoring cost per well per event:	$1,200	
Freq. Events/Year, First 10 yr:	2	
Surface Mon Points currently Permitted:	5	
Surf Water Mon Cost per point per even	$900	
Freq. Events/Year, First 10 yr:	2	
Landfill Gas Mon. Cost per Event:	$600	(2 men @ $35 each; 8 hrs/event, $50 expenses)
Annual Leachate Mon. Cost:	$3,000	
Clay Cap Acres:	0	
Synthetic Cap Acres:	68	
Leachate from other areas (gal):	0	
Cost per gal to Haul Leachate:	$0.0300	(Present cost)
Cost per gal to Treat Leachate:	$0.0750	(Present cost)

WMI LANDFILL CLOSURE COST ESTIMATE SUMMARY
1994 BUDGET

SITE:	Piedmont Landfill & Recycling Ctr		
LOCATION	Kernersville, NC		
DIV. PRES:	William Lewis, P.E.		
PREPARED BY:	William Lewis, Ed Gibson	**DATE:**	March 1994
REVIEWED BY		**DATE:**	

I. CLOSURE DATA

PROJECTED CLOSURE DATE: 1999
AREA REQUIRING CLOSURE: 68 ACRES

II. CLOSURE COSTS

GRADING OF WASTE	$34,000
SUPPLY AND PLACEMENT OF COVER SOIL	$411,400
SUPPLY AND PLACEMENT OF GEOSYNTHETICS	$3,452,240
SUPPLY AND PLACEMENT OF TOPSOIL OR ALTERNATE	$523,600
SUPPLY AND APPLICATION OF SEED	$74,800
EARTHWORKS FOR STORMWATER MANAGEMENT	$0
STORMWATER MANAGEMENT STRUCTURES	$0
LEACHATE COLLECTION SYSTEM	$0
GAS CONTROL SYSTEM	$0
SECURITY, FENCING, GATES, SIGNS, ACCESS, ETC.	$0
CLOSURE CERTIFICATION	$2,627,000
EXCEPTIONAL ITEMS (ITEMIZED)	$0
TOTAL	**$7,123,040**

Piedmont Landfill & Recycling Ctr
1994 CLOSURE COST ESTIMATE SUMMARY
PAGE 1

FINAL COVER PROFILE:

Fine Graded Waste	
Cohesive Soil Thickness (in):	18
Geomembrane:	40 mil VLDPE
Drainage Layer:	Texnet
Protective Soil Thickness (in):	24
Shrinkage Factor for Imported Soils:	25%

FINE GRADE WASTE:

Final preparation of waste prior to 18" soil placement
D-8 Dozer, 5 hours/acre @ $100

TOTAL FINE GRADE WASTE= **$34,000**

SUPPLY AND PLACEMENT OF COVER SOILS

INSTALL COHESIVE SOIL

Soil must be free of large particles that could damage liner.

CY Required Per Acre: 2420

Hourly Cost including Operators, Fuel, Maint, etc.:

D-8 Dozer:	$90
Komatsu 300 Track Hoe:	$95
Volvo Dump Truck:	$85
Volvo Dump Truck:	$85
Supervisor with pickup:	$30
Total Hourly Cost:	$385

Placement Rate (cy/hr):	160	(this assumes source of cover is within 2000 feet of landfill)
Time to Place Cohesive Soil (hr):	15	
Cost to Place & Compact:		$5,775
Add 1 Smooth Drum Roller for 3 hours/ac at $45 =		$135
Rock Picking: 2 Laborers, 10 hours @ $7 =		$140
Total labor & equipment per acre to install cohesive soil=		$6,050
(Cost per cubic yard =	$2.50)	

TOTAL SUPPLY AND PLACEMENT OF COVER SOILS: **$411,400**

Piedmont Landfill & Recycling Ctr
1994 CLOSURE COST ESTIMATE SUMMARY
PAGE 2

SUPPLY AND PLACEMENT OF GEOSYNTHETICS

ACRES OF GEOSYNTHETICS REQUIRED:		68		
MOBILIZATION				
Lump Sum Per Acre		$500		
MATERIALS				
40 mil Textured VLDPE (sf)	$0.254			
Texnet - Fabric both sides (sf)	$0.493			
Total materials per sf =	$0.747			
Total Materials per acre =		$32,539		
INSTALLATION				
Install VLDPE (sf)	$0.180			
VLDPE Slope Surcharge (sf)	$0.094			
Install Texnet (sf)	$0.108			
Texnet Slope Surcharge (sf)	$0.025			
Total Install per sf =	$0.407			
Total Install per acre =		$17,729		
Total labor & materials per acre:			$50,768	
TOTAL SUPPLY AND PLACEMENT OF GEOSYNTHETICS=				**$3,452,240**

SUPPLY AND PLACEMENT OF TOPSOIL: (fertilize/lime on-site soil)

CY Required Per acre:	3227		
Placement Rate (cy/hr):	160		
Time to Place Cohesive Soil (hr):	20		
Cost to Place & Compact:		$7,700	
(Cost per cubic yard =	$2.39)		
			$523,600

SUPPLY AND APPLICATION OF SEED (includes fertilization/lime):

Cost Per Acre:	$1,100	
TOTAL SUPPLY AND PLACEMENT OF SEED=		**$74,800**

EARTHWORKS FOR STORMWATER MANAGEMENT:

TOTAL EARTHWORKS FOR STORMWATER MANAGEMENT=	**$0**

STORMWATER MANAGEMENT STRUCTURES:

TOTAL STORMWATER MANAGEMENT STRUCTURES=	**$0**

GAS CONTROL SYSTEM:

TOTAL GAS CONTROL SYSTEM =	**$0**

CQA/CLOSURE CERTIFICATION:

Assumed project size (acres):	68		
Survey lay-out prior to fine grading of waste (per acre)			
Survey Crew hours per acre:	5		
Survey Crew cost per hour:	$75		
Total lay-out cost per acre:		$375	
Const. time - Base Soil (days):	2		
Const time - Synthetics (days):	22.6667		
2 x Const. time - Veg. layer:	6		
Total const days:	30.6667		
Mob/Demob (total LS):	$4,000		
Mob/Demob per acre:		$59	
Inspector cost / day:	$1,200		
Inspector cost / acre:		$36,800	
Perm tests @ 1 per acre:		$325	
CQA Report (total):	$5,000		
CQA Report/acre:		$74	
As-Built (same as Lay-out):		$1,000	
Total per acre =		$38,632	
TOTAL CLOSURE CERTIFICATION =			**$2,627,000**

EXCEPTIONAL ITEMS:

APPENDIX B

Post Closure Cost Estimate

WMI LANDFILL POST-CLOSURE COST ESTIMATE SUMMARY

TERM = 30 YEARS

Site:	Piedmont Landfill & Recycling Ctr		
Location:	Kernersville, NC		
Div. Pres.:	William Lewis, P.E.		
Prepared By:	William Lewis, Ed Gibson	Date:	March 1994
Reviewed By:		Date:	

I. CLOSURE DATA

Date of Closure:	1999
Area Landfilled:	30 acres
Period of Post-Closure Care:	30 YEARS
Basis for Period:	North Carolina Solid Waste Regulations

II. MAINTENANCE COSTS

Security, Fencing, Gates, Signs, Access, Etc.	$15,000
Erosion Repair, Settlement Repair, Revegetation	$95,000
Surface Water Control Maintenance	$90,000
Monitoring System Maintenance, Repair, Replacement	$87,750
Leachate Collection System Maintenance, Repair, Replacement	$22,200
Gas Control System Maintenance, Repair, Replacement	$0

III. MONITORING COSTS

Groundwater & Surface Water	$468,000
Landfill Gas	$24,000
Air Quality	$0
Leachate Analysis	$90,000
Inspection Costs	$20,000

WMI LANDFILL POST-CLOSURE COST ESTIMATE SUMMARY
Piedmont Landfill & Recycling Ctr
Page 2

IV. ENVIRONMENTAL PROTECTION COSTS

Leachate Management	$727,006
Gas Control	$0
Surface Water Management	$0
Groundwater Management	$0

V. EXCEPTIONAL ITEMS

$40,000

TOTAL: **$1,678,956**

Site:	Piedmont Landfill & Recycling Ctr
Location:	Kernersville, NC
Division Pres.:	William Lewis, P.E.

Piedmont Landfill & Recycling Ctr
Post Closure Cost Estimate Detail - March 1994
TERM (YEARS): 30

II - MAINTENANCE COSTS

SECURITY, FENCING, ETC:

Estimated cost per year	$500	x TERM=	**$15,000**

EROSION, SETTLEMENT REPAIR & REVEGETATION,(Includes Vegetative Maintenance Program):

$10,000/yr for 1st 5 years
$5,000/yr for 2nd 5 years
$1,000/yr for last 20 years

TOTAL **$95,000**

SURFACE WATER CONTROL MAINTENANCE:

Number of Ponds at Closure, (Currently Permitted only):		3		
Events in Term @ 1 per 5 years:		6		
Cost per Pond per Event: 50 Hours per Pond @ $100:		$5,000		
# of Ponds x Events x Cost:				**$90,000**

MONITORING SYSTEM MAINT., REPAIR, REPLACEMENT:

# of GW Wells Curr. Permitted:		6		
Capital Investent/Well:	$8,500	$51,000		
# Gas Probes Curr. Permitted:		5		
Capital Investment/Probe:	$1,500	$7,500		
Total Capital, GW Wells & Probes:		$58,500		
5% of Capital Per Year:		$2,925	x TERM=	**$87,750**

LEACHATE COLLECTION SYSTEM MAINT, REPAIR, REPLACEMENT:

Leachate Sumps Permitted:	8			
Capital per pump (sump):	$7,500			
Pump Capital:		$60,000		
Leachate Tanks in Place:				
Capital - Leachate Tank #1:	$7,000			
Capital - Leachate Tank #2:	$7,000			
Capital - Leachate Tank #3:	$0			
Capital - Leachate Tank #4:	$0			
Tank Capital:	>>>>>>>>	$14,000		
Misc Capital:		$0		
TOTAL SYSTEM CAPITAL:		$74,000		
1% of Capital:		$740	x TERM=	**$22,200**

GAS CONTROL SYSTEM MAINT, REPAIR, REPLACEMENT:

Capital - Recovery Plant:	Assume Recovery Plants are self-supporting			
Capital - Mgmt System:		$0		
Total Capital		$0		
2% of Capital:		$0	x TERM=	**$0**

III. MONITORING COST

GROUNDWATER & SURFACE WATER:

# of GW Wells Currently Permitted:		6	
Cost per Well per Event:		$1,200	
Total Cost per Event:		$7,200	
Freq. Events/Year, First 10 yr:		2	
Events in First 10 Years:		20	
Events for Remainder of Term:		20	
	TOTAL EVENTS:	40	
Total Events x Cost per Event:		$288,000	
# of Surf. Points Curr. Permitted:		5	
Cost per Point per Event:		$900	
Total Cost per Event:		$4,500	
Freq. Events/Year, First 10 Yr:		2	
Events in First 10 Years:		20	
Events for Remainder of Term:		20	
	TOTAL EVENTS:	40	
Total Events x Cost per Event:		$180,000	
TOTAL COST - GROUNDWATER & SURFACE WATER:			**$468,000**

LANDFILL GAS:

Cost per Event:	$600		
Total Events in Term (same as GW):	40		**$24,000**

AIR QUALITY:
Not Required at Present

$0

LEACHATE ANALYSIS:

Annual Leachate Mon. Cost:	$3,000	
Term	30	
Cost x Term:		**$90,000**

INSPECTION COSTS

Freq. Events/Year, First 5 yrs:	4		
Total evts, 1st 5 yrs:		20	
Freq. Events/yr in Next 5 Years:	2		
Total evts, 2nd 5 yrs:		10	
Freq Events/yr for Remainder of Term:	1		
Total evts, Remainder of Term:		20	
TOTAL EVTS:		50	
Labor/event: 1 Man @ $50/hr, 8 hr inspection time:		$400	
Total Inspection costs:			**$20,000**

IV. ENVIRONMENTAL PROTECTION COSTS

LEACHATE MANAGEMENT:

LEACHATE PRODUCTION RATES:

Clay Cap:	1	in/ac/yr	27,152.4	gal/ac/yr
Synthetic Cap:	0.125	in/ac/yr	3,394.1	gal/ac/yr

AREAS OF LANDFILL:

Clay Cap:	0	acres	0.0	gal/yr
Synthetic Cap:	68	acres	230,795.4	gal/yr
Other Sources:			0.0	gal/yr
Totals:	68		230,795.4	gal/yr

Cost to haul:	$0.0300	per gal
Cost to treat:	$0.0750	per gal
Total:	$0.1050	per gal

LEACHATE COST PER YEAR:	$24,234	
LEACHATE COST FOR TERM:		**$727,006**

V. EXCEPTIONAL ITEMS

POST CLOSURE CERTIFICATION: (includes deed notation), estimate **$40,000**

TOTAL POST CLOSURE ESTIMATE: **$1,678,956**

APPENDIX C

Vegetation Plan

Table 6.11k
Seeding No. 1P for: Steep Slopes or Poor Soils; Low Maintenance

Seeding mixture

Species[1]	Rate (lb/acre)
Tall fescue	100
Sericea lespedeza	30
Kobe lespedeza	10

Seeding notes

1. In Eastern Piedmont add 25 lb/acre Pensacola Bahiagrass or 10 lb/acre common Bermudagrass. Use common Bermudagrass only where it is unlikely to become a pest.
2. After Aug. 15 use unscarified sericea seed.
3. Where a neat appearance is desired, omit sericea and substitute 40 lb/acre Bahiagrass or 15 lb/acre Bermudagrass.
4. To extend spring seeding dates into June, add 15 lb/acre hulled Bermudagrass. However, it is preferable to seed temporary cover and seed fescue in Sept.

Nurse plants

Between May 1 and Aug. 15, add 10 lb/acre German millet or 15 lb/acre Sudangrass. Prior to May 1 or after Aug. 15, add 40 lb/acre rye (grain).

Seeding dates

	Best	Possible
Fall:	Aug. 25 - Sept.15	Aug. 20 - Oct. 25
Late winter:	Feb. 15 - Mar. 21	Feb. 1 - Apr. 15

Fall is best for tall fescue and late winter for lespedezas. Overseeding of Kobe lespedeza over fall-seeded tall fescue is very effective. Use unhulled Bermudagrass seed in fall.

Soil amendments

Apply lime and fertilizer according to soil tests, or apply 4,000 lb/acre ground agricultural limestone and 1,000 lb/acre 10-10-10 fertilizer.

Mulch

Apply 4,000-5,000 lb/acre grain straw, or equivalent cover of another suitable mulching material. Anchor mulch by tacking with asphalt, roving, or netting. Netting is the preferred anchoring method on steep slopes.

Maintenance

Refertilize in the second year unless growth is fully adequate. May be mowed once or twice a year, but mowing is not necessary. Reseed, fertilize, and mulch damaged areas immediately.

[1]Refer to *Appendix 8.02* for botanical names.

TRANSITION APPLICATION AND PERMIT RENEWAL SITE DEVELOPMENT PLANS FOR WASTE MANAGEMENT OF CAROLINAS, INC.

PIEDMONT LANDFILL AND RECYCLING CENTER

FORSYTH COUNTY, NORTH CAROLINA

PERMIT NO. 34-06

LOCATION MAP

BELEWS CREEK QUADRANGLE
BELEWS CREEK, NC
36080-B1-TF-024

-N-

2000 0 2000 4000 6000
SCALE IN FEET

LIST OF DRAWINGS

SHEET NO.	TITLE
1	FACILITY PLAN
2	SUBGRADE PLAN
3	BASE GRADE PLAN (TOP OF COMPOSITE LINER)
4	LEACHATE MANAGEMENT PLAN (TOP OF PROTECTIVE COVER)
4a	PHASE 1 DEVELPMENT PLAN
4b	PHASE 2 DEVELOPMENT PLAN
5	FINAL DEVELOPMENT PLAN
6	SEDIMENT AND EROSION CONTROL PLAN
7	CROSS-SECTIONS A-A THROUGH C-C
8	CROSS-SECTIONS D-D THROUGH F-F
9	BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS
10	BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS
11	BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS
12	BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS
13	MISCELLANEOUS DETAILS
14	EROSION AND SEDIMENT CONTROL DETAILS
15	LEACHATE TRANSFER SYSTEM PLAN
16	LEACHATE TRANSFER SYSTEM DETAILS
17	GAS MANAGEMENT SYSTEM PLAN
18	GAS MANAGEMENT SYSTEM DETAILS
19	GAS MANAGEMENT SYSTEM DETAILS
20	GAS MANAGEMENT SYSTEM DETAILS
21	STOCKPILE AREA PLAN

RUST ENVIRONMENT & INFRASTRUCTURE

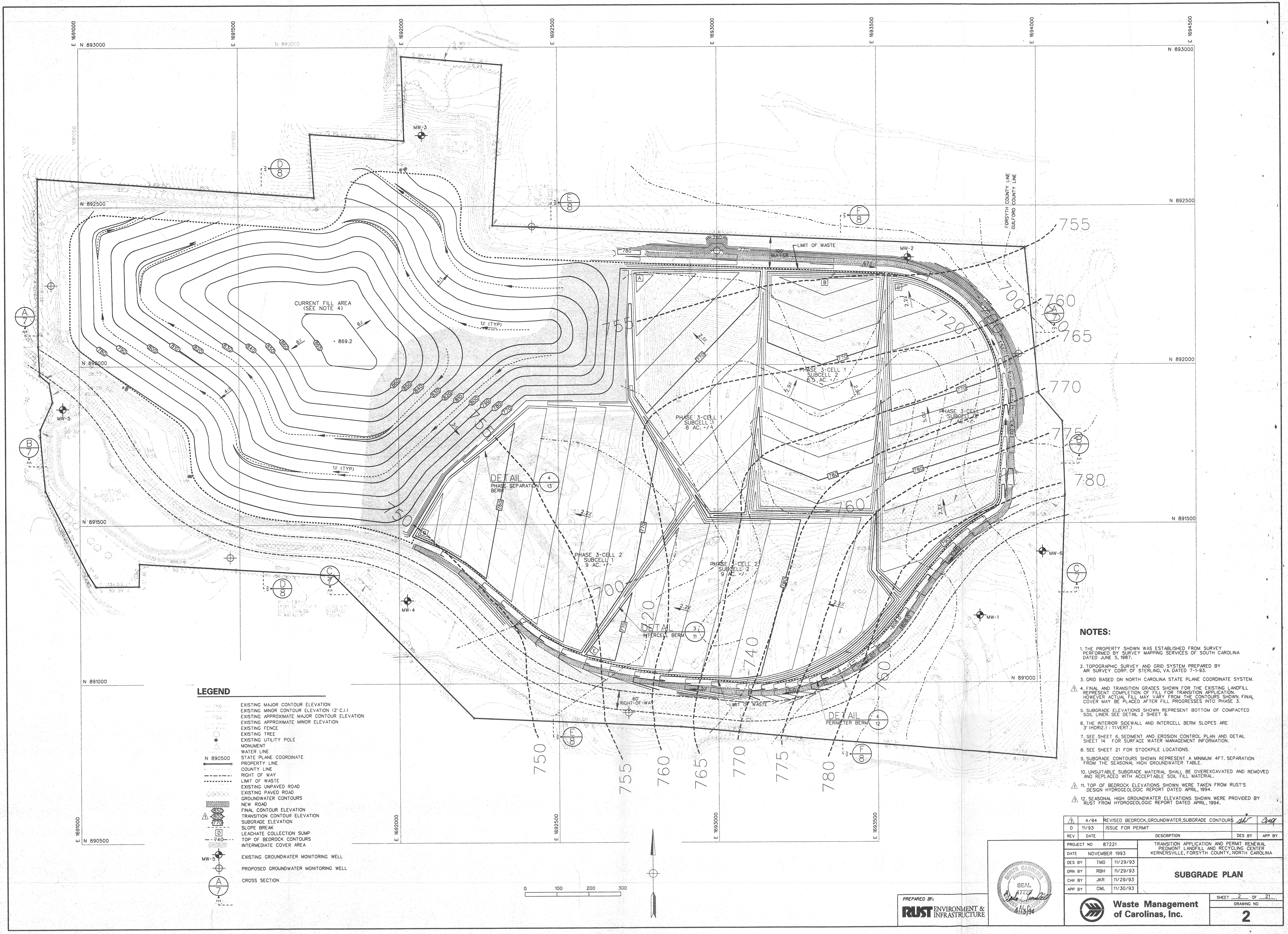
PREPARED BY:
RUST ENVIRONMENT & INFRASTRUCTURE
15 BRENDAN WAY
GREENVILLE, SOUTH CAROLINA 29615
(803) 234-3000

PREPARED FOR:
WASTE MANAGEMENT OF CAROLINAS, INC.
PIEDMONT LANDFILL AND RECYCLING CENTER
9900 FREEMAN ROAD
KERNERSVILLE, NORTH CAROLINA 27284

NORTH CAROLINA PROFESSIONAL SEAL 4/13/94 ENGINEER M. LUNDELL

REVISED: APRIL 1994
PROJECT NO. 87221

Apr 5 08:03:59 1994 \PLOT\QUEUE\MBOND.acf N:\MBOND\7221NCV1.PRF



LEGEND

- EXISTING MAJOR CONTOUR ELEVATION
- EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
- EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
- EXISTING APPROXIMATE MINOR ELEVATION
- EXISTING FENCE
- EXISTING TREE
- EXISTING UTILITY POLE
- MONUMENT
- WATER LINE
- N 890500 STATE PLANE COORDINATE
- PROPERTY LINE
- COUNTY LINE
- RIGHT OF WAY
- LIMIT OF WASTE
- EXISTING UNPAVED ROAD
- EXISTING PAVED ROAD
- GROUNDWATER CONTOURS
- NEW ROAD
- 850 FINAL CONTOUR ELEVATION
- 800 TRANSITION CONTOUR ELEVATION
- 770 SUBGRADE ELEVATION
- SLOPE BREAK
- D LEACHATE COLLECTION SUMP
- 740 TOP OF BEDROCK CONTOURS
- INTERMEDIATE COVER AREA
- MW-5 EXISTING GROUNDWATER MONITORING WELL
- PROPOSED GROUNDWATER MONITORING WELL
- A/7 CROSS SECTION

NOTES:

1. THE PROPERTY SHOWN WAS ESTABLISHED FROM SURVEY PERFORMED BY SURVEY MAPPING SERVICES OF SOUTH CAROLINA DATED JUNE 5, 1987.
2. TOPOGRAPHIC SURVEY AND GRID SYSTEM PREPARED BY AIR SURVEY CORP. OF STERLING, VA. DATED 7-1-93.
3. GRID BASED ON NORTH CAROLINA STATE PLANE COORDINATE SYSTEM.
4. FINAL AND TRANSITION GRADES SHOWN FOR THE EXISTING LANDFILL REPRESENT COMPLETION OF FILL FOR TRANSITION APPLICATION. HOWEVER ACTUAL FILL MAY VARY FROM THE CONTOURS SHOWN. FINAL COVER MAY BE PLACED AFTER FILL PROGRESSES INTO PHASE 3.
5. SUBGRADE ELEVATIONS SHOWN REPRESENT BOTTOM OF COMPACTED SOIL LINER. SEE DETAIL 2 SHEET 9.
6. THE INTERIOR SIDEWALL AND INTERCELL BERM SLOPES ARE 3' (HORIZ.) : 1'(VERT.)
7. SEE SHEET 6, SEDIMENT AND EROSION CONTROL PLAN AND DETAIL SHEET 14 FOR SURFACE WATER MANAGEMENT INFORMATION.
8. SEE SHEET 21 FOR STOCKPILE LOCATIONS.
9. SUBGRADE CONTOURS SHOWN REPRESENT A MINIMUM 4FT. SEPARATION FROM THE SEASONAL HIGH GROUNDWATER TABLE.
10. UNSUITABLE SUBGRADE MATERIAL SHALL BE OVEREXCAVATED AND REMOVED AND REPLACED WITH ACCEPTABLE SOIL FILL MATERIAL.
11. TOP OF BEDROCK ELEVATIONS SHOWN WERE TAKEN FROM RUST'S DESIGN HYDROGEOLOGIC REPORT DATED APRIL, 1994.
12. SEASONAL HIGH GROUNDWATER ELEVATIONS SHOWN WERE PROVIDED BY RUST FROM HYDROGEOLOGIC REPORT DATED APRIL, 1994.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISED BEDROCK,GROUNDWATER,SUBGRADE CONTOURS	JKR	CMJ
0	11/93	ISSUE FOR PERMIT		

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	TMD 11/29/93	
DRN BY	RBH 11/29/93	
CHK BY	JKR 11/29/93	
APP BY	CML 11/30/93	

SUBGRADE PLAN

PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE

NORTH CAROLINA SEAL 11/30/93

Waste Management of Carolinas, Inc.

SHEET 2 OF 21
DRAWING NO 2

7221NC2R.DGN

E 1691000 E 1691500 E 1692000 E 1692500 E 1693000 E 1693500 E 1694000 E 1694500

N 893000 N 892500 N 892000 N 891500 N 891000 N 890500

CURRENT FILL AREA
(SEE NOTE 4)

• 869.2

PHASE 3-CELL 1
SUBCELL 1
6 AC. +/-

PHASE 3-CELL 1
SUBCELL 2
6.5 AC. +/-

PHASE 3-CELL 1
SUBCELL 3
7 AC. +/-

PHASE 3-CELL 2
SUBCELL 1
9 AC. +/-

PHASE 3-CELL 2
SUBCELL 2
9 AC. +/-

DETAIL 4/13 PHASE SEPARATION BERM

DETAIL 3/11 INTERCELL BERM

DETAIL 4/12 PERIMETER BERM

LIMIT OF WASTE

100' BUFFER

60' RIGHT-OF-WAY

FORSYTH COUNTY LINE
GUILFORD COUNTY LINE

1% (TYP)

LEGEND

- EXISTING MAJOR CONTOUR ELEVATION
- EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
- EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
- EXISTING APPROXIMATE MINOR ELEVATION
- EXISTING FENCE
- EXISTING TREE
- EXISTING UTILITY POLE
- MONUMENT
- WATER LINE
- N 890500 STATE PLANE COORDINATE
- PROPERTY LINE
- COUNTY LINE
- RIGHT OF WAY
- LIMIT OF WAY
- EXISTING UNPAVED ROAD
- EXISTING PAVED ROAD
- NEW FENCE
- PERIMETER ACCESS ROAD
- 850 FINAL CONTOUR ELEVATION
- 1 800 INTERMEDIATE CONTOUR ELEVATION
- 770 BASE GRADE ELEVATION
- D SLOPE BREAK
- LEACHATE COLLECTION SUMP
- INTERMEDIATE COVER AREA
- A/7 CROSS SECTION

0 100 200 300

NOTES:

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5. BASE GRADE ELEVATIONS SHOWN REPRESENT TOP OF GEOMEMBRANE LINER. SEE DETAIL 2 SHEET 9.
6. THE INTERIOR SIDEWALL AND INTERCELL BERM SLOPES ARE 3' (HORIZ.) : 1' (VERT.)
7. SEE SHEET 6, SEDIMENT AND EROSION CONTROL PLAN AND DETAIL SHEET 14 FOR SURFACE WATER MANAGEMENT INFORMATION.
8. SEE SHEET 21 FOR STOCKPILE LOCATIONS.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISED BASEGRADE CONTOURS		
0	11/93	ISSUE FOR PERMIT		

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	TMD 11/29/93	
DRN BY	RBH 11/29/93	
CHK BY	JKR 11/29/93	
APP BY	CML 11/30/93	

BASE GRADE PLAN (TOP OF COMPOSITE LINER)

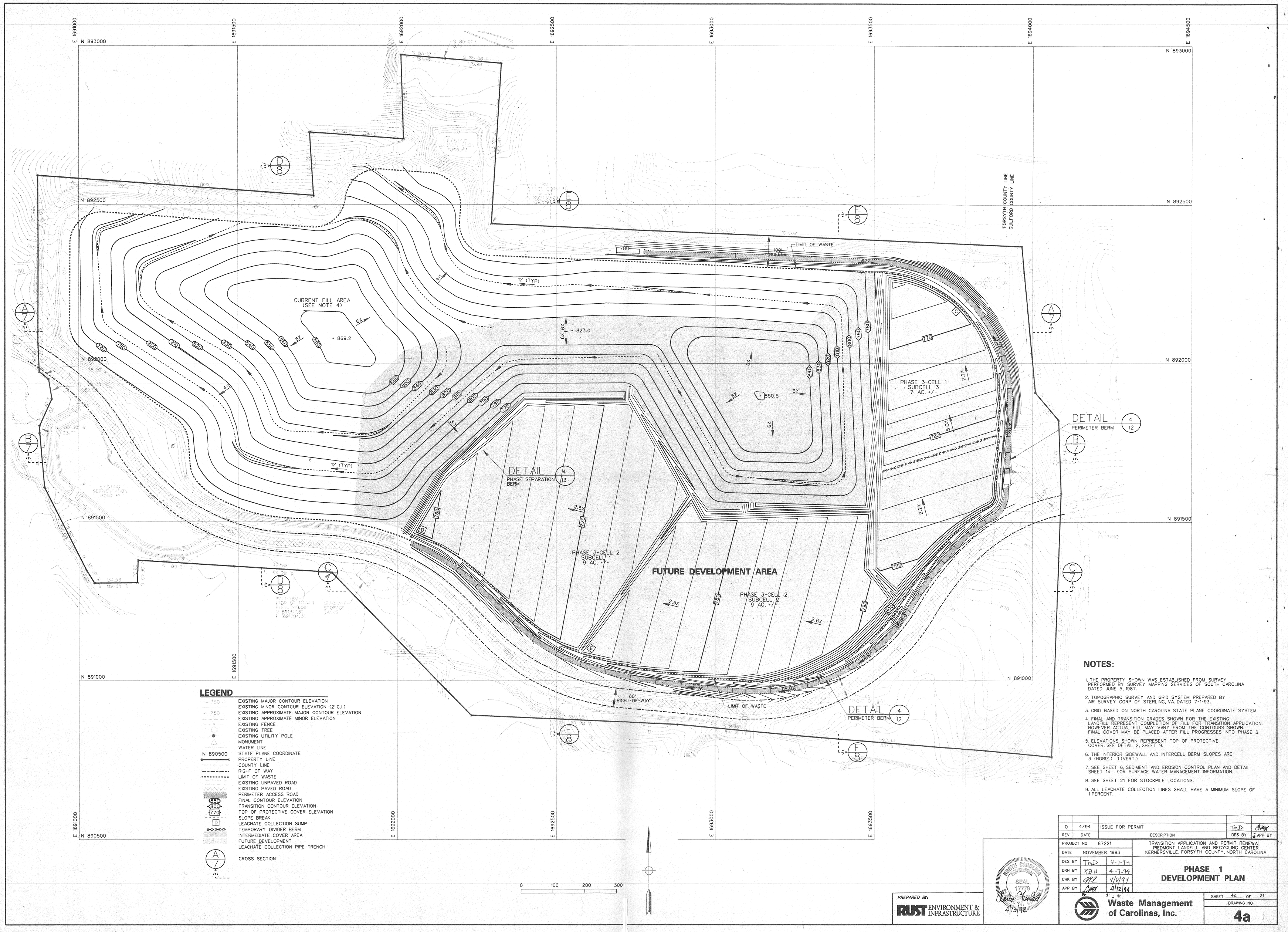
PREPARED BY:
RUST ENVIRONMENT & INFRASTRUCTURE

NORTH CAROLINA REGISTERED SEAL 4/13/94

Waste Management of Carolinas, Inc.

SHEET 3 OF 21
DRAWING NO 3

7221NCR3.DGN



LEGEND

750	EXISTING MAJOR CONTOUR ELEVATION
	EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
750	EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
	EXISTING APPROXIMATE MINOR ELEVATION
	EXISTING FENCE
	EXISTING TREE
	EXISTING UTILITY POLE
	MONUMENT
	WATER LINE
N 890500	STATE PLANE COORDINATE
	PROPERTY LINE
	COUNTY LINE
	RIGHT OF WAY
	LIMIT OF WASTE
	EXISTING UNPAVED ROAD
	EXISTING PAVED ROAD
	PERIMETER ACCESS ROAD
650	FINAL CONTOUR ELEVATION
800	TRANSITION CONTOUR ELEVATION
770	TOP OF PROTECTIVE COVER ELEVATION
	SLOPE BREAK
D	LEACHATE COLLECTION SUMP
	TEMPORARY DIVIDER BERM
	INTERMEDIATE COVER AREA
	FUTURE DEVELOPMENT
	LEACHATE COLLECTION PIPE TRENCH
A/7	CROSS SECTION

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5. ELEVATIONS SHOWN REPRESENT TOP OF PROTECTIVE COVER. SEE DETAIL 2, SHEET 9.
6. THE INTERIOR SIDEWALL AND INTERCELL BERM SLOPES ARE 3 (HORIZ.) : 1 (VERT.)
7. SEE SHEET 6, SEDIMENT AND EROSION CONTROL PLAN AND DETAIL SHEET 14 FOR SURFACE WATER MANAGEMENT INFORMATION.
8. SEE SHEET 21 FOR STOCKPILE LOCATIONS.
9. ALL LEACHATE COLLECTION LINES SHALL HAVE A MINIMUM SLOPE OF 1 PERCENT.

REV	DATE	DESCRIPTION	DES BY	APP BY
0	4/94	ISSUE FOR PERMIT	TmD	CMX

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	TmD 4-7-94	PHASE 1 DEVELOPMENT PLAN
DRN BY	RBH 4-7-94	
CHK BY	JKR 4/6/94	
APP BY	CMX 4/12/94	

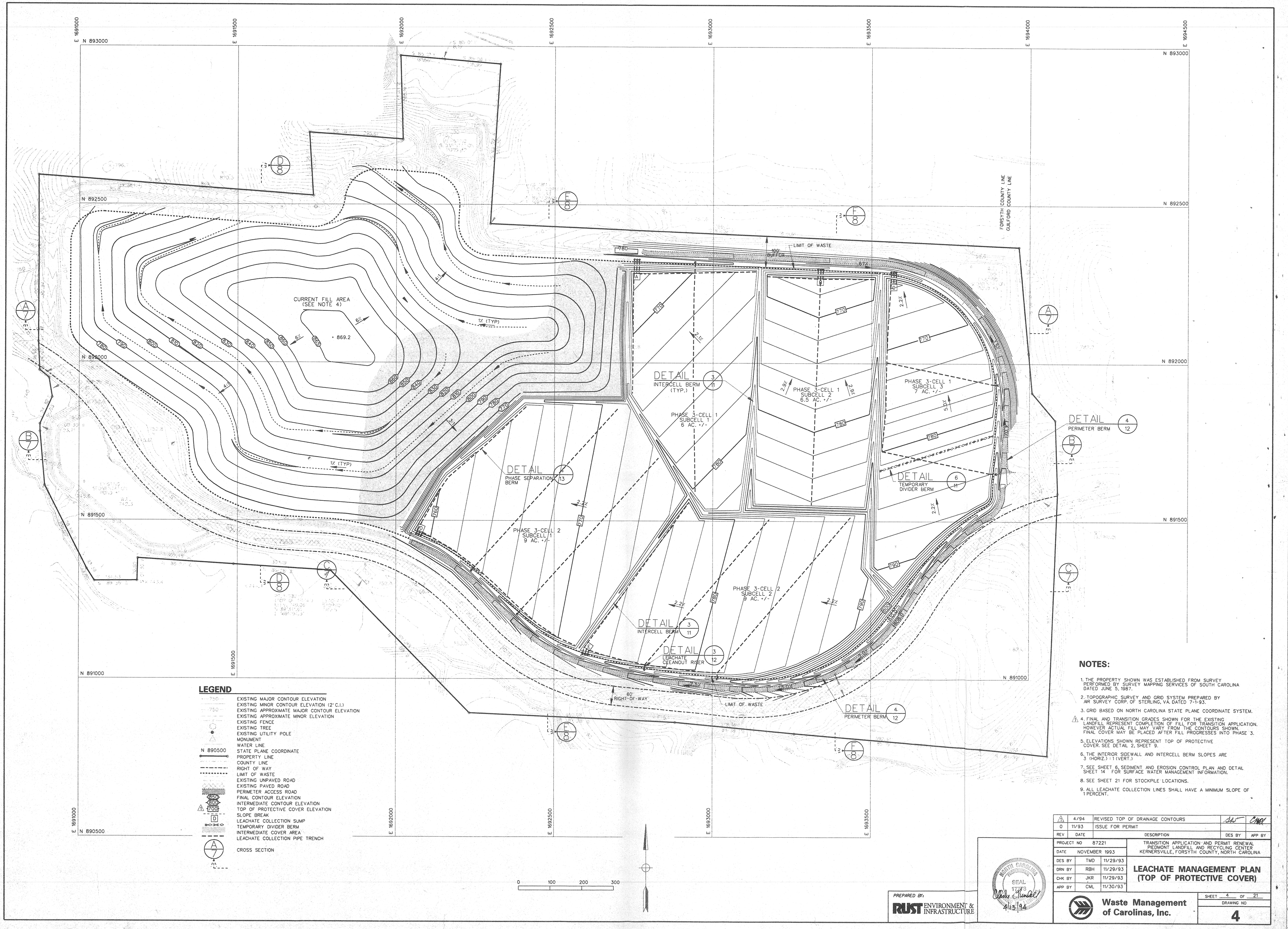
PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE

NORTH CAROLINA PROFESSIONAL SEAL 17778 ENGINEER 4/13/94

Waste Management of Carolinas, Inc.

SHEET 4a OF 21

DRAWING NO 4a



LEGEND

- EXISTING MAJOR CONTOUR ELEVATION
- EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
- EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
- EXISTING APPROXIMATE MINOR ELEVATION
- EXISTING FENCE
- EXISTING TREE
- EXISTING UTILITY POLE
- MONUMENT
- WATER LINE
- N 890500 STATE PLANE COORDINATE
- PROPERTY LINE
- COUNTY LINE
- RIGHT OF WAY
- LIMIT OF WASTE
- EXISTING UNPAVED ROAD
- EXISTING PAVED ROAD
- PERIMETER ACCESS ROAD
- 850 FINAL CONTOUR ELEVATION
- 800 INTERMEDIATE CONTOUR ELEVATION
- 700 TOP OF PROTECTIVE COVER ELEVATION
- SLOPE BREAK
- D LEACHATE COLLECTION SUMP
- TEMPORARY DIVIDER BERM
- INTERMEDIATE COVER AREA
- LEACHATE COLLECTION PIPE TRENCH
- A/7 CROSS SECTION

NOTES:

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6. THE INTERIOR SIDEWALL AND INTERCELL BERM SLOPES ARE 3 (HORIZ.) : 1 (VERT.)
7. SEE SHEET 6, SEDIMENT AND EROSION CONTROL PLAN AND DETAIL SHEET 14 FOR SURFACE WATER MANAGEMENT INFORMATION.
8. SEE SHEET 21 FOR STOCKPILE LOCATIONS.
9. ALL LEACHATE COLLECTION LINES SHALL HAVE A MINIMUM SLOPE OF 1 PERCENT.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISED TOP OF DRAINAGE CONTOURS	SAS	CML
0	11/93	ISSUE FOR PERMIT		

PROJECT NO	87221	
DATE	NOVEMBER 1993	
DES BY	TMD	11/29/93
DRN BY	RBH	11/29/93
CHK BY	JKR	11/29/93
APP BY	CML	11/30/93

TRANSITION APPLICATION AND PERMIT RENEWAL
PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

LEACHATE MANAGEMENT PLAN (TOP OF PROTECTIVE COVER)

PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE

NORTH CAROLINA PROFESSIONAL SEAL 17773 ENGINEER 4/13/94

Waste Management of Carolinas, Inc.

SHEET 4 OF 21
DRAWING NO 4

7221NCR4.DGN

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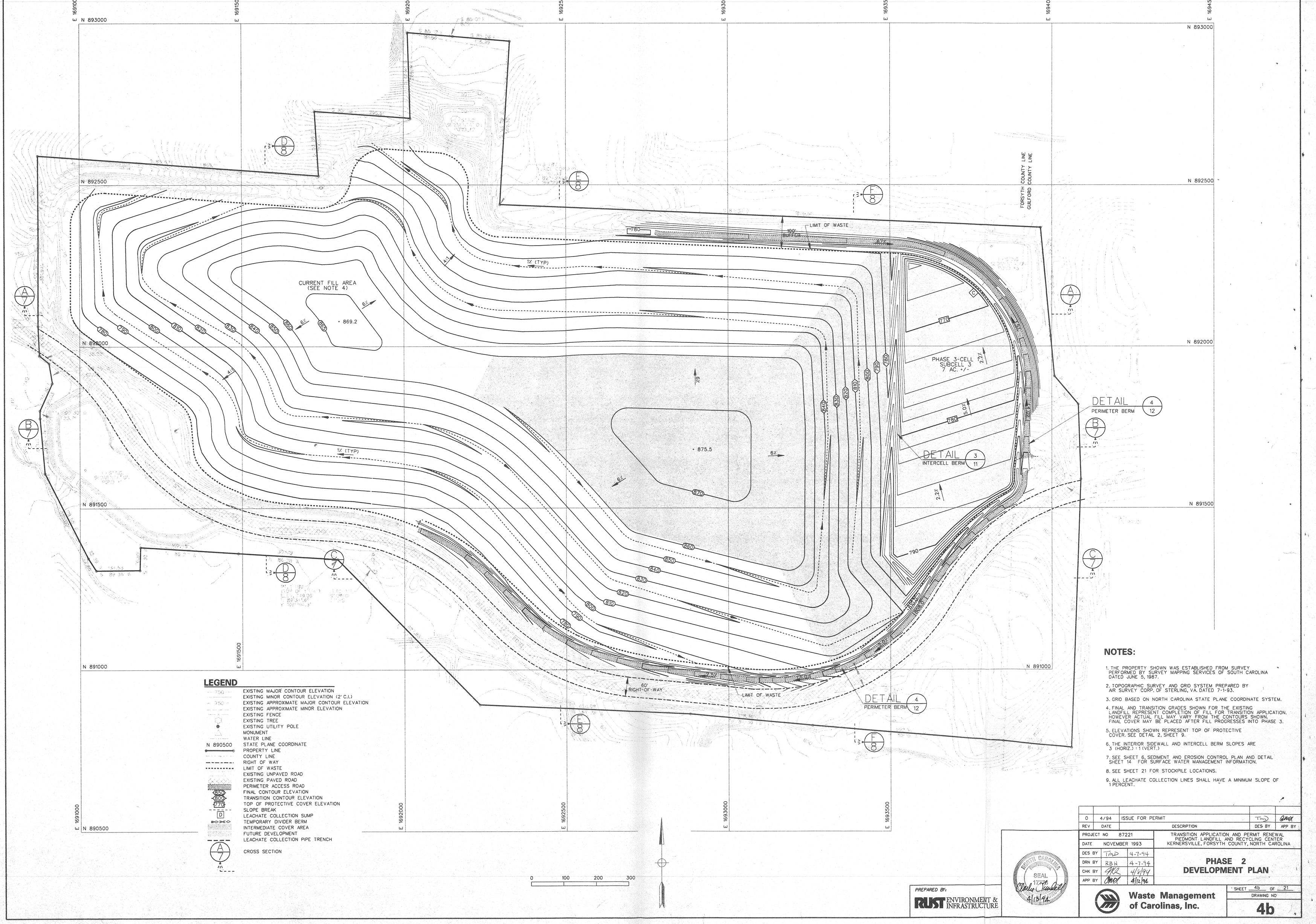

LEGEND

- EXISTING MAJOR CONTOUR ELEVATION
- EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
- EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
- EXISTING APPROXIMATE MINOR ELEVATION
- EXISTING FENCE
- EXISTING TREE
- EXISTING UTILITY POLE
- MONUMENT
- WATER LINE
- N 890500 STATE PLANE COORDINATE
- PROPERTY LINE
- COUNTY LINE
- RIGHT OF WAY
- LIMIT OF WASTE
- EXISTING UNPAVED ROAD
- EXISTING PAVED ROAD
- PERIMETER ACCESS ROAD
- 850 FINAL CONTOUR ELEVATION
- 800 TRANSITION CONTOUR ELEVATION
- 770 TOP OF PROTECTIVE COVER ELEVATION
- SLOPE BREAK
- D LEACHATE COLLECTION SUMP
- TEMPORARY DIVIDER BERM
- INTERMEDIATE COVER AREA
- FUTURE DEVELOPMENT
- LEACHATE COLLECTION PIPE TRENCH
- A/7 CROSS SECTION

NOTES:

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7. SEE SHEET 6, SEDIMENT AND EROSION CONTROL PLAN AND DETAIL SHEET 14 FOR SURFACE WATER MANAGEMENT INFORMATION.
8. SEE SHEET 21 FOR STOCKPILE LOCATIONS.
9. ALL LEACHATE COLLECTION LINES SHALL HAVE A MINIMUM SLOPE OF 1 PERCENT.

REV	DATE	DESCRIPTION	DES BY	APP BY
0	4/94	ISSUE FOR PERMIT	TMD	[illegible]

PROJECT NO	87221	
DATE	NOVEMBER 1993	
DES BY	TMD	4-7-94
DRN BY	RBH	4-7-94
CHK BY	[illegible]	4/8/94
APP BY	[illegible]	4/12/94

TRANSITION APPLICATION AND PERMIT RENEWAL
PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

PHASE 2 DEVELOPMENT PLAN

PREPARED BY:
RUST ENVIRONMENT & INFRASTRUCTURE

NORTH CAROLINA PROFESSIONAL SEAL 17774 4/13/94

Waste Management of Carolinas, Inc.

SHEET 4b OF 21
DRAWING NO
4b

Fri Apr 8 17:19:00 1994 \PLOT\QUEUE\MBOND.qcf N:\MBOND\7221NC4B.PRF

E 1691000 | E 1691500 | E 1692000 | E 1692500 | E 1693000 | E 1693500 | E 1694000 | E 1694500

N 893000 | N 892500 | N 892000 | N 891500 | N 891000 | N 890500

FORSYTH COUNTY LINE
GUILFORD COUNTY LINE

100' BUFFER
LIMIT OF WASTE
60' RIGHT-OF-WAY
LIMIT OF WASTE

ACCESS ROAD

PERMANENT LANDFILL ACCESS ROAD — DETAIL 2/13

DETAIL — DRAINAGE TERRACE 3/14

DETAIL — LANDFILL BENCH OUTFALL DETAIL 4/14

DETAIL — PERIMETER BERM 4/12

•869.2
•863.8
•899.5

LEGEND

- EXISTING MAJOR CONTOUR ELEVATION
- EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
- EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
- EXISTING APPROXIMATE MINOR ELEVATION
- EXISTING FENCE
- EXISTING TREE
- EXISTING UTILITY POLE
- MONUMENT
- WATER LINE
- N 890500 STATE PLANE COORDINATE
- PROPERTY LINE
- COUNTY LINE
- RIGHT OF WAY
- LIMIT OF WASTE
- EXISTING UNPAVED ROAD
- EXISTING PAVED ROAD
- ACCESS ROAD
- 850 FINAL CONTOUR ELEVATION
- DRAINAGE FLOW DIRECTION
- A/7 CROSS SECTION

0 100 200 300

NOTES:

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3. GRID BASED ON NORTH CAROLINA STATE PLANE COORDINATE SYSTEM.
4. CONTOUR ELEVATIONS SHOWN REPRESENT TOP OF FINAL COVER SEE DETAIL 3 SHEET 9.
5. LANDFILL BENCHES SLOPE TOWARD THE LANDFILL AT A MINIMUM OF 10 PERCENT.
6. SEE SHEET 6 FOR EROSION CONTROL PLAN AND DETAILS SHEET 14 FOR SURFACE WATER MANAGEMENT INFORMATION.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISED FINAL CONTOURS	TMD	
0	11/93	ISSUE FOR PERMIT		

PROJECT NO 87221
DATE NOVEMBER 1993
DES BY TMD 11/29/93
DRN BY RBH 11/29/93
CHK BY JKR 11/29/93
APP BY CML 11/29/93

TRANSITION APPLICATION AND PERMIT RENEWAL
PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

FINAL DEVELOPMENT PLAN

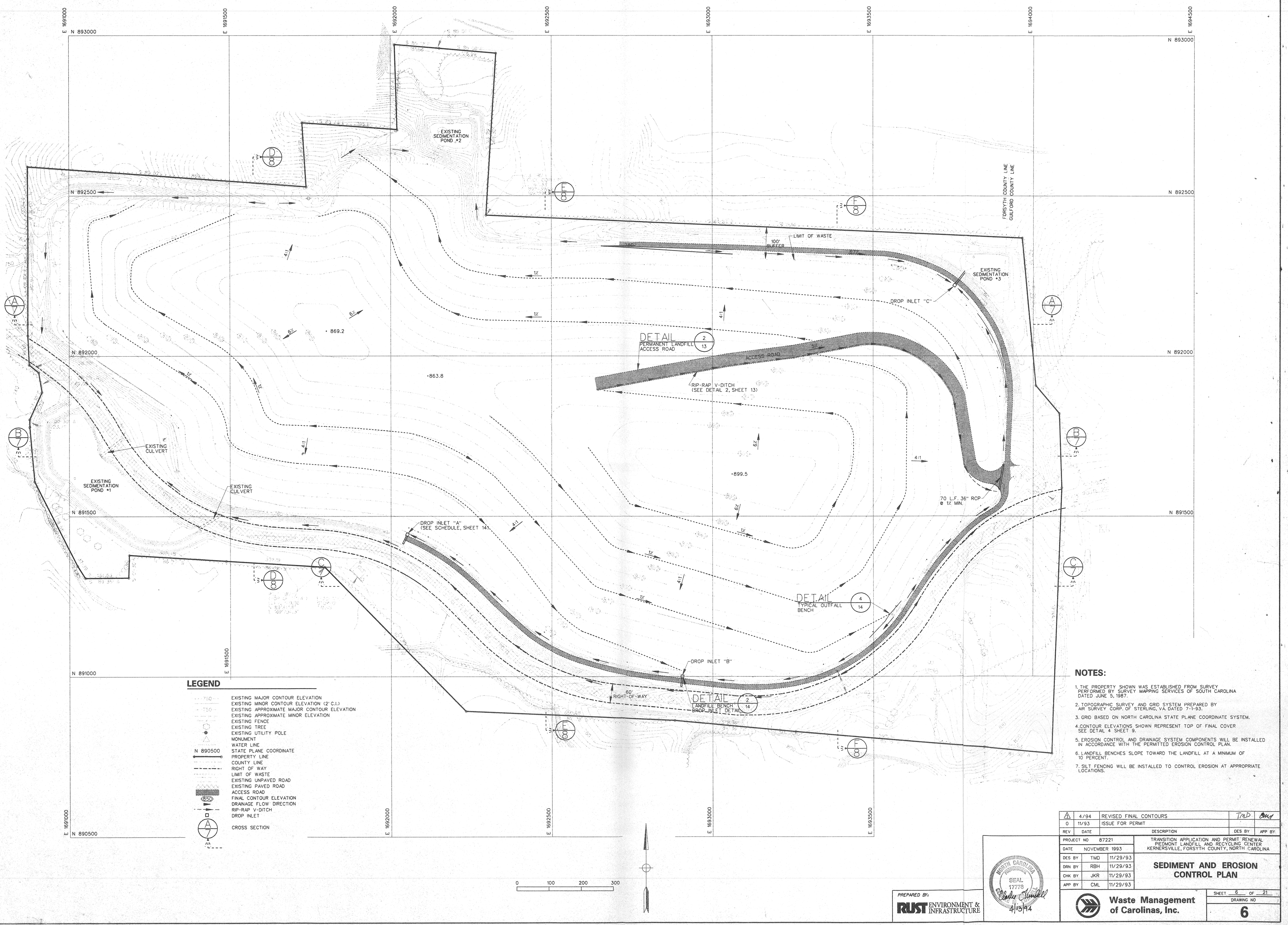
SEAL 17778 — 4/13/94

PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE

Waste Management of Carolinas, Inc.

SHEET 5 OF 21
DRAWING NO 5

\PLOT\QUEUE\MBOND.722INC05.PRF
Apr 8 17:33:05 1994 N:\MBOND\



LEGEND

- EXISTING MAJOR CONTOUR ELEVATION
- EXISTING MINOR CONTOUR ELEVATION (2' C.I.)
- EXISTING APPROXIMATE MAJOR CONTOUR ELEVATION
- EXISTING APPROXIMATE MINOR ELEVATION
- EXISTING FENCE
- EXISTING TREE
- EXISTING UTILITY POLE
- MONUMENT
- WATER LINE
- STATE PLANE COORDINATE
- PROPERTY LINE
- COUNTY LINE
- RIGHT OF WAY
- LIMIT OF WAY
- EXISTING UNPAVED ROAD
- EXISTING PAVED ROAD
- ACCESS ROAD
- FINAL CONTOUR ELEVATION
- DRAINAGE FLOW DIRECTION
- RIP-RAP V-DITCH
- DROP INLET
- CROSS SECTION

NOTES:

1. THE PROPERTY SHOWN WAS ESTABLISHED FROM SURVEY PERFORMED BY SURVEY MAPPING SERVICES OF SOUTH CAROLINA DATED JUNE 5, 1987.
2. TOPOGRAPHIC SURVEY AND GRID SYSTEM PREPARED BY AIR SURVEY CORP. OF STERLING, VA. DATED 7-1-93.
3. GRID BASED ON NORTH CAROLINA STATE PLANE COORDINATE SYSTEM.
4. CONTOUR ELEVATIONS SHOWN REPRESENT TOP OF FINAL COVER SEE DETAIL 4 SHEET 9.
5. EROSION CONTROL AND DRAINAGE SYSTEM COMPONENTS WILL BE INSTALLED IN ACCORDANCE WITH THE PERMITTED EROSION CONTROL PLAN.
6. LANDFILL BENCHES SLOPE TOWARD THE LANDFILL AT A MINIMUM OF 10 PERCENT.
7. SILT FENCING WILL BE INSTALLED TO CONTROL EROSION AT APPROPRIATE LOCATIONS.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISED FINAL CONTOURS	TMD	BMG
0	11/93	ISSUE FOR PERMIT		

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	TMD 11/29/93	SEDIMENT AND EROSION CONTROL PLAN
DRN BY	RBH 11/29/93	
CHK BY	JKR 11/29/93	
APP BY	CML 11/29/93	

NORTH CAROLINA PROFESSIONAL ENGINEER SEAL 17778 Charles Lunsford 4/13/94

PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE

Waste Management of Carolinas, Inc.

SHEET 6 OF 21
DRAWING NO 6

4/8/94 17:44:40 J:\87221\DWG\GROUND\87221NC06.PRE

PREPARED BY:

RUST ENVIRONMENT & INFRASTRUCTURE

4/13/94

Waste Management of Carolinas, Inc.

SHEET 7 OF 21 — DRAWING NO **7**

CROSS-SECTIONS A-A THROUGH C-C

APP BY	CHK.	11/29/93
CHK BY	JKR	11/29/93
DRN BY	RBH	11/29/93
DES BY	TMD	11/29/93

PROJECT NO 87221 — DATE NOVEMBER 1993

TRANSITION APPLICATION
PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

REV	DATE	DESCRIPTION	DES BY	APP BY
0	11/93	ISSUE FOR PERMIT	GRE	DMK
1	4/94	REVISED BASEGRADES		

NOTE:
1. BASE LINER SYSTEM IS SHOWN IN NEW AREA ONLY. EXISTING BASE LINER SYSTEM IS NOT SHOWN.
2. TOP OF COMPETENT BEDROCK NOT SHOWN IN AREAS DROPPING BELOW LIMIT OF SECTION.

KEY MAP
NO SCALE

HORIZONTAL SCALE
0 100 200 300

VERTICAL SCALE
0 20 40 60

SECTION "C"

E 1692 000 E 1692 500 E 1693 000 E 1693 500 E 1694 000

E.O.B. EL. 668.84 — P8S,P8A,P8D (PROJ.)
E.O.B. EL. 751.40 — P2S,P2D (PROJ.)

0-00 1-00 2-00 3-00 4-00 5-00 6-00 7-00 8-00 9-00 10-00 11-00 12-00 13-00 14-00 15-00 16-00 17-00 18-00 19-00 20-00 21-00 22-00 23-00 24-00 25-00

740 760 780 800 820 840 860 880

WESTERN PROPERTY BOUNDARY
LINER SYSTEM (TYPICAL) 9/2
SEASONAL HIGH GROUND WATER
INTERCELL BERM (TYP.) 11/3
TOP OF PROTECTIVE COVER
EXISTING GRADE
EASTERN PROPERTY BOUNDARY
EXISTING GRADE
PROPOSED FINAL COVER 9/3
1 4

SECTION "B"

E 1691 500 E 1692 000 E 1692 500 E 1693 000 E 1693 500 E 1694 000

E.O.B. EL. 752.47 — P1S,P1D,P1DD

0-00 1-00 2-00 3-00 4-00 5-00 6-00 7-00 8-00 9-00 10-00 11-00 12-00 13-00 14-00 15-00 16-00 17-00 18-00 19-00 20-00 21-00 22-00 23-00 24-00 25-00 26-00 27-00 28-00 29-00 30-00 31-00 32-00 33-00 34-00

740 760 780 800 820 840 860 880 900

WESTERN PROPERTY BOUNDARY
SEASONAL HIGH GROUND WATER
TOP OF COMPETENT BEDROCK
PERIMETER BERM/LAYOUT 4/12
LINER SYSTEM (TYPICAL) 9/2
TOP OF PROTECTIVE COVER
INTERCELL BERM (TYP.) 11/3
EXISTING GRADE
CURRENT FILL AREAS
PROPOSED DEVELOPMENT AREAS
INTERMEDIATE COVER
EXISTING WASTE GRADE
EXISTING WASTE FILL AREA
EASTERN PROPERTY BOUNDARY
DRAINAGE TERRACE (TYP.) 14/3
PROPOSED FINAL COVER 9/3
3 1 4

SECTION "A"

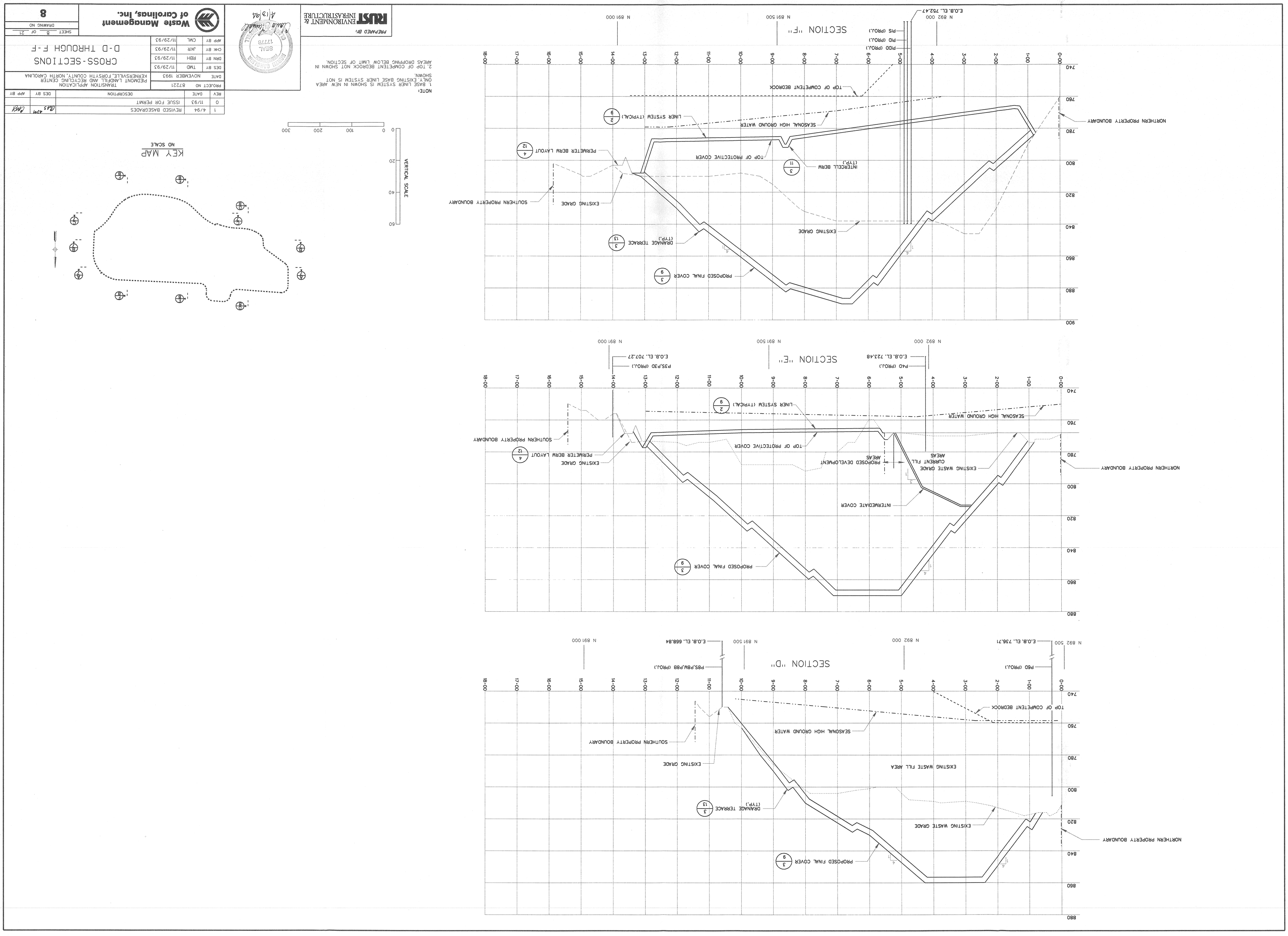
E 1691 000 E 1691 500 E 1692 000 E 1693 000 E 1693 500 E 1694 000

E.O.B. EL. 726.79 — P7S,P7D (PROJ.)
E.O.B. EL. 723.48 — P4D (PROJ.)
E.O.B. EL. 719.96 — 7055,7050 (PROJ.)

0-00 1-00 2-00 3-00 4-00 5-00 6-00 7-00 8-00 9-00 10-00 11-00 12-00 13-00 14-00 15-00 16-00 17-00 18-00 19-00 20-00 21-00 22-00 23-00 24-00 25-00 26-00 27-00 28-00 29-00 30-00 31-00 32-00 33-00 34-00

740 760 780 800 820 840 860 880

WESTERN PROPERTY BOUNDARY
SEASONAL HIGH GROUND WATER
LINER SYSTEM (TYPICAL) 9/2
TOP OF PROTECTIVE COVER
INTERCELL BERM (TYP.) 11/3
PROPOSED DEVELOPMENT AREAS
CURRENT FILL AREAS
EXISTING WASTE GRADE
EXISTING WASTE FILL AREA
EASTERN PROPERTY BOUNDARY
EXISTING GRADE
INTERMEDIATE COVER
PROPOSED FINAL COVER 9/3
DRAINAGE TERRACE (TYP.) 14/3
3 1 4



NOTE:
1. BASE LINER SYSTEM IS SHOWN IN NEW AREA ONLY. EXISTING BASE LINER SYSTEM IS NOT SHOWN.
2. TOP OF COMPETENT BEDROCK NOT SHOWN IN AREAS DROPPING BELOW LIMIT OF SECTION.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISED BASEGRADES		
0	11/93	ISSUE FOR PERMIT		

PROJECT NO	87221	TRANSITION APPLICATION
DATE	NOVEMBER 1993	PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DES BY	TMD	11/29/93
DRN BY	RBH	11/29/93
CHK BY	JKR	11/29/93
APP BY	CML	11/29/93

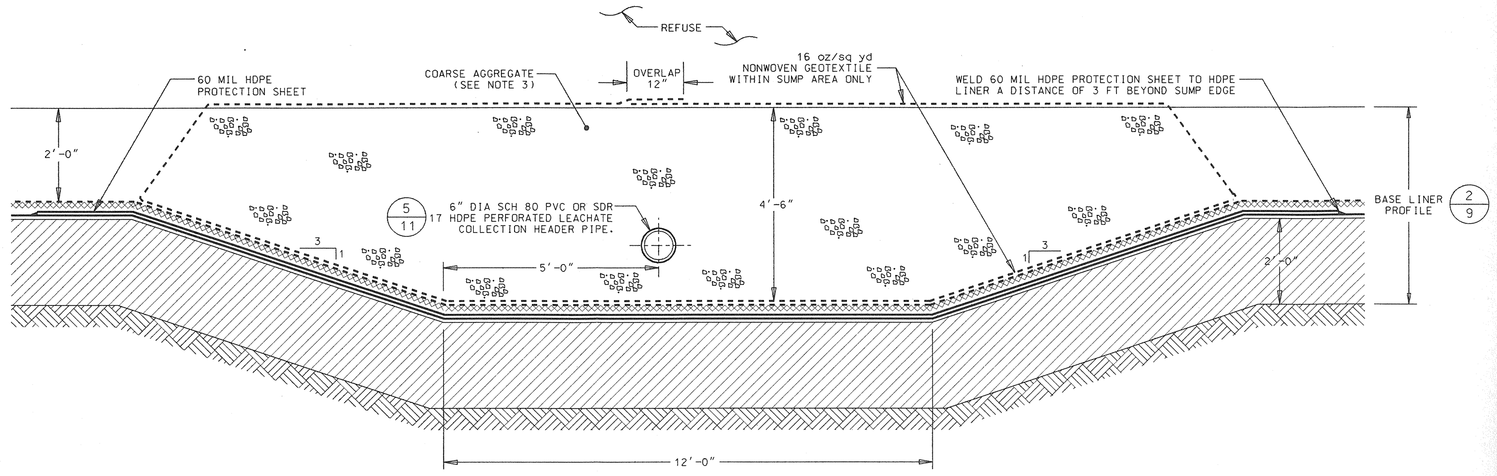
CROSS-SECTIONS
D-D THROUGH F-F

PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE

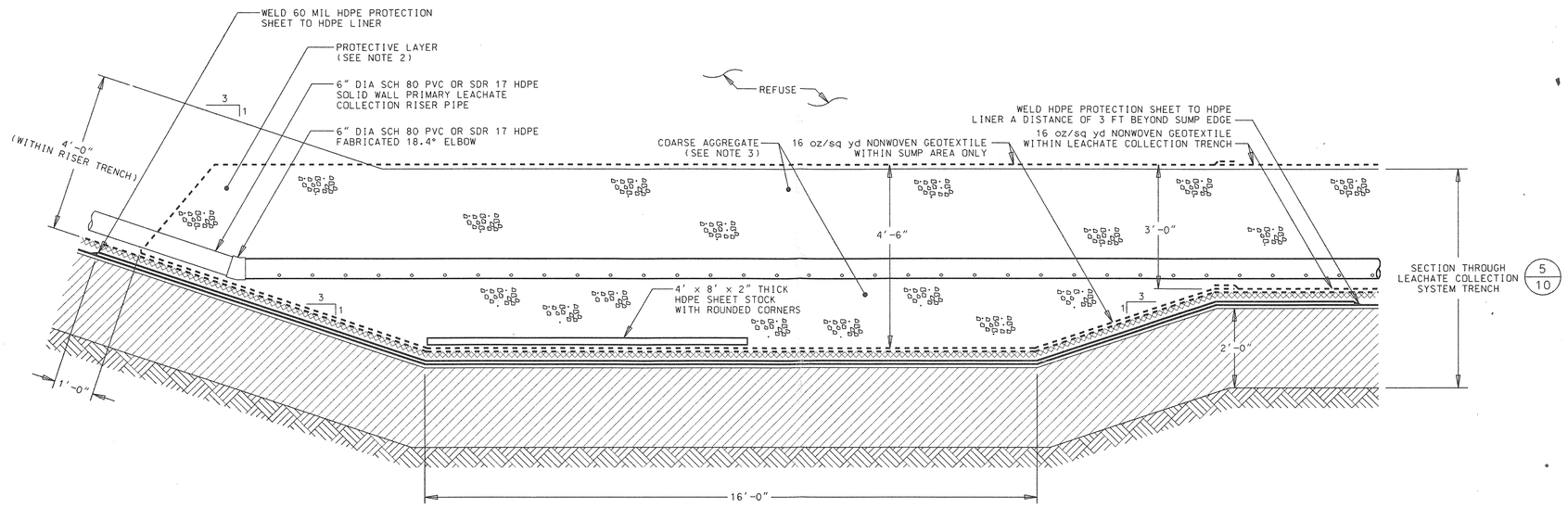
Waste Management of Carolinas, Inc.

SHEET 8 OF 21

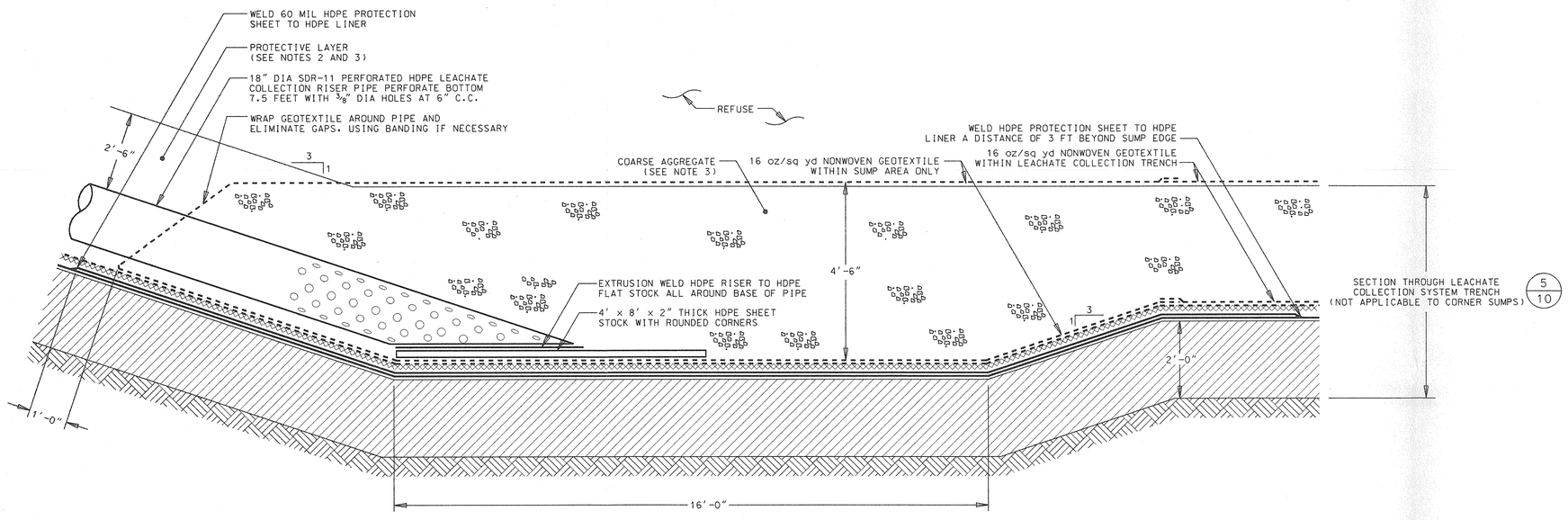
DRAWING NO 8



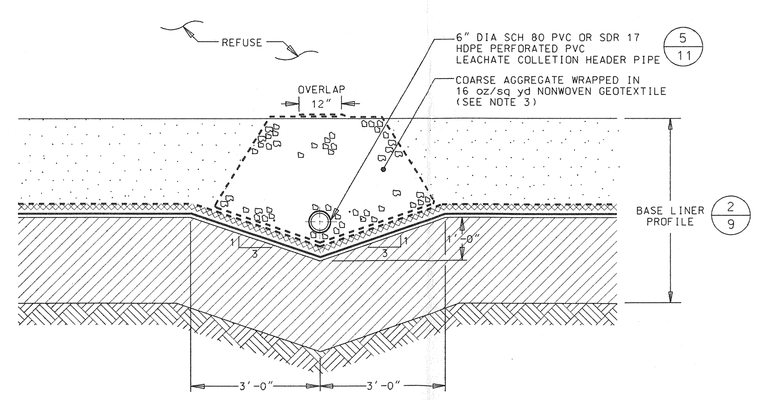
1/10 SUMP AREA CROSS SECTION A – A' ⚠



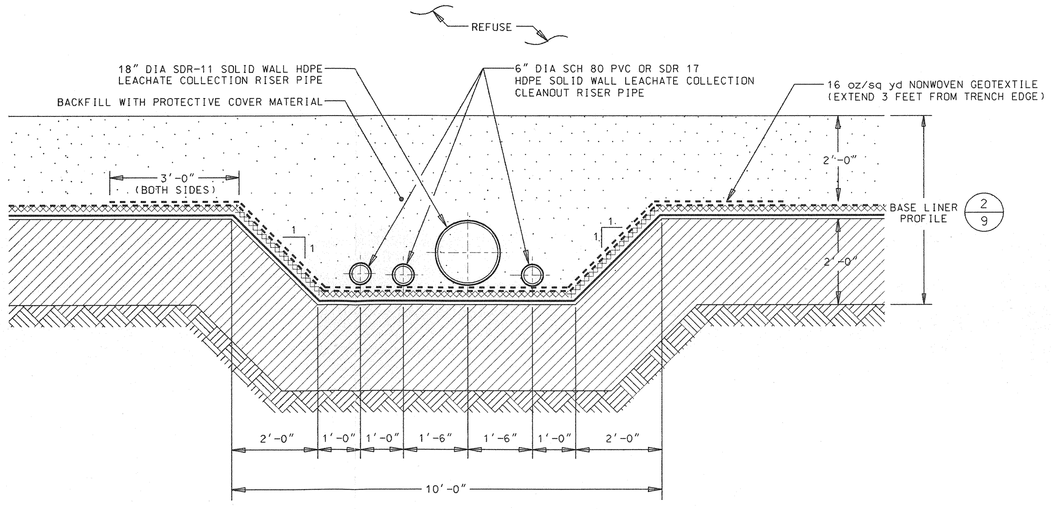
4/10 LEACHATE COLLECTION SYSTEM CLEANOUT PIPE CROSS SECTION D–D' ⚠



2/10 LEACHATE COLLECTION SUMP AND RISER PIPE CROSS SECTION B – B' ⚠



5/10 LEACHATE COLLECTION SYSTEM TRENCH CROSS SECTION E–E' ⚠



3/10 LEACHATE COLLECTION SUMP RISER TRENCH CROSS SECTION C – C'

NOTE: LEACHATE COLLECTION CLEANOUT RISER PIPE QUANTITIES AND LOCATIONS MAY VARY DEPENDENT ON SUMP LOCATION

NOTES

1. THE VERTICAL DIMENSIONS INDICATED ARE NOT TO SCALE DUE TO THE GRAPHICAL REPRESENTATION OF THE LINER COMPONENTS.
2. THE PROTECTIVE LAYER ON THE BASE OF THE LANDFILL WILL CONSIST OF AN EARTHFILL HAVING ROUNDED TO SUBROUNDED GRAINS WITH A MAXIMUM PARTICLE SIZE OF 1 INCH AND A MINIMUM HYDRAULIC CONDUCTIVITY OF 1×10^{-4} cm/sec. THE MINIMUM HYDRAULIC CONDUCTIVITY REQUIREMENT OF THE PROTECTIVE LAYER MAY BE REDUCED ON THE LANDFILL SIDEWALL AT MINIMUM ELEVATIONS FIVE FEET ABOVE THE SIDEWALL TOE, AS MEASURED FROM THE BASE GRADE. HOWEVER, THE MAXIMUM PARTICLE SIZE REQUIREMENT WILL STILL APPLY.
3. COARSE AGGREGATE WILL HAVE A MAXIMUM PARTICLE SIZE OF 1-1/2 INCHES, WITH LESS THAN 5% SMALLER THAN ⅜ INCH.
4. ALL DIMENSIONS INDICATED REPRESENT MINIMUM VALUES UNLESS OTHERWISE SPECIFIED.

REV	DATE	DESCRIPTION	DES BY	APP BY
⚠	4/94	RESPONSE TO NCDEHNR COMMENTS		
0	11/93	ISSUE FOR PERMIT		

ALL DRAWINGS ARE NOT TO SCALE EXCEPT WHERE NOTED.

PREPARED BY:
RUST ENVIRONMENT & INFRASTRUCTURE
SOLID WASTE DIVISION – NAPERVILLE

NORTH CAROLINA PROFESSIONAL ENGINEER SEAL 17778 — 4/15/94

PROJECT NO	87221	
DATE	NOVEMBER 1993	
DES BY	JMC	11-29-94
DRN BY	PAA	11-29-94
CHK BY	AJM	11-29-94
APP BY	CML	11-29-94

TRANSITION APPLICATION AND PERMIT RENEWAL
PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS

Waste Management of Carolinas, Inc.

SHEET 10 OF 21
DRAWING NO **10**

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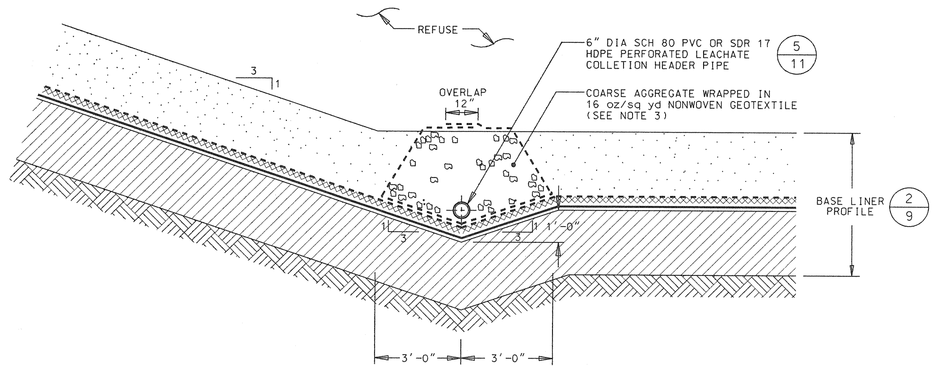
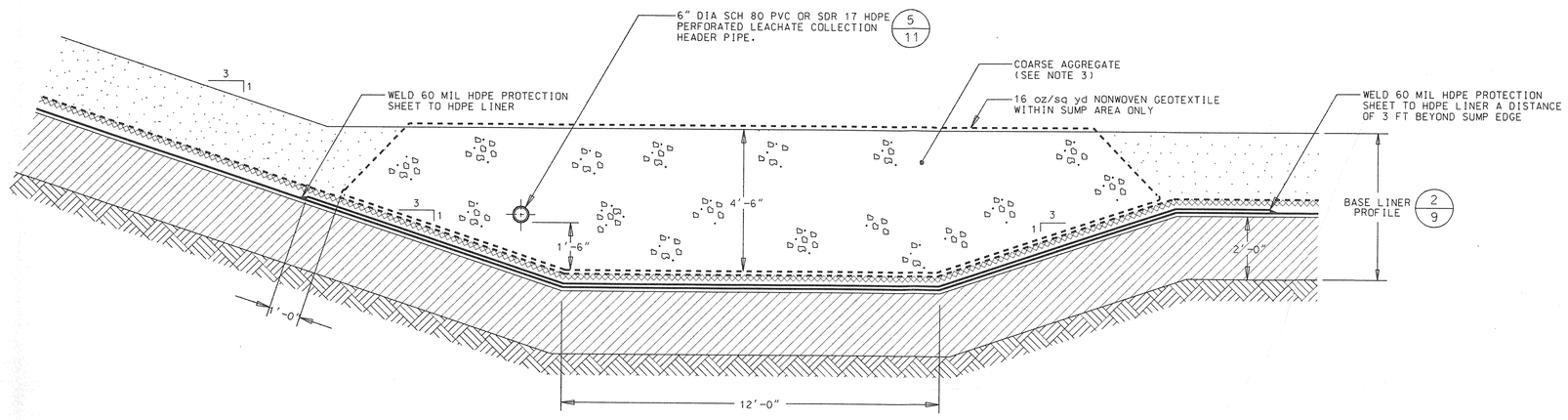
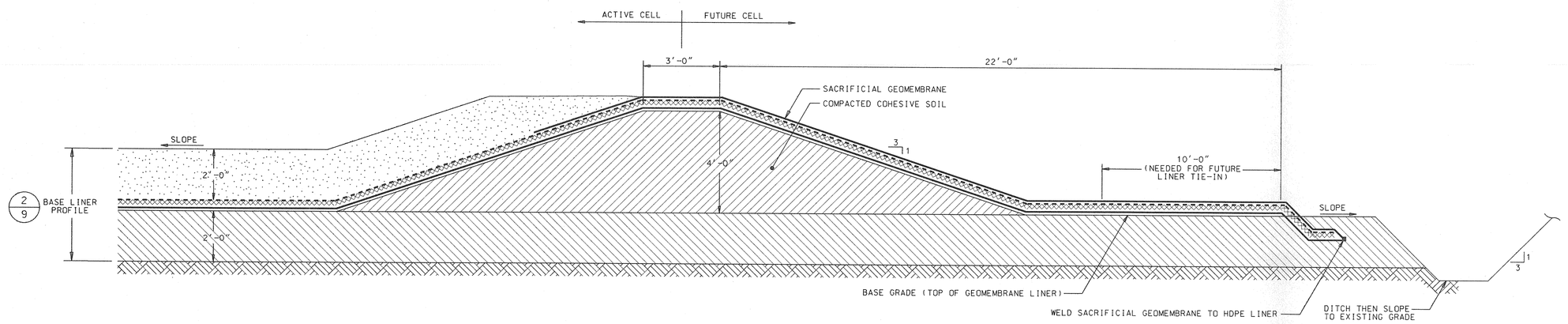
PLOT DATE: 12-APR-1994 15:27



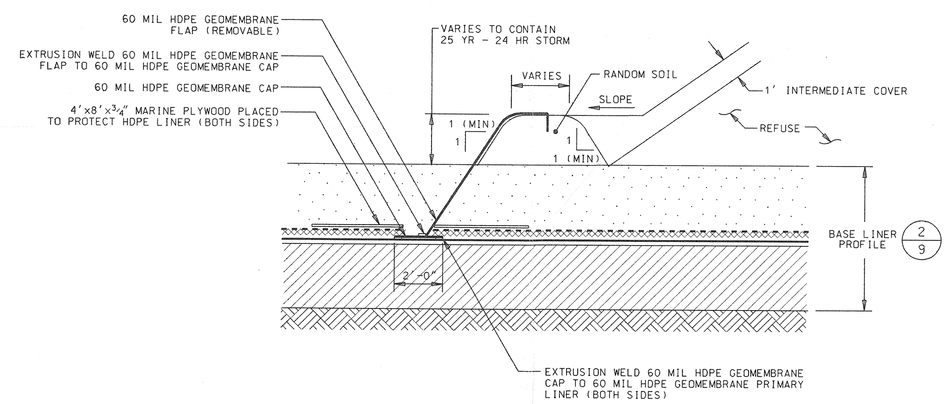
1/11 LEACHATE COLLECTION SYSTEM TRENCH CROSS SECTION F-F' ⚠1



2/11 SUMP AREA CROSS SECTION G - G' ⚠1



3/11 INTERCELL BERM

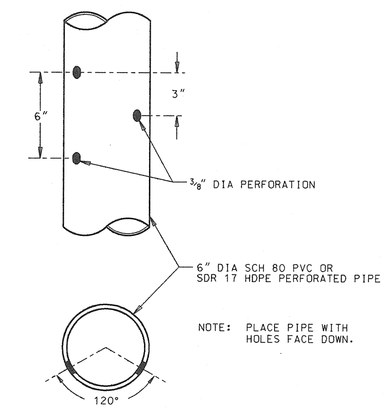


BERM REMOVAL SEQUENCE

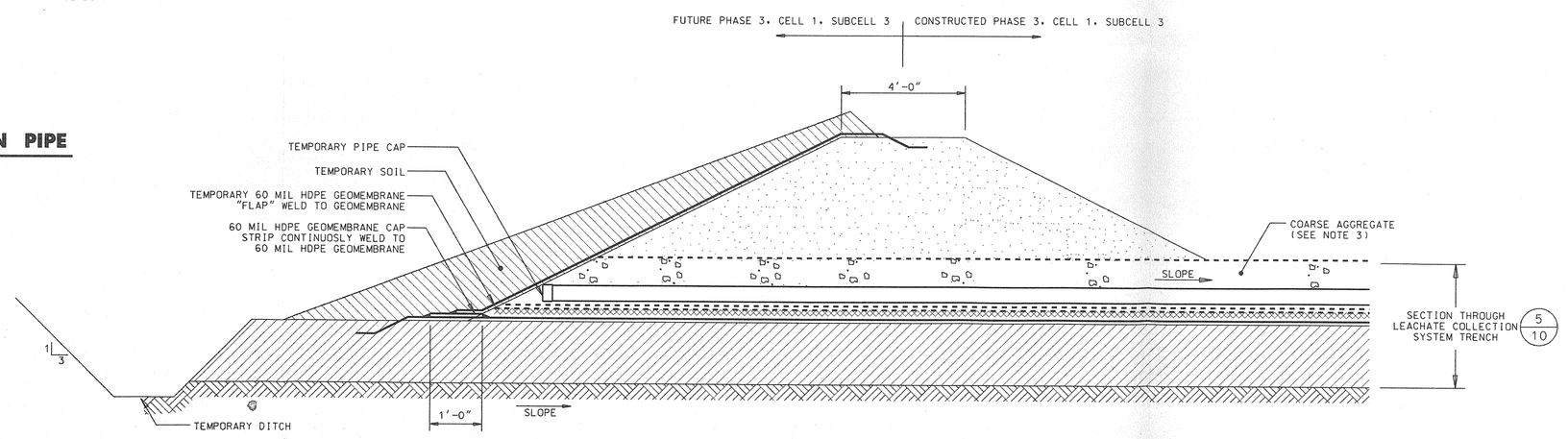
1. REMOVE TEMPORARY RANDOM SOIL BERM.
2. REMOVE PROTECTIVE LAYER AND PROTECTIVE PLYWOOD.
3. TRIM GEOMEMBRANE FLAP WITHIN 2 INCHES OF WELD. LAY REMAINING FLAP ON LINER.
4. TIE GEONET LAYERS TOGETHER.
5. SEW NONWOVEN GEOTEXTILE LAYERS TOGETHER.
6. REPLACE PROTECTIVE LAYER.

4/11 TEMPORARY STORMWATER FLAP/BERM

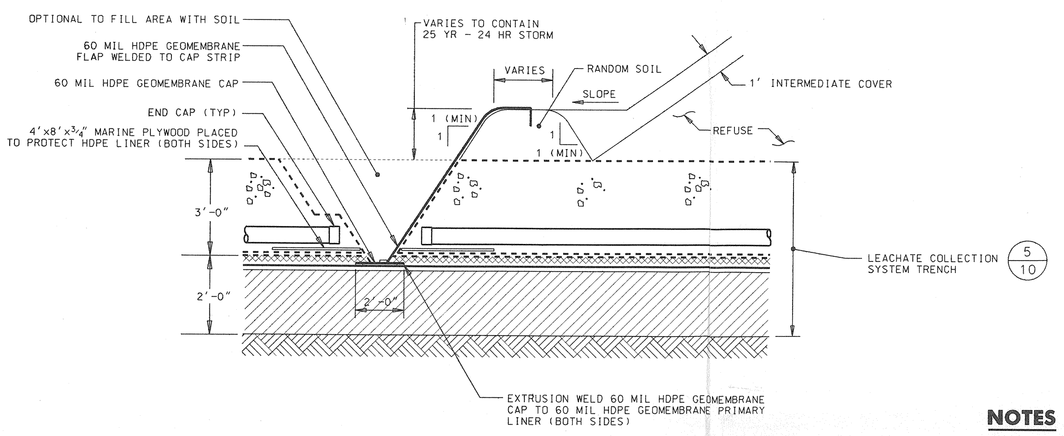
NOTE: BERM CONFIGURATIONS MAY VARY FROM THAT SHOWN. HOWEVER. THE GEOSYNTHETIC COMPONENTS WILL BE INCORPORATED



5/11 LEACHATE COLLECTION PIPE



6/11 TEMPORARY DIVIDER BERM AT PHASE 3, CELL 1, SUBCELL 3 ⚠1



7/11 TEMPORARY STORMWATER FLAP/BERM AT LEACHATE COLLECTION TRENCH ⚠1

NOTES

1. THE VERTICAL DIMENSIONS INDICATED ARE NOT TO SCALE DUE TO THE GRAPHICAL REPRESENTATION OF THE LINER COMPONENTS.
2. THE PROTECTIVE LAYER ON THE BASE OF THE LANDFILL WILL CONSIST OF AN EARTHFILL HAVING ROUNDED TO SUBROUNDED GRAINS WITH A MAXIMUM PARTICLE SIZE OF 1 INCH AND A MINIMUM HYDRAULIC CONDUCTIVITY OF 1×10^{-4} cm/sec. THE MINIMUM HYDRAULIC CONDUCTIVITY REQUIREMENT OF THE PROTECTIVE LAYER MAY BE REDUCED ON THE LANDFILL SIDEWALL AT MINIMUM ELEVATIONS FIVE FEET ABOVE THE SIDEWALL TOE. AS MEASURED FROM THE BASE GRADE. HOWEVER. THE MAXIMUM PARTICLE SIZE REQUIREMENT WILL STILL APPLY.
3. COARSE AGGREGATE WILL HAVE A MAXIMUM PARTICLE SIZE OF 1-1/2 INCHES. WITH LESS THAN 5% SMALLER THAN 3/8 INCH.
4. ALL DIMENSIONS INDICATED REPRESENT MINIMUM VALUES UNLESS OTHERWISE SPECIFIED.

⚠1	4/94	RESPONSE TO NCDEHNR COMMENTS	afm	CML
0	11/93	ISSUE FOR PERMIT		
REV	DATE	DESCRIPTION	DES BY	APP BY

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	JMC 11-29-93	BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS
DRN BY	PAA 11-29-93	
CHK BY	AJM 11-29-93	
APP BY	CML 11-29-93	

ALL DRAWINGS ARE NOT TO SCALE EXCEPT WHERE NOTED.

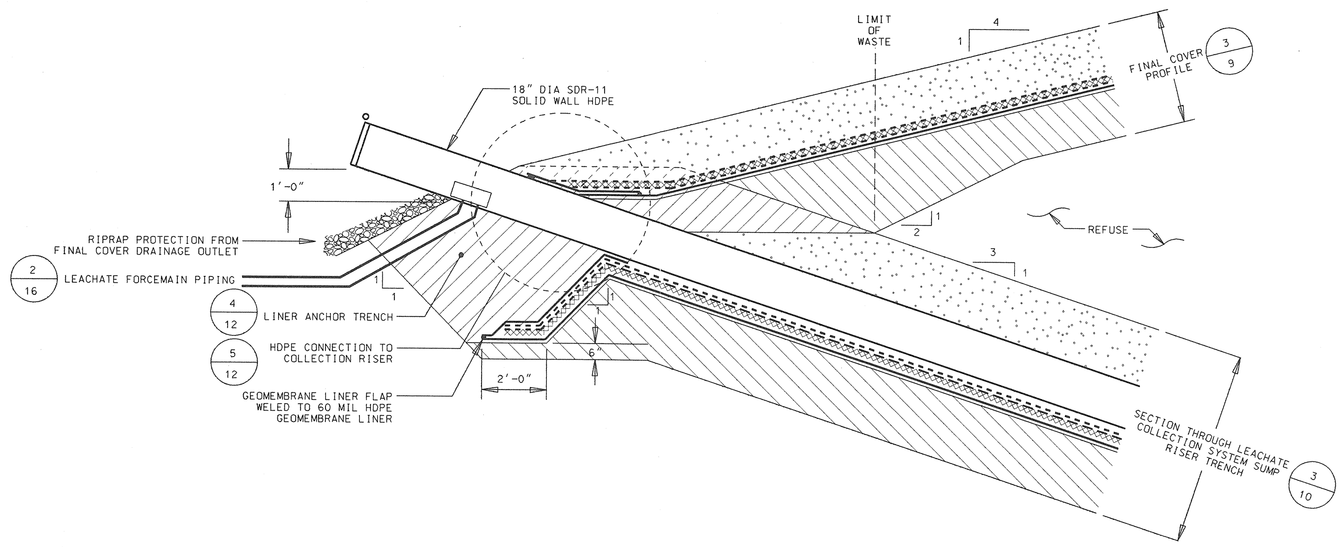
PREPARED BY: RUST ENVIRONMENT & INFRASTRUCTURE, SOLID WASTE DIVISION - NAPERVILLE

NORTH CAROLINA REGISTERED PROFESSIONAL ENGINEER SEAL 11772 — 4/13/94

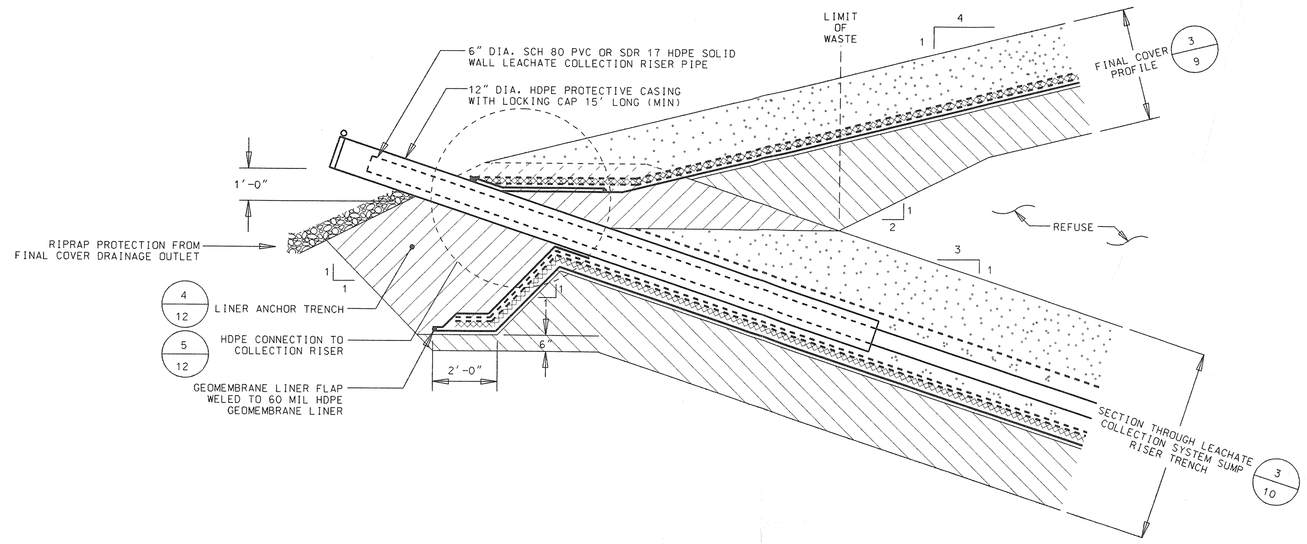
Waste Management of Carolinas, Inc.

SHEET 11 OF 21 — DRAWING NO 11

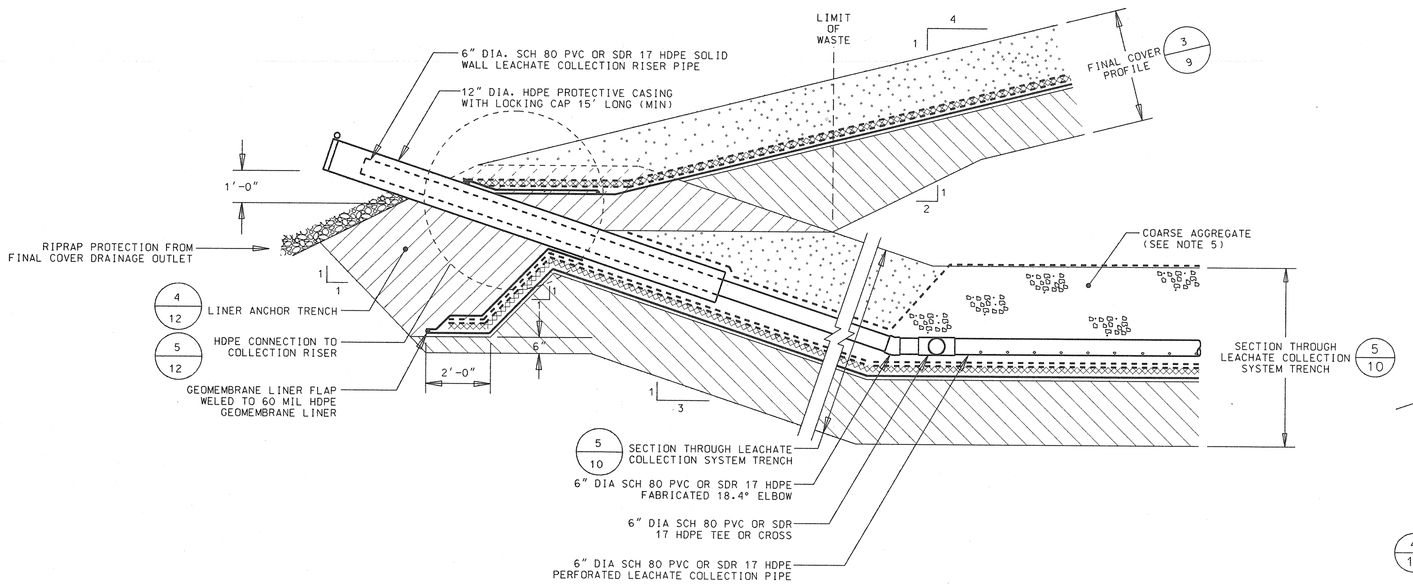
FILE NAME: C:\pdf\pdf\87221\5ask21.dgn
PLOT DATE: 04/19/94 10:03:00



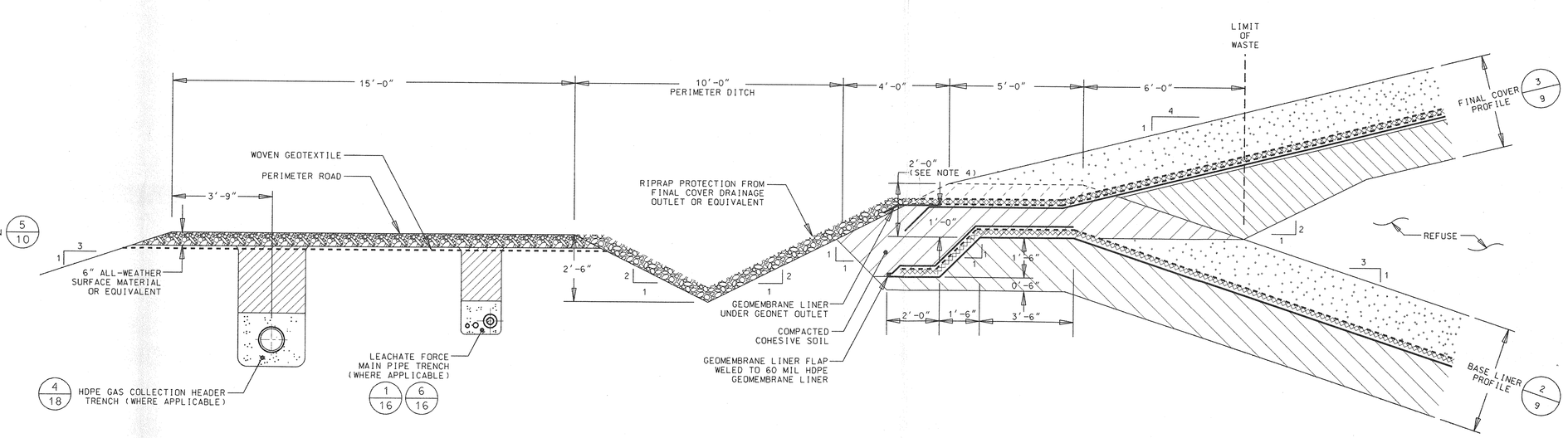
1/12 LEACHATE COLLECTION SUMP RISER



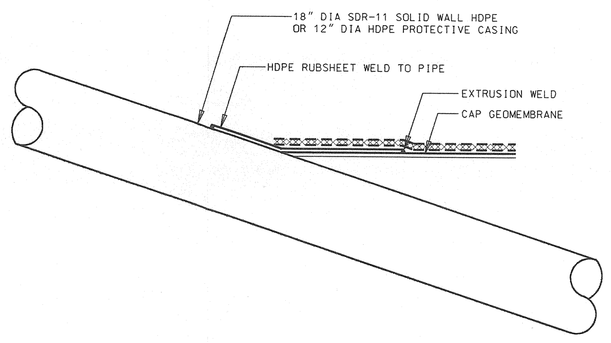
2/12 LEACHATE COLLECTION CLEANOUT RISER AT COLLECTION SUMP



3/12 LEACHATE COLLECTION CLEANOUT RISER ⚠



4/12 ANCHOR TRENCH / PERIMETER BERM



5/12 HDPE CONNECTION TO COLLECTION RISER

NOTES

1. THE VERTICAL DIMENSIONS INDICATED ARE NOT TO SCALE DUE TO THE GRAPHICAL REPRESENTATION OF THE LINER COMPONENTS.
2. THE PROTECTIVE LAYER ON THE BASE OF THE LANDFILL WILL CONSIST OF AN EARTHFILL HAVING ROUNDED TO SUBROUNDED GRAINS WITH A MAXIMUM PARTICLE SIZE OF 1 INCH AND A MINIMUM HYDRAULIC CONDUCTIVITY OF 1×10^{-4} cm/sec. THE MINIMUM HYDRAULIC CONDUCTIVITY REQUIREMENT OF THE PROTECTIVE LAYER MAY BE REDUCED ON THE LANDFILL SIDEWALL AT MINIMUM ELEVATIONS FIVE FEET ABOVE THE SIDEWALL TOE, AS MEASURED FROM THE BASE GRADE. HOWEVER, THE MAXIMUM PARTICLE SIZE REQUIREMENT WILL STILL APPLY.
3. ALL DIMENSIONS INDICATED REPRESENT MINIMUM VALUES UNLESS OTHERWISE SPECIFIED.
4. PRIOR TO CONSTRUCTION OF THE LANDFILL FINAL COVER, TWO FEET OF COMPACTED COHESIVE SOIL WILL BE PLACED ABOVE THE GEOSYNTHETIC COMPONENTS OR THE BASE LINER. THE TWO FEET OF SOIL WILL BE REDUCED TO ONE FOOT UPON CONSTRUCTION OF THE FINAL COVER. THE REMOVED COMPACTED COHESIVE SOILS WILL BE REPLACED WITH FINAL COVER PROTECTIVE SOILS.
5. COARSE AGGREGATE WILL HAVE A MAXIMUM PARTICLE SIZE OF 1-1/2 INCHES, WITH LESS THAN 5% SMALLER THAN $\frac{3}{8}$ INCH.

⚠	4/94	RESPONSE TO NCDEHNR COMMENTS		
0	11/93	ISSUE FOR PERMIT		
REV	DATE	DESCRIPTION	DES BY	APP BY

ALL DRAWINGS ARE NOT TO SCALE EXCEPT WHERE NOTED.

PREPARED BY:
RUST ENVIRONMENT & INFRASTRUCTURE
SOLID WASTE DIVISION — NAPERVILLE

NORTH CAROLINA PROFESSIONAL SEAL 17773 ENGINEER — 4/5/94

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	JMC 11-29-93	BASE LINER AND LEACHATE COLLECTION SYSTEM DETAILS
DRN BY	PAA 11-29-93	
CHK BY	AJM 11-29-93	
APP BY	CML 11-29-93	

Waste Management of Carolinas, Inc.

SHEET 12 OF 21
DRAWING NO 12

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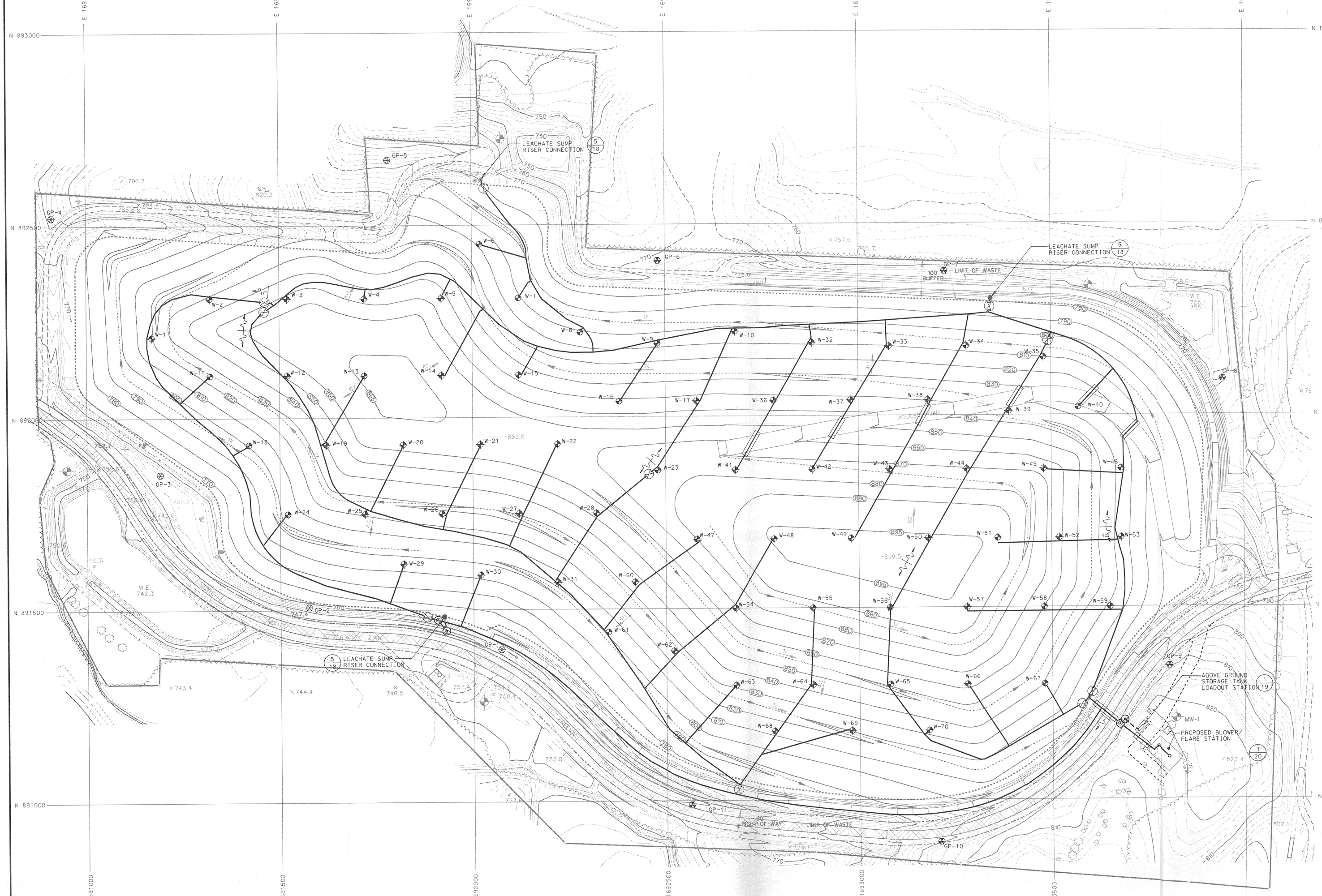
PLOT DATE: 29 MARCH 1994 10:01



LEGEND

- PROPERTY BOUNDARY
- N 11500 — GRID COORDINATE
- 590 — EXISTING 10' CONTOUR
- EXISTING 2' CONTOUR
- EXISTING FENCE LINE
- EXISTING ROAD
- EXISTING BUILDING
- EXISTING TREELINE
- EXISTING BUSH
- × 567.2 — EXISTING SPOT ELEVATION
- 840 — FINAL GRADE CONTOUR
- GP-1 — EXISTING GAS MONITORING PROBE
- GP-7 — PROPOSED GAS MONITORING PROBE
- W-1 — PROPOSED GAS EXTRACTION WELL
- GAS COLLECTION HEADER PIPELINE
- UNDERGROUND CONTROL VALVE
- HEADER ROAD CASING
- CONDENSATE KNOCKOUT
- PUMP STATION
- FORCEMAIN
- HIGH POINT IN HEADER
- 1/20 — DETAIL REFERENCE
- LEACHATE CLEANOUT RISER

NOTES

1. LANDFILL GAS MANAGEMENT SYSTEM ALIGNMENT, LOCATION, DETAILS, AND NUMBER OF EXTRACTION WELLS AND LOCATIONS AND NUMBERS OF PROPOSED GAS MONITORING PROBES MAY CHANGE DUE TO SITE CONDITIONS AT THE TIME OF CONSTRUCTION.
2. DUE TO CURRENT WASTE DISPOSAL OPERATIONS, THE PLAN VIEW TOPOGRAPHIC MAP MAY NOT REFLECT CURRENT EXISTING SITE CONDITIONS. ACTUAL GROUND SURFACE ELEVATIONS ARE TO BE FIELD SURVEY VERIFIED PRIOR TO CONSTRUCTION.
3. LOCATION AND INSTALLATION OF THE PROPOSED GAS EXTRACTION WELLS, HEADER AND LATERAL ALIGNMENT, AND LIQUID MANAGEMENT FACILITIES MAY VARY TO ACCOMMODATE LANDFILL FIELD SLOPE.
4. MINIMUM ACCEPTABLE COLLECTION HEADER AND LATERAL PIPELINE SLOPE IS THREE PERCENT (3%) WHEN LOCATED IN WASTE DISPOSAL AREAS.
5. MINIMUM ACCEPTABLE COLLECTION HEADER AND LATERAL PIPELINE SLOPE IS ONE PERCENT (1%) WHEN LOCATED OUT OF WASTE DISPOSAL AREAS.
6. ALL GAS EXTRACTION WELL LATERALS ARE SIX (6) INCH DIAMETER UNLESS OTHERWISE SPECIFIED. GAS EXTRACTION WELLS WILL BE OFFSET A MINIMUM OF TEN (10) FEET FORM THE HEADER PIPELINE.

REV	DATE	DESCRIPTION	DES BY	APP BY
1	4/94	REVISE SYMBOLOGY OF EXISTING GAS MONITORING PROBES GP-1 THROUGH GP-5	CFM	CML
0	11/93	ISSUE FOR PERMIT		

PREPARED BY:
RUST ENVIRONMENT & INFRASTRUCTURE
SOLID WASTE DIVISION – NAPERVILLE

NORTH CAROLINA REGISTERED PROFESSIONAL ENGINEER SEAL 17770
4/15/94

PROJECT NO	87221	TRANSITION APPLICATION AND PERMIT RENEWAL PIEDMONT LANDFILL AND RECYCLING CENTER KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA
DATE	NOVEMBER 1993	
DES BY	KK 11-29-93	GAS MANAGEMENT SYSTEM PLAN
DRN BY	PAA 11-29-93	
CHK BY	AJM 11-29-93	
APP BY	CML 11-29-93	

Waste Management of Carolinas, Inc.

SHEET 17 OF 21
DRAWING NO 17

FILE NAME: I:\ar\gis\wc\87221\gmp221.dgn
PLOT DATE: 04/15/94 TIME: 15:08