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August 26, 1998 96-248-79

Mr. Steven J. Donohue
United States Environmental Protection Agency
Region 3
1650 Arch Street
Philadelphia, PA 19103-2029

RE: Tonolli Corporation Superfund Site
Landfill Cap Re-Design Package

Dear Steve:

Enclosed please find the final package for the Tonolli landfill cap re-design. This final package responds to the USEPA, USACE, and PADEP comments to the July 29, 1998 cap re-design submission and includes a discussion of pertinent design issues, revised specifications, drawings, and calculations associated with the re-design of the landfill cap. The USEPA, USACE, and PADEP comments were addressed as follows:

- The proposed site fence along the eastern portion of the Site has been re-located to the base of the landfill embankment. The proposed fence alignment is shown on Sheet 14.

- The GCL to be placed over the western and northern portions of the landfill embankment will extend from the proposed cap anchor trench to the toe of the embankment.

- The Final Construction Specifications were reviewed for completeness with the design changes. Only Section 02751 (Cap Drainage Layer) and Section 02756 (Geosynthetic Clay Layer) required revision. These revised specifications are enclosed.

- Three landfill settlement monuments will be placed on the cap following completion of the cap construction. The proposed locations are shown on Sheet 14.

- Compaction equipment and procedures for soil and waste placement in the landfill are included in Section 02209 (Soil and Waste Removal/Handling/Placement) of the Final Construction Specifications.

- The cap bench/drainage swale was designed based on a rainfall intensity of 8.2 inches/hour for a 5 minute period, which generated a higher peak runoff rate than a 15 minute storm.

- AGC does not anticipate any significant differential settlement to occur across the 10-foot wide cap bench/drainage swales. The bends of the cap bench/swales were modified to lessen the degree of curvature.

- The final elevation of the waste will be dependent upon final excavation volumes. The waste will be placed at a 20% slope from the edge of the existing liner anchor trench until excavations at the Site are complete. Benches will be constructed as waste is placed at every 20 feet of rise in the cap. The top of the cap will be graded no flatter than 5%. The need for additional benches will be evaluated after all waste is placed in the landfill and the final height is established.

- The rip rap down slope drain protection shown on Sheet 14A was continued about 20 feet up the cap bench/drainage swales.

- The cap bench/drainage swale was realigned so that the southern landfill manhole is outside of the swale.

- The gradation of the rip rap for the drainage channels was modified to include a range of 3 inch to 9 inch aggregate.

- The width of the 2% portion of the cap bench/drainage swale subgrade has been provided on Sheet 15.

- The cap bench/drainage swale capacity calculations were evaluated assuming runoff coefficients for the pre-vegetation condition. Based on these calculations, the drainage swales are capable of handling a 100-year storm event for the pre-vegetation condition. AGC believes that a slope of 2 to 3% for the cap anchor trench is excessive and that the specified 0.5% is adequate. A 2 to 3% slope is not practical with a flat top of berm and would result in excessively deep anchor trenches or numerous outlet pipes.

- Sheet 15A does not show a plan view. However, the details on this sheet clearly reference the location of the GCL anchor and GCL end from the proposed cap anchor trench. The plan view of the proposed cap anchor trench is shown on Sheet 11.

- As requested by Joseph Mueller (USACE), the following additional cap stability calculations were performed.

    1. A factor of safety against sliding between the 60 mil LLDPE and the GCL was calculated. Based on the interface friction testing performed for the re-design, the interface between the LLDPE and GCL has the lowest interface friction.

    2. The internal shear strength of the GCL to be placed on the northern and western embankment slopes was check against the overburden stress carried through the geosynthetic components of the cap.

Summarized below are changes resulting from the re-design of the cap to a 20% maximum slope:

- The volume now provided above the current top of landfill embankment (EL. 1024) will be on the order of 90,500 cubic yards.

- The drainage layer component of the cap has been changed. The drainage layer shall now be a HDPE geonet with a geotextile bonded to both sides of the geonet. MACTEC has submitted a composite drainage layer manufactured by Evergreen Technologies. The double-sided geonet manufactured by Evergreen Technologies shall include the following:

  1. The geotextile shall be TG 700, a U.V. stabilized, spunbonded, continuous filament, needlepunched, non-woven, polypropylene geotextile bonded to both sides of the geonet.

  2. The geonet shall be Drainage Composite DC3205.

- The geosynthetic clay layer (GCL) has been changed to Bentomat DN as manufactured by CETCO.

- Benches in the landfill cap have been added at every 20 foot rise in the waste elevation.

- A GCL will be placed on the northern and western embankment slope in areas where contaminated materials are left in-place.

The revised specifications, drawings and calculations include the following:

Specifications

- Section 02751 (Cap Drainage Layer)
- Section 02756 (Geosynthetic Clay Layer)

Drawings

- Landfill Preparation and Top of Waste Plan (Sheet 11)
- Landfill Final Grading Plan (Sheet 14)
- Landfill Erosion and Sediment Control Plan (Sheet 14A)
- Landfill Capping Details (Sheet 15)
- Western Embankment Slope Capping Details (Sheet 15A)

Calculations

- Cap Stability
- Slope Stability
- Landfill Settlement
- Drainage and Erosion Control Calculations
- Geonet Transmissivity Calculations
- Western Embankment Slope GCL Stability Calculations

Discussions of pertinent design issues, revised specifications and supporting documentation for the design calculations are enclosed. The revised drawings are attached.

If you have any questions concerning this matter, please contact us at (610) 558-3300.

Sincerely,

ADVANCED GEOSERVICES CORP.

Todd D Trotman

Todd D. Trotman, P.E.
Project Engineer

Thomas Legel

Thomas M. Legel, P.E.
Project Manager

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Enclosures

cc: Jeff Leed
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Susan Schriner
John Lathram
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File

TONOLLI CORPORATION SUPERFUND SITE

LANDFILL CAP RE-DESIGN

Prepared For:

TONOLLI SITE RD/RA
STEERING COMMITTEE

Prepared By:

ADVANCED GEOSERVICES CORP.
Chadds Ford, Pennsylvania

Project No. 96-248-79
July 29, 1998
(Revised August 26, 1998)

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DISCUSSION OF DESIGN ISSUES

LANDFILL CAPPING

The cap will comply with the requirements of the Pennsylvania Hazardous Waste Regulations and will consist (from top to bottom) of a 6-inch layer of topsoil; 18-inches of select soil fill; a composite (geotextile/geonet/geotextile) drainage layer; a 60 mil LLDPE geomembrane; a geosynthetic clay layer (GCL); and 6-inches of select soil fill. The composite drainage layer has been changed to a geonet with geotextile bonded to both sides and the GCL has been changed to Bentomat DN as part of this redesign. A 6-inch layer of lime amended remediated soils will be placed below the select soil fill. The design drawings and calculations have been modified for a maximum finished cap slope of 20%, which will provide a total capacity of 90,500 cy above the existing embankment. The final elevation of the waste will be dependent upon final excavated soil volumes; however, the waste will be placed at a 20% slope until excavations at the Site are complete. The top of the cap will then be graded no flatter than 5%. Benches will be provided for every 20 feet of rise in the cap elevation.

A detailed landfill cap evaluation was performed as part of the design process. This evaluation included a cap stability analysis and an effectiveness evaluation using the HELP model. A discussion of these evaluations is provided below:

HELP Model

The HELP model evaluation was performed as part of the previous Final Design and the results do not change since this evaluation assumed a 3% finished slope (slopes steeper than 3% will result in less infiltration, resulting in a more conservative evaluation). Therefore, the HELP model calculations have not been revised.

Cap Stability Analysis

As part of the cap stability analysis performed during the re-design process, interface friction testing was performed on the geosynthetic cap components. This testing included the following:

1. Interface friction testing between the geonet and the LLDPE liner.

2. Interface friction testing between the geotextile portion of the geonet and the LLDPE liner.

3. Interface friction testing between the LLDPE liner and the geocomposite clay liner (GCL).

4. Interface friction testing between the GCL and landfill cap fill.

Cap stability calculations are attached and indicated a 20% slope can be achieved with the revised cap components.

Transmissivity calculations for the new drainage layer have been performed and indicate that sufficient drainage is provided by a single geonet sandwiched between two geotextiles for the proposed application. The calculations are attached.

SLOPE STABILITY

As presented in the Final Design, the existing landfill embankment slopes will be filled/regraded to achieve a 3:1 final slope. Therefore, no changes to the gabion wall or embankment grading as presented in the Final Design have been performed. Stability calculations for both the static and dynamic loading conditions have been revised for the 3:1 embankment slope and the gabion wall configurations assuming a 20% cap slope. The calculations demonstrate that the required factor of safety of 1.5 has been achieved for both loading conditions and are attached. As requested by the USACE, the slope stability calculations include the minimum factors of safety for both the 3:1 embankment slope and gabion wall configuration, as well as several other slip surfaces that pass through the landfill cap and embankment.

LANDFILL SETTLEMENT

A detailed analysis of the potential landfill settlement during and following closure activities was performed as part of the Final Design and has been modified to account for the additional waste that may be placed in the landfill. The analysis of potential settlements during and following landfill closure was performed for the following two conditions:

1. The removal of the standing landfill liquid.

2. The placement of additional waste and site soils.

The results of the settlement analysis for these two conditions are summarized below. The settlement calculations are attached.

Settlement Resulting from the Removal of Landfill Liquid

When the liquid (i.e., 30 feet of liquid) is removed from the waste within the existing landfill, the waste will experience an increase in stress equivalent to about 0.94 tons per square foot (tsf) due to the removal of the buoyancy effect of the liquid. The settlement due to this increased load is estimated to be a maximum of about 5-inches using Schmertmann's method for granular soils. Due to the granular nature of the waste materials, this settlement will occur during the removal of the liquid.

The majority of the liquid will be removed prior to and/or concurrent with waste placement. The foundation materials beneath the liner will experience a relaxation in overburden stresses equal to about 0.94 tsf resulting from the removal of liquid above the liner. Therefore, there will be no settlement of foundation materials caused by the removal of landfill liquids.

Settlement Due to the Placement of Additional Waste

Based on a 20% final cap slope, a maximum of about 37 feet of material (i.e., waste, site soils, and cap) at the landfill's highest point may be placed during closure activities. Based on Schmertmann's method, it is estimated that the settlement of the existing waste materials caused by this additional load could range from 0 inches to 12 inches. However, due to the granular nature of the existing waste, this settlement will occur during fill placement activities.

Settlement of Material Beneath Landfill

The average load of this additional material (i.e., 18.5 ft. x 120 psf) will be on the order of 1.1 tsf. Therefore, the net load (the difference between the relaxation of stress caused by the removal of the impounded liquid and the increase in stress caused by the placement of additional material) on the foundation materials beneath the landfill will be on the order of 0.16 tsf (representing a negligible increase in load). If some water is still in the landfill at the completion of waste placement, the increased load will be no greater than 0.30 tsf. Due to the competency of the foundation material, foundation settlement will be negligible for either of these conditions.

DRAINAGE CALCULATIONS

Increasing the cap slope to 20% will increase the surface water runoff from the cap. To handle this runoff, a bench at the base of the cap (top of landfill embankment) as well as on the cap slope at each 20 foot rise in elevation will be constructed. These benches will be grassed-lined, will have a width of 10 feet, a depth of 0.5 feet and will be sloped at a 1% (minimum) longitudinal grade. The capacity of these benches/swales is greater than the runoff produced from a 100-year storm event for both the pre-vegetation and post-vegetation condition. Swales carrying surface water from the bench to the base of the cap will be lined with rip rap. Hydraulic/hydrologic calculations supporting the design of these benches/swales are attached.

Storm routing for Basins 2 and 3 were also re-calculated. The results of the storm routing are very similar to those presented in the Final Design. Therefore, no changes to the basins are necessary.

NORTHERN AND WESTERN EMBANKMENT SLOPE CAPPING

The potential exists for soil containing lead to be present in the excavation sidewalls along the northern and western portion of the landfill embankment, and the removal of these materials may endanger the stability of the existing landfill. Therefore, the following excavation and capping procedures along the northern and western face of the landfill embankment are proposed, if sidewall samples contain lead at greater than 1,000 mg/kg.

- Excavate the soil removal areas along the northern and western embankment using sheeting and shoring, to be proposed by MACTEC and approved by AGC, as required.

- If soil with total lead concentrations above 1,000 mg/kg is encountered along the face of the embankment, no additional excavation will be performed into the landfill embankment unless AGC believes that the additional excavations will not jeopardized the stability of the landfill embankment. All other portions of the soil removal areas will be excavated to the required clean-up level of 1,000 mg/kg total lead.

- In areas where soil with lead concentrations above 1,000 mg/kg remain within the embankment, a GCL will be placed on the embankment slope. The GCL will be anchored in a trench constructed on the top of the landfill embankment and rolled down the slope. Placement of the GCL will be in accordance with Section 02756 of the Construction Specifications.

Details regarding the placement of the GCL are provided on Sheet 15A. Calculations regarding the stability of the GCL on the 3:1 slope are enclosed.

REVISED SPECIFICATIONS

SECTION 02756
GEOSYNTHETIC CLAY LAYER

PART 1: GENERAL

1.1 Description

This work shall include furnishing all materials, labor and equipment necessary to install a geotextile/bentonite/[redacted] composite liner (GCL) in accordance with the contract documents and as directed by the Resident Engineer.

1.2 Related Sections

A. Section 01050 - Field Engineering
B. Section 01300 - Submittals
C. Section 02210 - Earthwork
D. Section 02755 - Geomembrane

1.3 References

ASTM D5084	-	Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Parameter

1.4 Submittals

The Contractor shall submit to the Steering Committee and Resident Engineer a document, with sketches as appropriate, describing the method of placement and joining the proposed materials in the field in conformance with manufacturers recommended installation procedures.

The Contractor shall also submit samples of the materials proposed for use on the project, and sufficient information demonstrating that these materials comply with the applicable provisions of this Specification, including a certification from the manufacturer that the GCL meets the requirements for permeability.

1.5 Storage

The GCL rolls delivered to the project site shall be stored in their original, unopened wrapping in a dry area and protected from precipitation and the direct heat of the sun, especially when stored for a long period of time. The materials shall be stored above the ground surface and beneath a roof or shall be stored above the ground surface and beneath a roof or other protective covering. Care shall be taken to keep the GCL clean and free from debris prior to installation.

PART 2: PRODUCTS

2.1 Geosynthetic Clay Liner (GCL)

The GCL shall consist of a sodium bentonite core between two geotextiles (Bentomat DN as manufactured by CETCO, or the equivalent). The material shall have a minimum bentonite content of 0.75 pound per square foot. The GCL shall also have a typical permeability of 5 x 10-9 cm/sec, as determined by ASTM D5084. A certification from the manufacturer verifying the permeability of the material shall be obtained and submitted to the QA Official prior to installation.

PART 3: EXECUTION

3.1 Subbase Preparation

The Contractor shall be responsible for inspection of the GCL upon delivery to the job site. Should any of these materials show damage, they shall be identified by the Contractor and shall not be used. During installation of the GCL, the QA Official shall carry out visual inspections of all materials.

Any defects in materials shall be repaired or replaced, as approved by the Resident Engineer. The Contractor will be responsible for preparation of all surfaces prior to installation of the GCL. The soil subbase shall be rolled and compacted prior to GCL placement so as to be free of irregularities, protrusions, loose soil, and abrupt changes in grade. Compaction shall be performed as detailed in the Earthwork Specification.

The 6-inch subbase shall not contain sharp stones or protruding objects. If sharp or protruding objects are detected ruing subbase inspection, they shall be removed and any resultant voids shall be backfilled.

The Resident Engineer and Installer shall approve the subbase prior to GCL placement. The Installer shall certify in writing that the surface on which each section of GCL will be installed is acceptable. These certificates of acceptance shall be given by the Installer to the Resident Engineer prior to commencement of panel placement.

At any time prior to or during GCL placement, the Resident Engineer may indicate to the Contractor locations of uncovered subbase areas which may not provide adequate support for the GCL and which will require corrective action prior to GCL installation. The Contractor shall then perform the appropriate corrective action.

3.2 Installation

The GCL shall be laid out and installed by the Contractor in accordance with the Manufacturer's recommendations. The materials shall be placed and aligned from the top of the slope towards lower grade.

The layout of all materials shall be designed to minimize the number and length of overlap seams, consistent with Manufacturer's recommended method of installation. Seams shall be minimized, and whenever possible, run parallel to the direction of the slope if the slope is steeper than 3%.

Travel on the materials shall be controlled to prevent tracking, rutting or failure of materials or foundation.

Track equipment shall not be allowed to travel directly on top of the installed GCL.

The GCL shall not be installed when it is raining or when rain is pending. The GCL must be dry when installed and must be dry when covered. The overlying geomembrane shall be placed immediately over the installed GCL. No portion of the GCL shall be left uncovered overnight, or during periods of work stoppage. The leading edge of the GCL shall be secured at all times with sand bags or other means sufficient to hold it down during high winds. GCL that becomes wet shall be removed and replaced at the Contractor's expense.

Damaged areas shall be repaired by patching with pieces of GCL cut to overlap the perimeter of the damaged area by a minimum of twelve inches. Patches shall be held in place by completely covering them with sand bags, or as approved by the Resident Engineer.

The GCL shall be installed in a relaxed condition and shall be free of tension or stress upon completion of the installation. Stretching of the GCL to fit will not be allowed. The GCL shall be adjusted to smooth out creases or irregularities.

After the first roll has been laid, adjoining rolls shall be laid with a twelve inch overlap. All dirt shall be removed from the overlap area of the mat. Field seams shall be made as per the Manufacturer's recommendations.

All seams shall be protected against movement and wind damage during construction and until placement of overlying materials.

All overlapping of the GCL shall be inspected by the QA Official and Contractor to insure that the minimum overlap exists. In addition, the GCL shall be inspected by the Contractor for any tears or punctures and shall be repaired or replaced as deemed necessary by the Resident Engineer.

PART 4: MEASUREMENT AND PAYMENT

4.1 Measurement

Measurement for payment shall be based on the actual number of square yards of surface area of in-place GCL.

The price shall include, but will not be limited to, submittals; material manufacture, packaging, delivery, and storage; GCL deployment, seams, overlaps, and repairs; and clean up.

No additional payment shall be made for removing approved GCL material which is rendered unsuitable due to adverse weather conditions. Damaged material shall also be removed at no additional cost.

4.2 Payment

The completed work as measured for GCL shall be paid for according to the unit price schedule.

PAY ITEM	PAY UNIT
Geosynthetic Clay Liner	Square feet

SECTION 02751

CAP DRAINAGE LAYER

PART 1: GENERAL

1.1 Description

The work covered by this section includes installation of the cap drainage layer for the final closure cap systems. This includes manufacture, fabrication, packaging, delivery, and installation of all components. Specific components include the composite drainage layer (geotextile/geonet/geotextile composite), perforated anchor trench drain, granular fill, and geotextiles.

1.2 Related Sections

A. Section 01050 - Field Engineering
B. Section 01300 - Submittals
C. Section 02210 - Earthwork
D. Section 02755 - Geomembrane

1.3 References

ASTM D422	-	Test Method for Particle-size Analysis of Soils
ASTM D1682	-	Test Method for Strip Tensile Strength
ASTM D2487	-	Procedure for Classification of Soils for Engineering Purposes
ASTM D4354	-	Standard Practice for Sampling of Geosynthetics for Testing
ASTM D4533	-	Test Method for Trapezoid Tearing Strength of Geotextiles
ASTM D4595	-	Test Method for Tensile Properties of Geotextiles by the Wide Width Strip Method

ASTM D4632	-	Test Method for Breaking Load and Elongation of Geotextiles (Grab Method)
ASTM D4716	-	Test Method for Constant and Hydraulic Transmissivity of Geotextiles and Geotextile Related Products
ASTM D4751	-	Test Method for Determining Apparent Opening Size of a Geotextile
ASTM D4759	-	Standard Practice for Determining the Specification Conformance of Geosynthetics
ASTM D4833	-	Test Method for Index Puncture of Geotextiles, Geomembranes and Related Products

1.4 Submittals

The Contractor shall submit Manufacturer's literature and specification for perforated piping to the Resident Engineer for approval. The Contractor shall submit Manufacturer's specifications and physical property information for the composite drainage layer to the Resident Engineer for approval.

1.5 Storage

The composite drainage layer rolls delivered to the project site shall be stored in their original, unopened wrapping in a dry area and protected from precipitation and the direct heat of the sun, especially when stored for a long period of time. The materials shall be stored above the ground surface and beneath a roof or other protective covering.

1.6 Quality Assurance

Quality assurance of geosynthetic installation shall be performed in accordance with the Construction Quality Assurance Plan.

PART 2: PRODUCTS

2.1 Geonet

The geonet shall be a high density polyethylene (HDPE) material with intersecting material strands creating a three dimensional structure which supports planner water flow. The geonet shall ~~conform to the following requirements or the manufacturers minimum published values, whichever is more restrictive~~ be drainage composite DC3205 manufactured by Evergreen Technologies, Inc. or the equivalent.

~~Properties~~	~~Test Method~~	~~Required Value~~
~~Transmissivity (M²/S), min.~~	~~ASTM D4716~~ ~~i = 1.0~~ ~~σ = 2000 psf94.~~	~~1.4 x 10⁻³~~
~~Tensile Strength (lb/in), min.~~	~~ASTM D1682 or D4595~~	~~22~~

Contractor shall provide conformance testing as required by Construction Quality Assurance Plan.

2.2 Pipe

The pipe used within the perimeter cap drainage system (where required) shall be 4 inch perforated corrugated polyethylene tubing (Class 2 Perforations) meeting the requirements of AASHTO M25-94. The pipe shall include all appropriate connections and end protection recommended by the manufacturer and as shown on the design drawings.

2.3 Geotextile

The geotextile bonded to ~~the geonet~~ both sides of the geonet shall be ~~a non-woven material conforming to the following requirements~~ FG-700, a U.V. stabilized, spunbounded, continuous filament, needlepunched, non-woven, polypropylene geotextile manufactured by Evergreen

Technologies, Inc. or the equivalent. Geotextile shall be heat bonded to the geonet and extend a minimum distance of 6-inches beyond the geonet at either end.

~~Properties~~	~~Test Method~~	~~Required Value~~
~~Grab Strength (lbs.), min.~~	~~ASTM D4632~~	~~150~~
~~Puncture Strength (lbs.), min.~~	~~ASTM D4833~~	~~75~~
~~Tear Strength (lbs.), min.~~	~~ASTM D4533~~	~~70~~
~~Mass per Unit Area (oz/sy), min.~~	~~ASTM D3776~~	~~8~~
~~Apparent Opening (US sieve No.)~~	~~ASTM D4751~~	~~100~~
~~Ply Adhesion (lbs/in)~~	~~ASTM D413~~	~~2.0~~

The geotextile wrap used for the cap edge drains shall meet the same requirements but will not be bonded to the geonet.

2.4 Granular Fill

Granular fill shall be used as drainage material around the piping system for the perimeter cap drain and the cap edge drain. Granular fill shall be clean, rounded material with particles not larger than 1-1/2-inch in diameter and no greater than 5 percent fines and shall be AASHTO #57 gradation.

PART 3: EXECUTION

3.1 General

The work shall be coordinated with placement of the LLDPE geomembrane and anchor trench backfill. The cap drainage layer shall be placed directly above the LLDPE geomembrane.

Prior to placement of the cap drainage layer, the portion of the geomembrane to be covered by the ~~geotextile~~ geonet/geotextile composite shall have all required documentation complete. The surface of the geomembrane shall not contain stones or excessive dust that could cause damage.

The composite drainage layer shall be cut, if necessary, using an Resident Engineer approved cutter. Care must be taken to protect underlying geomembrane if the geonet or geotextile is being cut in place.

Equipment used to deploy the composite drainage layer shall not damage the materials or the underlying geomembrane.

3.2 Composite Drainage Layer

3.2.1 Placement

The Contractor shall keep the composite drainage layer clean and free from debris. Soils and debris shall be cleaned by the Contractor just prior to installation, as determined by the Resident Engineer. The Installer shall handle all rolls in a manner to ensure they are not damaged in any way. To prevent folds and wrinkles, tension should be kept on the materials. Materials shall not be placed across side slopes. Geotextile side of the composite shall be placed facing up.

In the presence of winds, the composite drainage layer shall be weighted with sandbags, as necessary. The Installer shall be responsible for damage caused by wind.

3.4.2 Connections

Adjacent geonet rolls shall be overlapped at least 6-inches and secured by plastic ties approximately every three (3) feet along the roll length. Plastic ties shall be white or another bright color for easy inspection. Metallic ties shall not be allowed. The heads of the ties must fit completely into the geonet channel space so that the head of the tie does not intrude into or against the primary liner.

Adjacent pieces of composite drainage layer shall have their geotextile components lystered together after the geonet is connected and accepted by QA Official.

Horizontal seams shall not be placed on side slopes greater than 3% unless approved by the Resident Engineer in the panel placement plan.

3.4.3 Repair

Patching of the composite shall be used to repair holes, tears, and defects. Patches shall provide 6" of overlap around the repaired area and shall be held in place with plastic ties. Composite shall be removed if areas with large defects are observed. The Resident Engineer shall determine the acceptability of the composite drainage layer.

3.5 Drainage Layer Edge Drain

The 4-inch diameter perforated polyethylene pipe shall be placed in the anchor trench following placement of the cap geomembrane and geotextile wrap. The Contractor shall place the pipe in a manner which ensures underlying materials are not damaged. Endcaps shall be placed on the upslope end of the perforated pipe. Details of the pipe layout can be seen in the Drawings.

Granular fill shall be placed around the pipe for drainage. Granular fill shall be placed by the Contractor in a manner which ensures surrounding materials are not damaged. Granular fill shall be placed to provide proper support for the overlying trench backfill. The Resident Engineer shall monitor fill placement.

3.6 Cap Drainage Layer Acceptance

The Contractor shall retain all ownership and responsibilities for the cap drainage layer until acceptance by the Steering Committee. The Steering Committee will accept the cap drainage layer when:

1. All required documentation from the Manufacturer and Installer has been received and approved.

2. The installation is complete.

PART 4: MEASUREMENT AND PAYMENT

4.1 Measurement

Measurement for payment for the composite drainage layer will be based on the actual number of square yards of covered surface area in-place.

The cap drainage layer edge drain shall be measured as lineal feet in-place and shall include required granular fill, perforated pipe, pipe fittings, and geotextile.

Granular fill will not be measured and will be considered incidental to pipe placement.

4.2 Payment

All prices shall include, but will not be limited to, submittals; material manufacture, packaging, delivery, and storage; deployment, patches, seams, overlaps, repairs; and cleanup.

All work associated with furnishing and hauling material will not be paid separately but shall be included in the work required, or as approved by the Resident Engineer.

No additional payment will be made for removing approved materials which are rendered unsuitable after placement or replacement or for removal, hauling, disposal and replacement of objectionable materials.

The completed work as measured for the cap drainage layer shall be paid for according to the unit price schedule.

PAY ITEM	PAY UNIT
Composite Drainage Layer	Square yard
Edge Drain (complete)	Linear foot

CAP STABILITY CALCULATIONS

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I INTRODUCTION

THE PURPOSE OF THIS ANALYSIS IS TO DEMONSTRATE THE FEASIBILITY OF CAPPING THE TONOLLI SITE LANDFILL.

THE ANALYSIS WILL BE PREFORMED IN TWO STAGES:

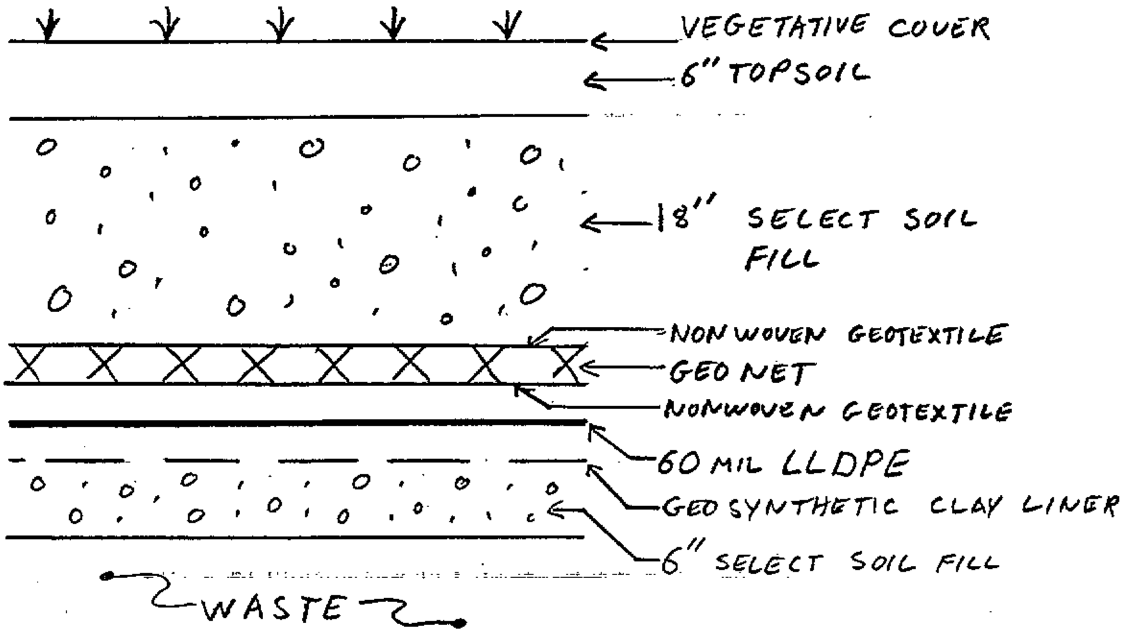
1.) THE FIRST STAGE WILL FOCUS ON THE STABILITY OF THE SOIL COVER. THE COVER MATERIAL WILL BE ANALYZED AS AN INFINITE SLOPE WITHOUT SEEPAGE Forces. (The DRAINAGE LAYER WILL minimize SEEPAGE Forces)

2.) THE SECOND STAGE WILL FOCUS ON THE STABILITY OF THE CAP COMPONENTS WITH REGARD TO THE GEOSYNTHETICS BELOW THE COVER SOIL.

IN ADDITION, STABILITY CALCULATIONS FOR THE GCL COVER ON THE NORTH AND WESTERN EMBANKMENT SLOPES ARE PROVIDED.

RESULTS OF INTERFACE FRICTION TESTING ARE ATTACHED.

CAP SYSTEM CONFIGURATION



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II SLOPE STABILITY CALCULATIONS FOR SOIL COVER

THE SLOPE STABILITY WILL BE ANALYZED VIA AN INFINITE SLOPE ASSUMING NO PORE-WATER PRESSURES (GEONET DRAIN BELOW THE SOIL COVER WILL ELIMINATE PORE WATER PRESSURES).

GIVEN THE SHEAR STRENGTH OF THE COVER MATERIAL:

$$\tau_f = c + \sigma \tan\phi$$

WHERE:

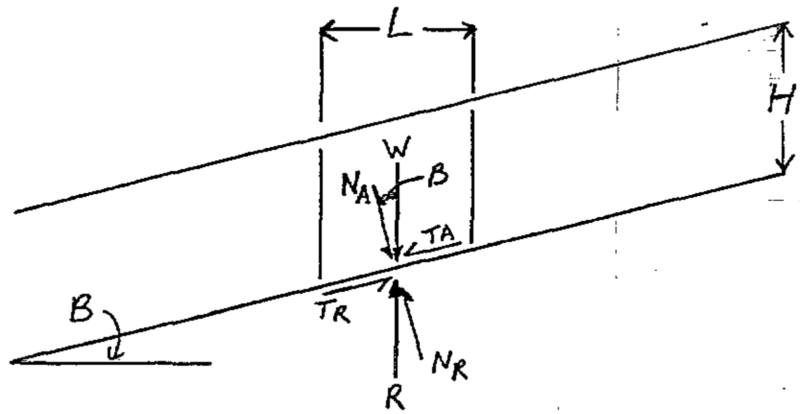
τ_f = SHEAR STRENGTH (lb/ft^2)

c = COHESION (lb/ft^2)

σ = NORMAL STRESS (lb/ft^2)

ϕ = ANGLE OF INTERNAL FRICTION (DEGREES)

FREE BODY DIAGRAM



THE WEIGHT OF THE SOIL ELEMENT IS:

$$W = \gamma L H$$

W/ COMPONENTS OF FORCE:

PERPENDICULAR (NORMAL), $N_A = W\cos\beta = \gamma L H \cos\beta$

PARALLEL (IN PLANE). $T_A = W\sin\beta = \gamma L H \sin\beta$

THE NORMAL STRESS IS:

$$\sigma = \frac{N_A}{A} = \frac{\gamma L H \cos\beta}{L/\cos\beta} = \gamma H \cos^2\beta$$

THE SHEAR STRESS IS:

$$\tau = \frac{T_A}{A} = \frac{\gamma L H \sin\beta}{L/\cos\beta} = \gamma H \sin\beta\cos\beta$$

FOR EQUILIBRIUM, THE RESISTANCE SHEAR STRESS DEVELOPED AT THE BASE OF THE ELEMENT IS:

$$\tau_d = c_d + \gamma H \cos^2\beta - TAN\phi$$

AND THE EQUATION FOR FACTOR OF SAFETY CAN BE DERIVED

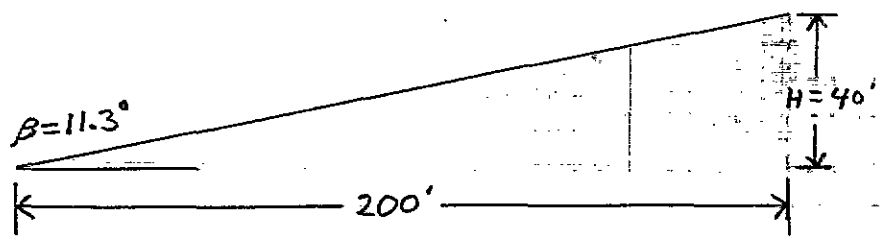
$$FS = \frac{c}{\gamma H \cos^2\beta \, TAN\beta} + \frac{TAN\phi}{TAN\beta}$$

SELECTION OF PARAMETERS FOR SLOPE STABILITY

ANTICIPATED SLOPE ANGLE, β_{MAX}, FOR THE COVER IS 20% or

$$\beta_{MAX} = 11.3°$$

THE MAXIMUM SLOPE LENGTH, L_{MAX}, IS EXPECTED TO BE 200 FEET



SOIL PARAMETERS (NAVFAC DM-7, 1991)

SOIL IS ASSUMED TO BE SM-SC, A SAND-SILT CLAY MIX

PARAMETERS	TYPICAL VALUES	VALUES USED
WET DENSITY, γ	110-130 pcf	130 pcf
COHESION, C SAT DRY	300 psf 1050 psf	200 psf
∡ INTERNAL FRICTION, ϕ	26°-33°	26°

EVALUATION OF CAP SOIL STABILITY

GIVEN:

$\gamma_{WET} = 130 \text{ pcf}$

$\phi = 26°$

$\beta = 11.3°$

$c = 200 \text{ psf}$

COVER HEIGHT $H = 2.0'$

USE FS (SEE SHEET 5)

$$FS = \frac{200 \text{ psf}}{(130 \text{pcf})(2.0')(\cos^2(11.3°)\ \text{TAN}(5.7°))} + \frac{\text{TAN}(26°)}{\text{TAN}(11.3°)}$$

$= 3.9 + 2.4$

$= 6.3$

$6.3 >> 1.5 \text{ MIN}$

THERE FORE, THE COVER SOIL WILL REMAIN IN PLACE GIVEN THAT THE SOIL HAS THE PARAMETERS IDENTIFIED HERE IN AND THAT THE COVER SOIL IS FREELY DRAINING, ELIMINATING SEEPAGE FORCES.

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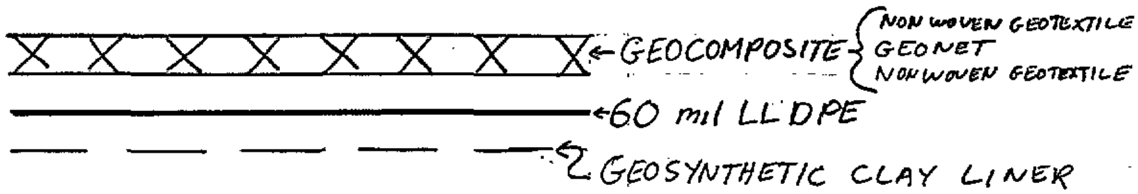
III SLOPE STABILITY OF THE GEOSYNTHETIC LAYER

THE GEOSYNTHETIC CROSS-SECTION WILL BE AS FOLLOWS:



GEOSYNTHETIC INTERFACE FRICTION

ASSUMPTIONS:

	TYPICAL VALUE	VALUE USED
SOIL FRICTION ANGLE, ϕ	26°-34°	26°
NONWOVEN GT. TO SOIL, δ_1	14°-22°	22°
NONWOVEN GT TO GEOMEMBRANE, δ_2	8°-15°	17° ACTUAL VALUE
GEOMEMBRANE TO GCL, δ_3	12°-18°	13° "Y" intercept @ 0.75 19.5° "Y" intercept @ 0
GCL TO SOIL, δ_4	14°-18°	27.6° ACTUAL VALUE

NOTE: GEOSYNTHETIC INTERFACE FRICTION ANGLES LAB TESTED BY, GEOSYSTEMS CONSULTANTS, INC; SINCE CAP SLOPE EXCEED 10 PERCENT.

SHEET 7 OF 13 PROJECT NO. 96-248-79 PROJECT NAME TONOLLI
BY CJN DATE 7-28-98 DESCRIPTION CAP STABILITY
CHK. BY TML DATE 7-29-98

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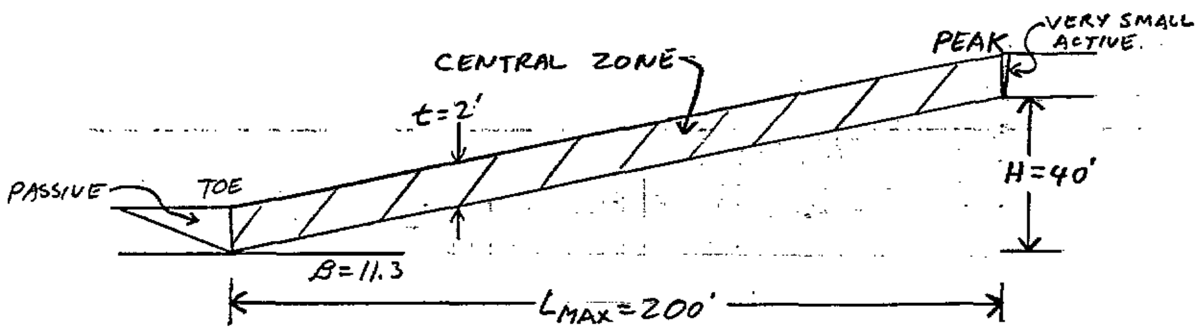
TYPICAL CROSS-SECTION AT MAXIMUM LENGTH



IGNORE BOTH ACTIVE AND PASSIVE CONTRIBUTIONS

WEIGHT OF CENTRAL ZONE, W_c:

$$W_c = L_{MAX} \times t \times \gamma = 200' \times 2.0' \times 130\,pcf = 52{,}000\ lb/ft\text{-}width$$

DRIVING FORCE, D_c:

$$D_c = W_c \sin\beta = 52{,}000\ lb/ft\text{-}w\,(\sin 11.3°) = 10{,}190\ lb/ft\ width$$

RESISTING FORCE, R_c:

$$R_c = W_c \cos\beta \tan\delta_1 = 52000(\cos 11.3)(\tan 22°) = 20{,}602\ lb/ft\text{-}width$$

WHERE δ_1 = INTERFACE FRICTION ANGLE BETWEEN SOIL AND UPPER LAYER OF GEOTEXTILE.

GIVEN:

$D_c = 10{,}190$ lb/ft width.

$R_c = 20{,}602$ lb/ft width

IF ALL OF R_c COULD BE MOBILIZED,

$$F.S. = R_c/D_c = 20{,}602/10190 = 2.0$$

EVALUATION OF CALCULATION

THIS ANALYSIS WAS PERFORMED IN ACCORDANCE WITH PUBLISHED METHODS. THE CONTRIBUTION OF PASSIVE RESISTANCE IS ANTICIPATED TO BE MUCH GREATER THAN THE CONTRIBUTION OF ACTIVE RESISTANCE. THIS COUPLED WITH THE ADEQUATE DRAINAGE PROVIDED BY THE GEONET WILL ALLOW FOR A STABLE DESIGN.

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IV SHEAR STRESS EVALUATION OF GEOSYNTHETIC LAYER

CROSS-SECTION

$\beta = 11.3°$

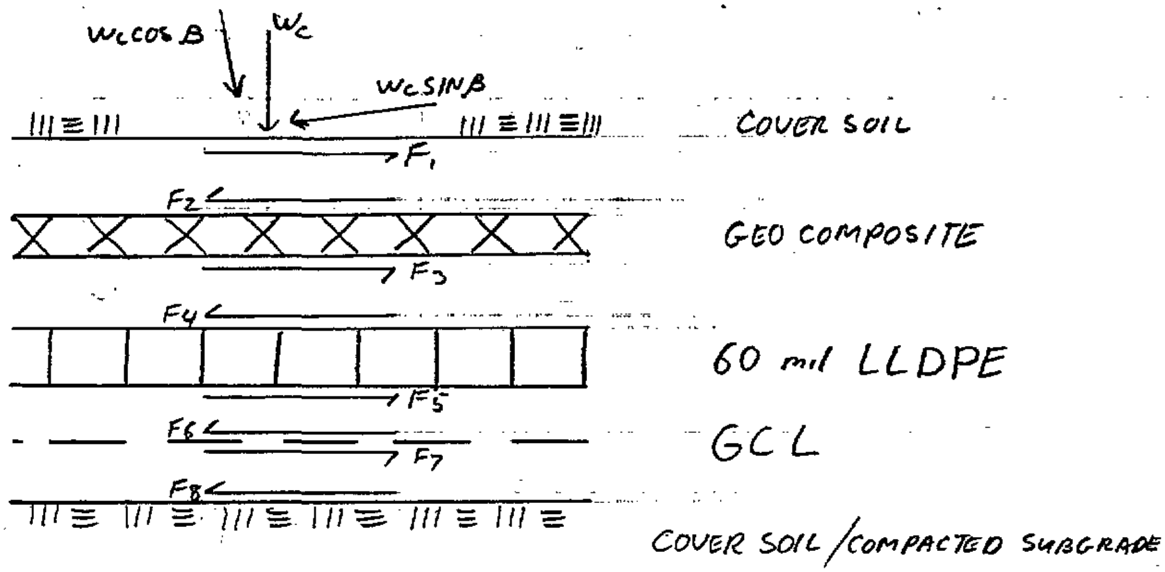
$\phi = 26°$

$\delta_1 = 22°$

$\delta_2 = 17°$

$\delta_3 = 13.1°$

$\delta_4 = 27.6°$



$W_c = 52{,}000$ lb/ft width (SEE PAGE 9)

$D_c = 10{,}190$ lb/ft width

F_1 = RESISTING FORCE $= R_c = 20{,}602$ lb/ft width (MAXIMUM CAN BE MOBILIZED)
(FROM PG. 9)

HOWEVER, F_1 WILL BE EQUAL TO D_c AS LONG AS $D_c \le R_c$

SO $F_1 = 10{,}190$ lb/ft width

$F_2 - F_1 = 0$ SO $F_2 = F_1 = 10{,}190$ lb/ft width

SHEET 10 OF 13 PROJECT NO. 96-248-79 PROJECT NAME TONOLLI
BY CJN DATE 7-28-98 DESCRIPTION CAP STABILITY
CHK. BY TML DATE 7-29-98

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$F_3 = W_C \cos\beta \tan\delta_2 = 52{,}000\ \text{lb/ft width} (\cos 11.3)\ \tan(17°)$

$F_3 = 15\,590\ \text{lb/ft width}$ (MAXIMUM CAN BE MOBILIZED)

SINCE $F_3 >> F_2$; $F_3 + F_2 \leq 0$

THERE IS $15{,}590 - 10{,}190 = 5400\ \text{lb/ft}$ OF FRICTION REMAINING TO BE MOBILIZED

$\therefore F_3 = F_2 = 10{,}190\ \text{lb/ft width}$

ALL OF THE OVERBURDEN STRESS IS CARRIED BY FRICTION OF THE GEOCOMPOSITE.

THE REMAINING STRESSES ARE:

$F_4 - F_3 = 0$ SO $F_4 = F_3 = 10{,}190\ \text{lb/ft width}$

$F_5 = W_C \cos\beta \tan\delta_3 = 52{,}000$ lb/ft width $(\cos 11.3°)\tan(13.1°)$

$F_5 = 11{,}866$ lb/ft width

SINCE $F_5 >> F_4$ & $F_5 + F_4 \leq 0$

THERE IS $11{,}866 - 10{,}190 = 1{,}676$ lb/ft RESERVE FRICTION

$F_5 = F_4 = 10{,}190$ lb/ft width

ALL OF THE OVERBURDEN STRESS IS CARRIED BY FRICTION OF THE LLDPE LINER.

$F_6 - F_5 = 0$ SO $F_6 = F_5 = 10{,}190$ lb/ft width

$F_7 = W_C \cos\beta \tan\delta_4 = 52000$ lb/ft width $(\cos 11.3)\tan(27.6°) =$

$F_7 = 26{,}660$ lb/ft width

SINCE $F_7 >> F_6$ & $F_7 + F_6 \leq 0$

THERE IS $26{,}660 - 10{,}190 = 16{,}470$ lb/ft RESERVE FRICTION

$F_7 = F_6 = 10{,}190$ lb/ft width

ALL OF THE OVERBURDEN STRESS IS CARRIED BY THE GCL.

THE INTERNAL SHEAR STRENGTH OF THE GCL IS 500 lb/sq ft.

∴ THE INTERNAL RESISTANCE = 500 lb/sq ft × 200' slope length.

= 100,000 lb/ft width

∴ GCL WILL CARRY LOAD THROUGH TO UNDERLYING SOIL.

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SUMMARY:

THESE CALCULATIONS INDICATE THAT INTERFACE FRICTION WILL CARRY THE COVER SOIL LOAD TO THE UNDERLYING SOIL, AND THAT NONE OF THE GEOTEXTILE LAYERS WILL EXPERIENCE TENSILE STRESSES DUE TO COVER SOIL LOADS.

THE WEAKEST INTERFACE WILL BE BETWEEN THE GEOMEMBRANE AND THE GCL ($\delta_3 = 13.1°$) IF SLIDING WERE TO OCCUR AT THIS INTERFACE THE TENSILE STRENGTH OF THE GEOMEMBRANE WOULD BE MOBILIZED, THE MINIMUM AVERAGE TENSILE STRENTH IS 255 lb/inch or 3060 lb/FT (AT BREAK)

AS THE GEOMEMBRANE ELONGATES THE TENSILE STRENGTH OF THE GEOCOMPOSITE WOULD BE MOBILIZED ALSO, WHICH IS 370 lb/FT

IN ADDITION, A PASSIVE RESISTANCE FORCE AT THE TOE OF SLOPE WOULD ALSO BE MOBILIZED.

$$F_P = \tfrac{1}{2} K_p \gamma H^2 \qquad K_p \approx \frac{1+\sin\phi}{1-\sin\phi} \qquad \phi = 26° \quad K_p = 2.56$$

$$F_P = \tfrac{1}{2}(2.56)(130)(2)^2 = 665.6 \text{ lb/ft width.}$$

$\therefore$ THE MINIMUM FACTOR OF SAFETY FOR THE SYSTEM IS:

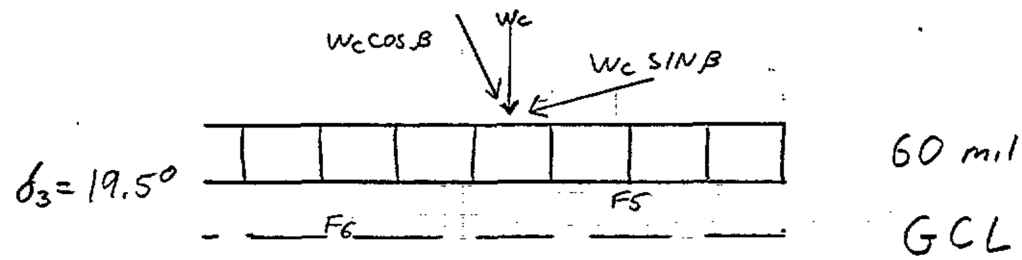
$$\frac{F_R}{F_D} = \frac{11{,}866 + 3060 + 370 + 665.5}{10{,}190} \approx 1.6$$

THIS IS AN ACCEPTABLE FACTOR OF SAFETY FOR THE SYSTEM

SHEET 13 OF 13	PROJECT NO. 96-248-7	PROJECT NAME	TONOLLI
BY CJN	DATE 8-5-98	DESCRIPTION	CAP STABILITY
CHK. BY TML	DATE 8-5-98		

ANALYSIS OF INTERFACE AT GEOMEMBRANE AND G.C.L.

CROSS-SECTION



WEIGHT OF CENTRAL ZONE, W_c:

$$W_c = L_{MAX} \times t \times \gamma = 200' \times 2.0' \times 130\,pcf = 52{,}000\ \text{lb/ft width.}$$

DRIVING FORCE, D_c:

$$D_c = W_c \sin\beta = 52{,}000\ \text{lb/ft width}\,(\sin 11.3^\circ) = 10{,}190\ \text{lb/ft width.}$$

(SEE TONOLLI CORPORATION SUPERFUND SITE LANDFILL CAP RE-DESIGN; CAP STABILITY CACULATIONS, SHEET 8 OF 13, DATED 7-29-98)

SHEET 1 OF 4	PROJECT NO. 96-248-79	PROJECT NAME TONOLLI
BY CJN	DATE 8-17-98	DESCRIPTION GEOMEMBRANE & GCL INTERFACE
CHK. BY	DATE	

THE MAXIMUM AMOUNT OF RESISTING FORCE DEVELOPED BETWEEN THE INTERFACE OF THE GEOMEMBRANE AND THE GCL (F_5) IS DETERMINED BY ASSUMING NO COHESION AT THE INTERFACE. DETERMINING A LINEAR REGRESSION FOR NO COHESION BETWEEN THE GEOMEMBRANE AND THE GEOSYNTHETIC CLAY LINER YIELDS AN INTERFACE FRICTION ANGLE OF $\delta_3 = 19.5°$.

$$F_5 = W_c \cos\beta \tan\delta_3 = 52000 \text{ lb/ft w} (\cos 11.3)(\tan 19.5°)$$

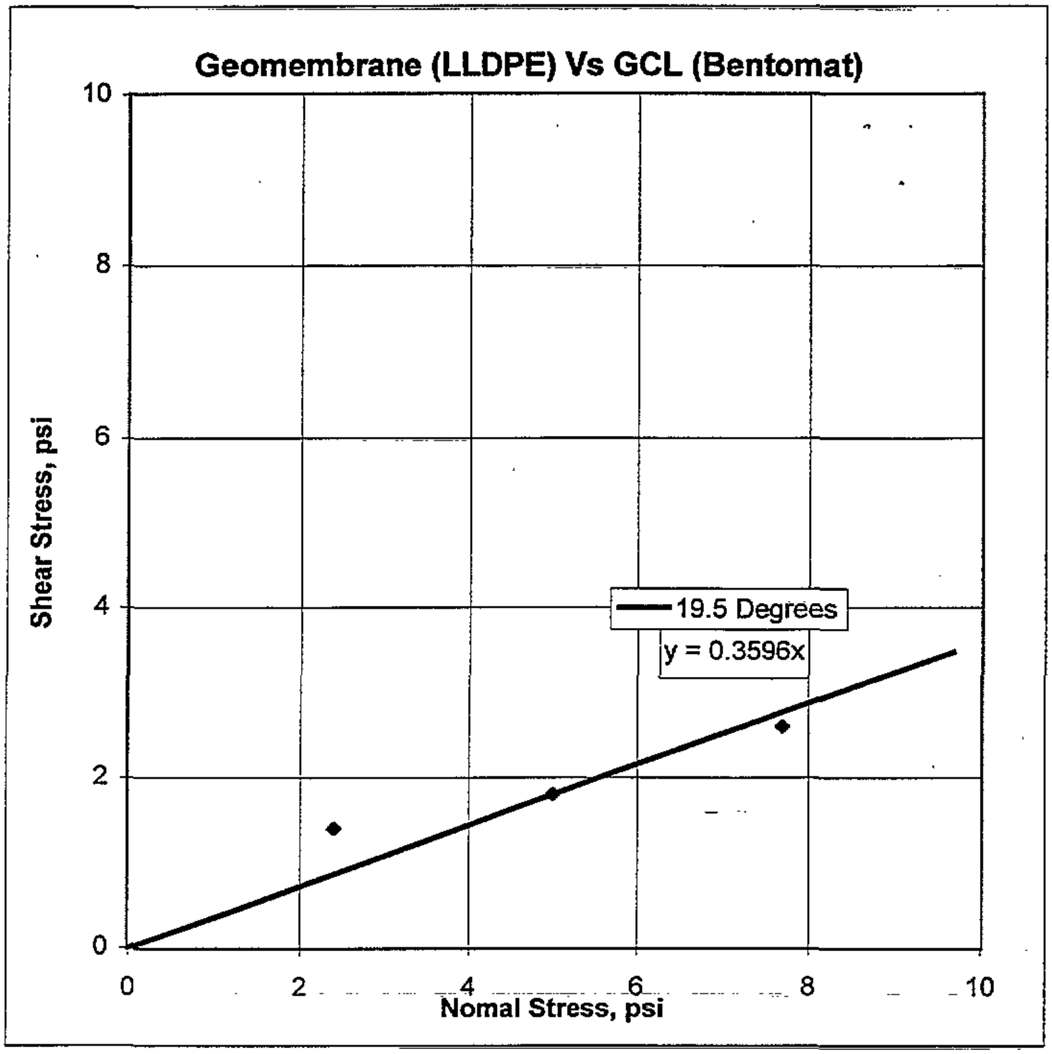
$$F_5 = 18{,}060 \text{ lb/ft width.}$$

SINCE $F_5 >> D_c$ & $F_5 + D_c \leq 0$

THERE IS $18{,}060 - 10{,}190 = 7{,}870$ lb/ft w OF RESERVE FRICTION

FACTOR OF SAFETY

$$\frac{F_5}{D_c} = \frac{18{,}060}{10{,}190} = 1.7$$



Note:
1 Data from GeoSystems Consultants, Inc

The strength envelope for the GCL and LLDPE as determined by the attached laboratory testing was re-drawn assuming no cohesion. The interface friction with cohesion, as determined by shear box testing, is 13.1°. The interface friction assuming, no cohesion, was calculated to be 19.5°.

GeoSystems Consultants , Inc.

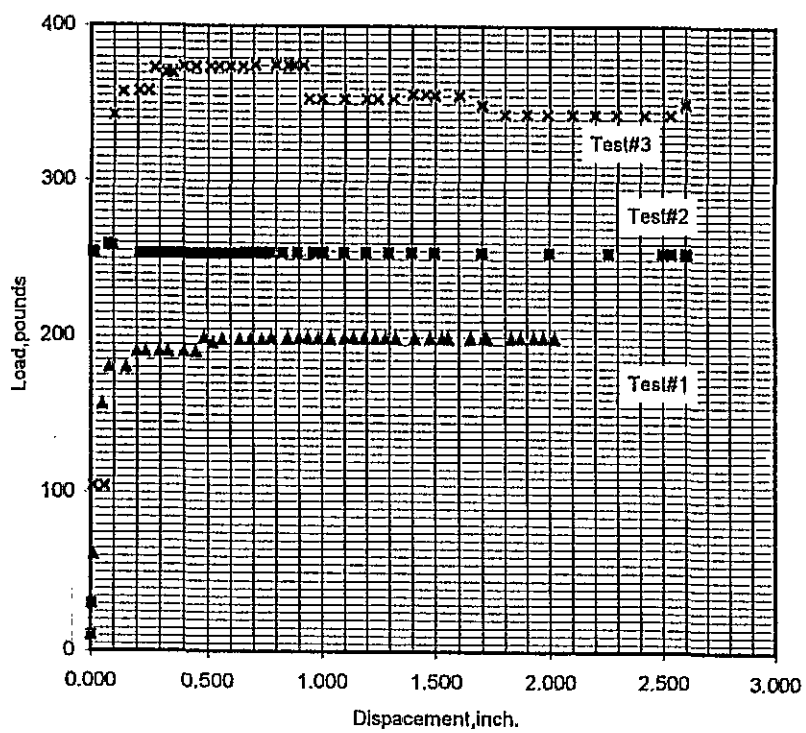
Fort Washington , Pa.

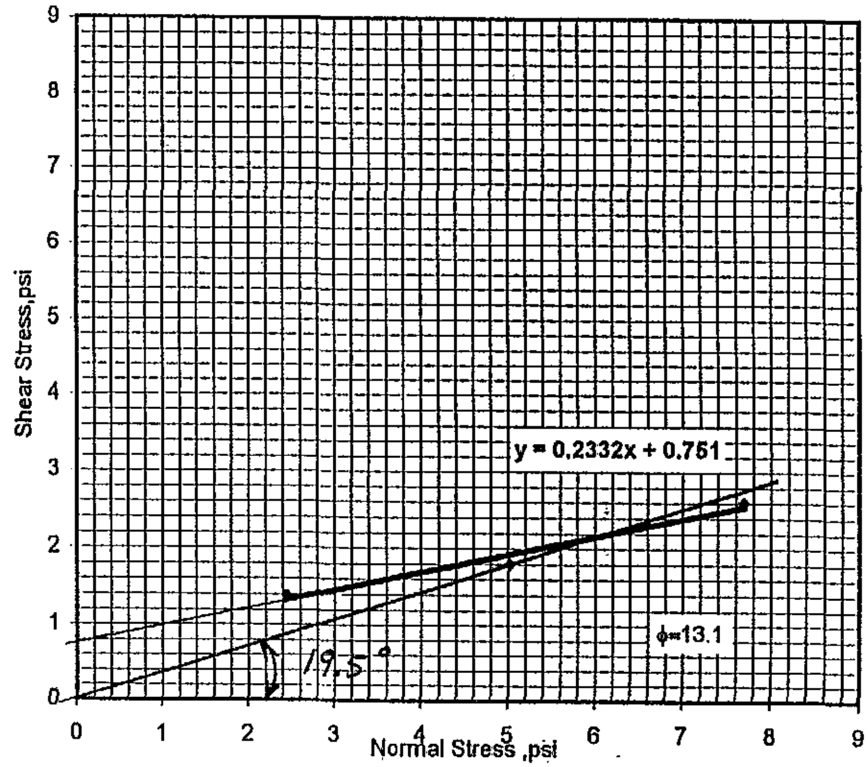
Job No. 98G022 Job Name AGC: Tonolli Superfund Site

Date 7/28/98

Geomembrane (LLDPE 60MIL) Vs GCL(Bentomat))

Test No.	Normal Stress,psi
1	1.0
2	3.0
3	6.0
4	9.0





98022ds3

AR305346

GeoSystems Consultants , Inc.

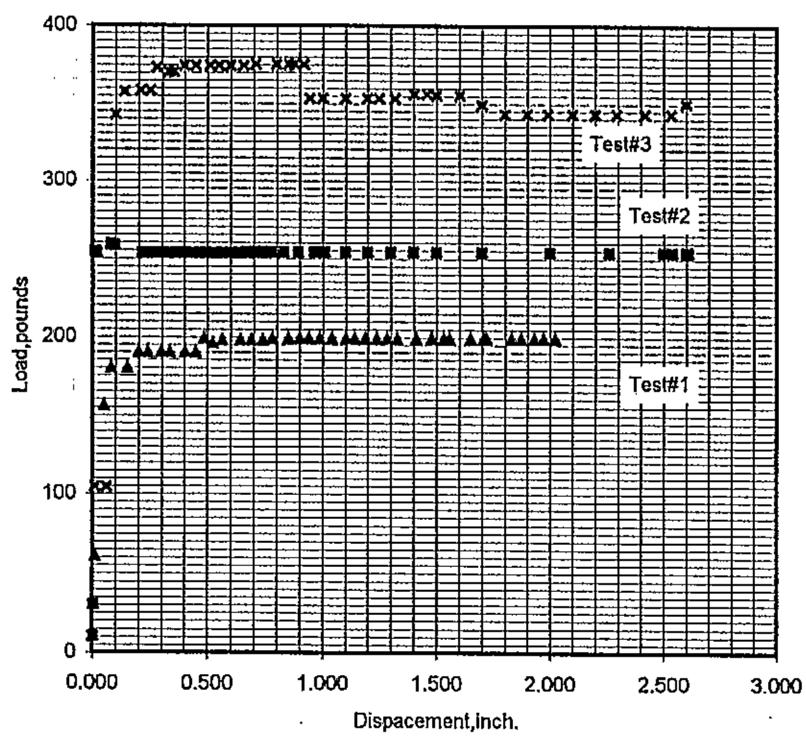
Fort Washington , Pa.

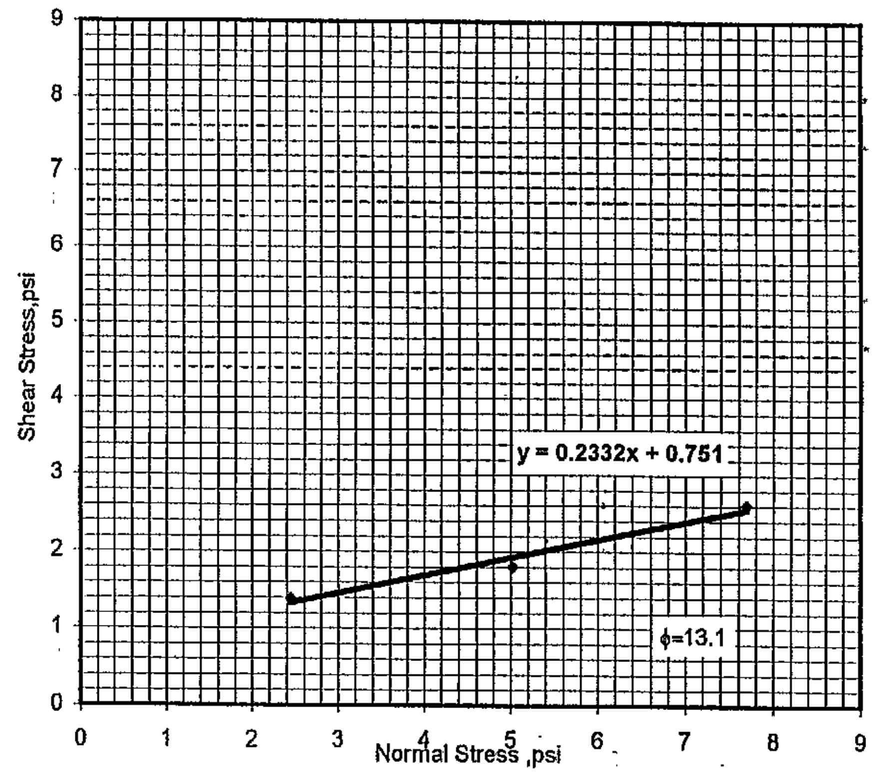
Job No. 98G022 Job Name AGC: Tonolli Superfund Site

Date 7/28/98

Geomembrane (LLDPE 60MIL) Vs GCL(Bentomat))

Test No.	Normal Stress,psi						
1	1.0						
2	3.0						
3	6.0						
4	9.0						





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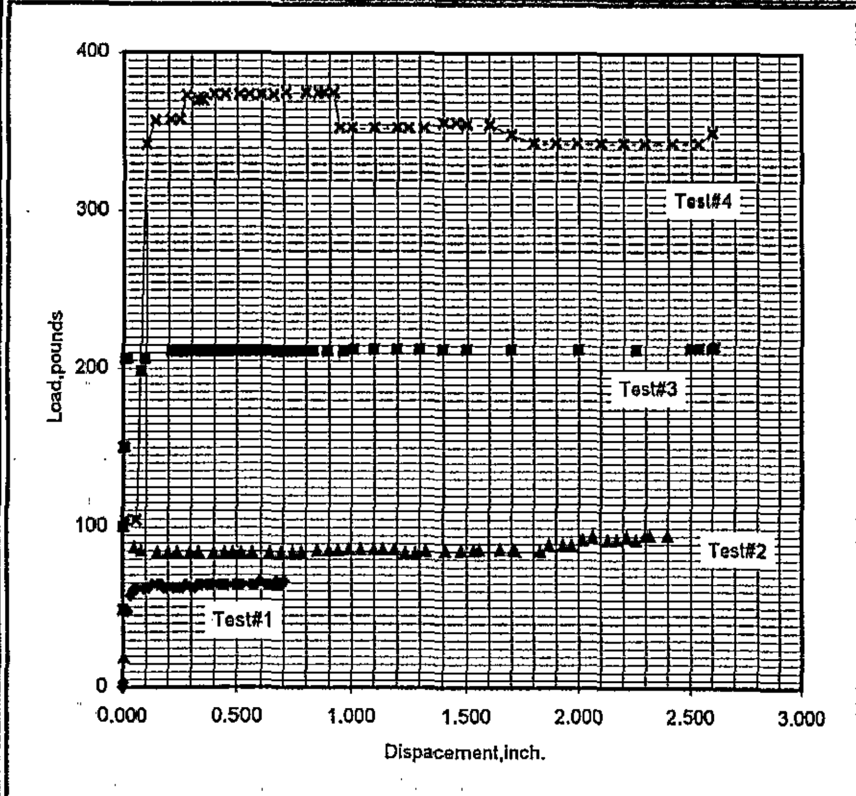
98022ds3

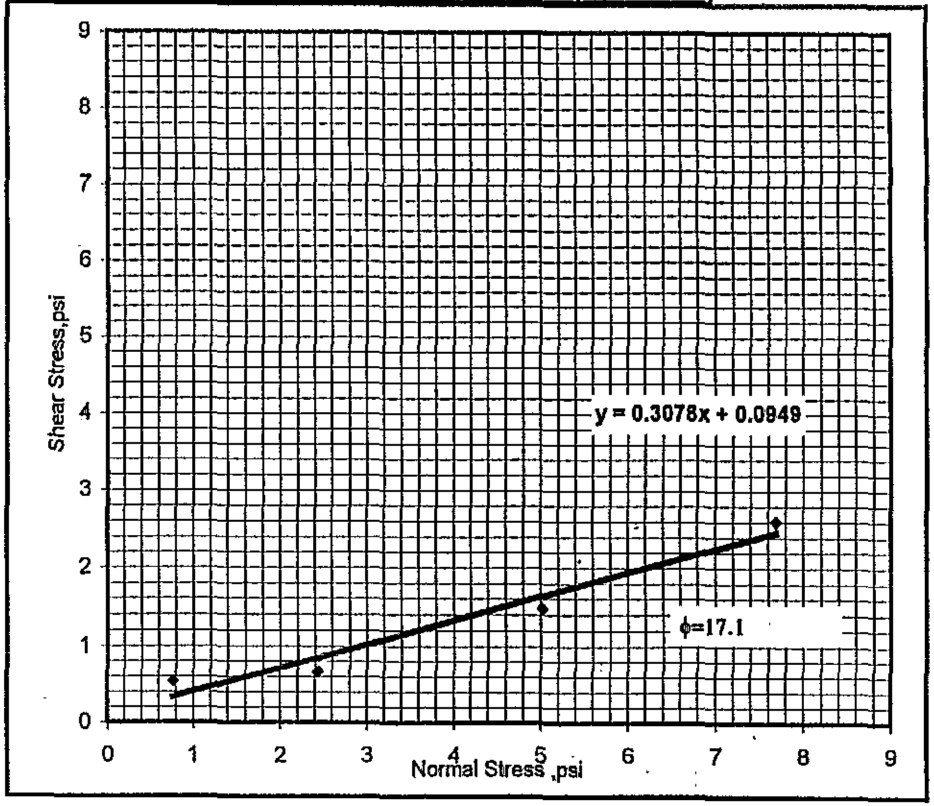
4 of 4

GeoSystems Consultants , Inc.

Fort Washington , Pa.

Job No. 98G022		Job Name AGC: Tonolli Superfund Site					
Date 7/24/98							
Geomembrane (LLDPE 60MIL) Vs Geotextile(Tensar DC3105/TG700)							
Test No.	Normal Stress,psi						
1	1.0						
2	3.0						
3	6.0						
4	9.0						





98022ds2

AR305348

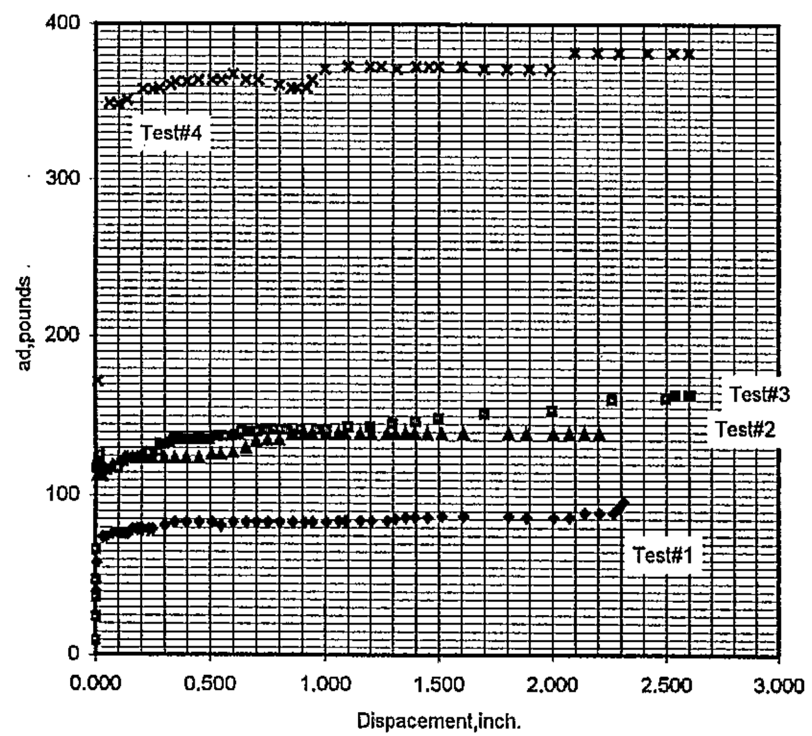
GeoSystems Consultants , Inc.

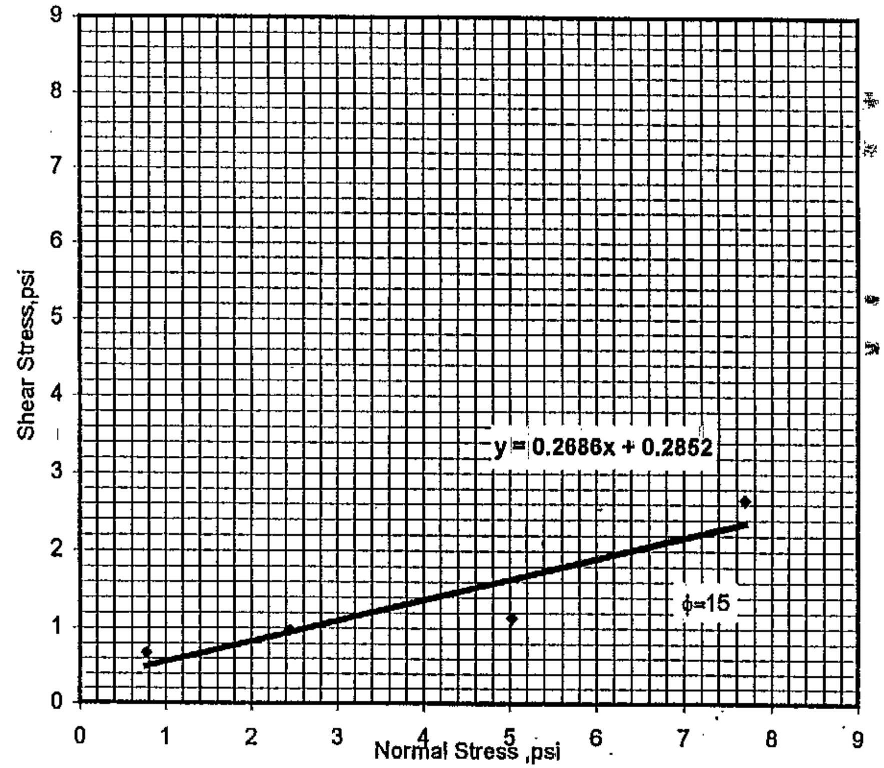
Fort Washington , Pa.

Job No.	98G022	Job Name	AGC: Tonolli Superfund Site
Date	7/24/98		

Geomembrane (LLDPE 60MIL) Vs Geonet(Tensar DC3105/TG700)

Test No.	Normal Stress,psi
1	1.0
2	3.0
3	6.0
4	9.0





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98022ds1

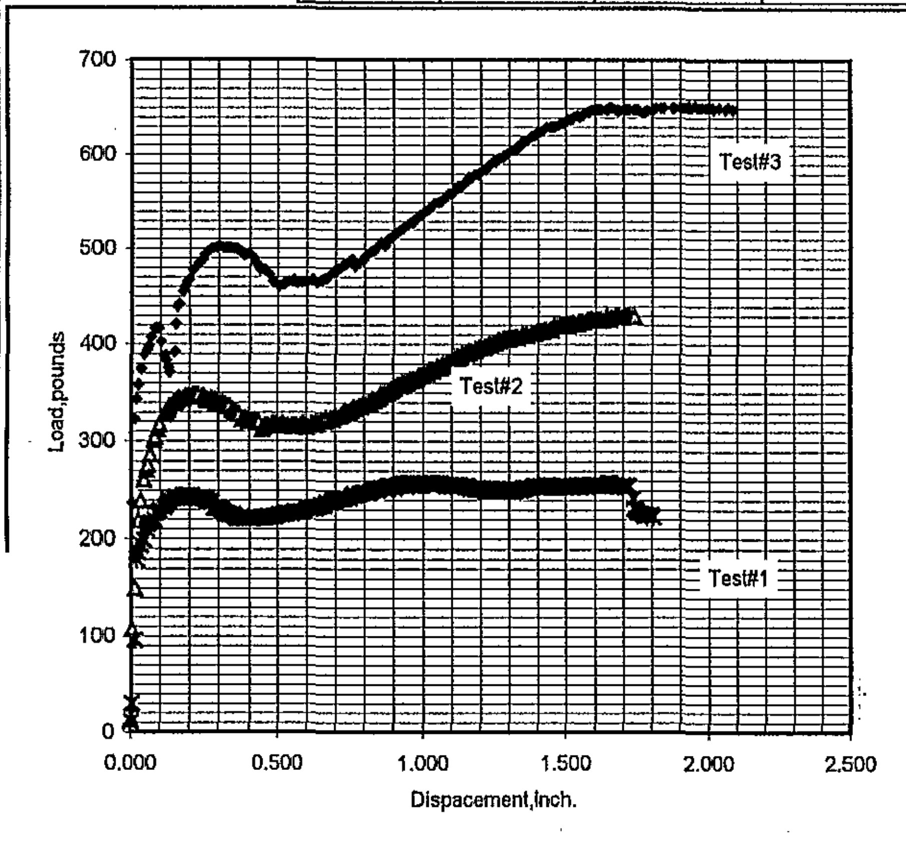
GeoSystems Consultants , Inc.

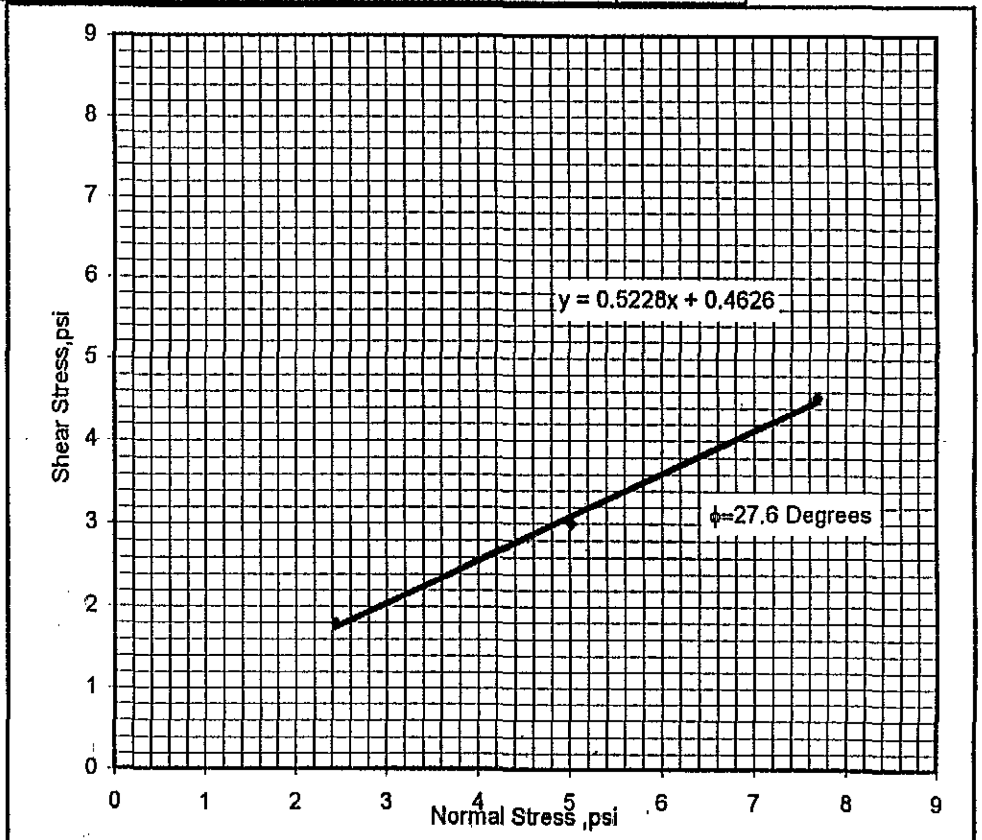
Fort Washington , Pa.

Job No.	98G022	Job Name	AGC: Tonolli Superfund Site
Date	8/3/98		

Compacted Soil (SM) Vs GCL(Bentomat))

Test No.	Normal Stress,psi
1	1.0
2	3.0
3	6.0
4	9.0





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POLY-FLEX
POLYETHYLENE GEOMEMBRANES

SMOOTH VFPE (LLDPE) GEOMEMBRANE DATA SHEET

Minimum Average Values

Property	Test Method	20 Mil	30 Mil	40 Mil	60 Mil	80 Mil
Thickness, mils	ASTM D 1593	18	27	36	54	72
Resin Density, g/cc	ASTM D 1505	0.915	0.915	0.915	0.915	0.915
Carbon Black Content, %	ASTM D 1603	2 -3	2 -3	2 -3	2 -3	2 -3
Carbon Black Dispersion	ASTM D 3015 or ASTM D 5596	A1, A2, B1 CAT.1 or 2	A1, A2, B1 CAT.1 or 2	A1, A2, B1 CAT.1 or 2	A1, A2, B1 CAT.1 or 2	A1, A2, B1 CAT.1 or 2
Tensile Properties	ASTM D 638 (Type IV Specimen @ 2 ipm)					
1. Tensile Strength at Yield, ppi		30	45	60	94	125
2. Elongation at Yield, %		13	13	13	13	13
3. Tensile Strength at Break, ppi		85	128	170	$\boxed{255}$	340
4. Elongation at Break, (2.0" G.L.) %		800	800	800	800	800
(2.5" G.L.) %		640	640	640	640	640
Tear Strength, lbs.	ASTM D 1004	11	17	22	33	44
Puncture Resistance, lbs.	FTMS 101 - 2065	26	39	52	78	104
	ASTM D 4833	34	51	68	102	136
Seam Properties	ASTM D 4437					
1. Shear Strength, ppi		29	44	58	90	120
2. Peel Strength, ppi		23 & FTB	37 & FTB	50 & FTB	75 & FTB	100 & FTB

Minimum average values, unless otherwise specified, are the average values of the required number of test specimens.
This data is provided for informational purposes only and is not intended as a warranty or guarantee.
Poly-Flex, Inc. assumes no responsibility in connection with the use of this data. These values are subject to change without notice.
NA - Not applicable.
REV. 7/97

CETCO
COLLOID ENVIRONMENTAL TECHNOLOGIES COMPANY

TECHNICAL DATA SHEET

BENTOMAT "DN" CERTIFIED PROPERTIES

MATERIAL PROPERTY	TEST METHOD	TEST FREQUENCY, ft^2 (m^2)	REQUIRED VALUES
Bentonite Swell Index[1]	ASTM D 5890	1 per 50 tonnes	24 mL/2g min.
Bentonite Fluid Loss[1]	ASTM D 5891	1 per 50 tonnes	18 mL max.
Bentonite Mass/Area[2]	ASTM D 5993	40,000 ft^2 (4,000 m^2)	0.75 lb/ft^2 (3.6 kg/m^2)
GCL Grab Strength[3]	ASTM D 4632	200,000 ft^2 (20,000 m^2)	150 lbs (660 N)
GCL Peel Strength[3]	ASTM D 4632	40,000 ft^2 (4,000 m^2)	15 lbs (65 N)
GCL Index Flux[4]	ASTM D 5887	Weekly	1×10^{-8} m^3/m^2/sec
GCL Permeability[4]	ASTM D 5084	Weekly	5×10^{-9} cm/sec
GCL Hydrated Internal Shear Strength[5]	ASTM D 5321	Periodic	500 psf (24 kPa) typical

Bentomat "DN" is a reinforced GCL consisting of a layer of sodium bentonite between two geotextiles which are needlepunched together.

Notes:

1 Bentonite property tests performed at CETCO's bentonite processing facility before shipment to CETCO's GCL production facilities.
2. Bentonite mass/area reported at 0 percent moisture content..
3 All tensile testing is performed in the machine direction, with results as minimum average roll values unless otherwise indicated.
4 Index flux and permeability testing with deaired distilled/deionized water at 80 psi (551 kPa) cell pressure, 77 psi (531 kPa) headwater pressure and 75 psi (517 kPa) tailwater pressure. Reported value is equivalent to 925 gal/acre/day. This flux value is equivalent to a permeability of 5×10^{-9} cm/sec for typical GCL thickness. This flux value should not be used for equivalency calculations unless the gradients used represent field conditions. A flux test using gradients that represent field conditions must be performed to determine equivalency. The last 20 weekly values prior the end of the production date of the supplied GCL may be provided.
5 Peak value measured at 200 psf (30 kPa) normal stress. Site-specific materials, GCL products, and test conditions must be used to verify internal and interface strength of the proposed design.

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GEONET TRANSMISSIVITY

CALCULATIONS

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GEONET TRANSMISSIVITY

The purpose of these calculations is to determine the required transmissivity of the geonet.

The peak daily rate from the HELP model based on 3% slopes is 2.1 inches/day. The longest distance from the top of the landfill to the anchor trench is on the southern slope, @ 250 feet.

Convert to cubic feet per day

$$(2.1\ \text{IN/day})\left(\frac{\text{cu ft}}{12\ \text{IN}}\right)\left(\frac{1}{\text{SQ ft}}\right) = 0.175\ \text{cu ft/day/sq ft}$$

convert to gallons per day per sq ft

$$(0.175\ \text{cuft/day})\left(\frac{7.48\ \text{gall}}{\text{cu ft}}\right) = 1.3\ \text{gal/day/sq ft}$$

convert to gallons per day per ft width of slope

$$\left(1.3\ \frac{\text{gal/day}}{\text{SQ ft}}\right)(250\ \text{ft}) = 325\ \text{gal/day/ft}$$

convert to gallons per min per foot

$$(325\ \text{gal/day/ft})\left(\frac{\text{day}}{1440\ \text{min}}\right) = \underline{\underline{0.225\ \text{gal/min/ft}}}$$

The proposed Drainage composite DC 3205 has a Transmissivity of $\approx 4\times10^{-4}\ m^2/\text{sec}$. (See attached product curve at 12 kPa)

$$4\times10^{-4}\frac{m^2}{\text{sec}} \times 10.76\frac{FT^2}{m^2} \times 1\ \text{FT width} \times \frac{7.48\ \text{gal}}{FT^3} \times \frac{60\ \text{sec}}{\text{min}} = 1.93\ \text{gal/min/FT}$$

SHEET 1 OF 5	PROJECT NO. 96-249-79	PROJECT NAME Tonolli Corporation Superfund Site
BY TJDT	DATE 7/27/98	DESCRIPTION TRANSMISSIVITY CALCS.
CHK. BY	DATE	

CHECK Transmissivity Against Cover Soil Permeability.

Cover Soil is a sandy loam, to sandy clay-loam
Permeability is expected to be on the order of
1×10^{-4} cm/sec ∴ flow from the cap soil could be:

$$1 \times 10^{-4} \frac{cm}{sec} \times \frac{1\ in}{2.54\ cm} \times \frac{1\ FT}{12\ in} \times 1\ FT\ width \times \frac{60\ sec}{min} \times \frac{7.48\ gal}{FT^3}$$

$\times$ 250 FT of slope length = 0.37 gal/min/ft
(assuming a gradient of 1)

∴ Geocomposite capacity of 1.93 gal/min/ft is adequate to handle peak daily rate from HELP model of 0.225 gal/min/ft. or the available infitration from the cover soil of 0.37 gal/min/ft

File
Tonelli

Evergreen Technologies 5775-B Glenridge Dr., Suite 450, Atlanta, GA 30328-5363

FAX

Date: 7/27/98

Number of pages including cover sheet: ~~3~~ 5

To: Todd Trautman

Phone:

Fax phone: 610-558-2620

CC:

From: Brenda Reynolds

Sales Coordinator

Phone: 404-250-1290

Fax phone: 404-705-9650

REMARKS: ☐ Urgent ☐ For your review ☐ Reply ASAP ☐ Please comment

Todd,

Enclosed is the DC4205/TG700 transmissivity curve and MPDS. DC 3205 transmissivity information is only available for 6 oz. textile. Transmissivity for TG700 should be similar to this data. Please call if you have any other questions.

Thanks!

AR305356

EVERGREEN TECHNOLOGIES, INC. GEOCOMPOSITE DC4205 - DOUBLE-SIDED WITH TG700 GEOTEXTILES

Geocomposite DC4205 shall consist of ETI NS1405 geonet bonded on both sides to 8 oz. non-woven polypropylene geotextiles (continuous filament). The geocomposite shall have a high compressive strength in order to ensure maximum flow capacity under high confining pressures. The bonding process shall not introduce adhesives or other foreign products. The geocomposite shall be resistant to all forms of biological or chemical degradation normally encountered in a soil environment. *The geocomposite shall be made from the geonet and geotextile products whose property requirements are listed below.* The resin used in the production of the geonet shall be a minimum 97% virgin polyethylene with a melt flow range between 0.1 to 1.0 grams/10 min (per ASTM D1238) and a density range of 0.932 to 0.963 grams/cc (per ASTM D792 or D1505). The geocomposite is delivered to the job site in roll form with each roll having unique identification and QA traceability.

PROPERTY	APPROVED TEST METHODS	UNITS	VALUE	SPECIFICATION	TEST FREQUENCY
QA CERTIFIED TEST PARAMETERS					
GEOTEXTILE PROPERTIES					
• Grab Tensile Strength	ASTM D 4632	N (lbs)	956 (215)	MARV	Per ASTM D4759
• AOS	ASTM D 4751	mm (US Std. Sieve)	0.215 (70)	MARV	
• Mass/Unit Area	ASTM D 5261 (or ASTM D 3776)	g/m^2 (oz/sy)	271 (8.0)	MARV	
• Water Permeability	ASTM D 4491	cm/sec	0.3	MARV	
• Water Flow Rate	ASTM D 4491	$m^3/sec/m^2$ (gpm/ft^2)	0.07 (100)	MARV	
• U.V. Resistanace	ASTM D 4355	%	70		
CORE NET PROPERTIES (97% minimum virgin polyethylene resin with 2-3% carbon black)					
• MD Ultimate Tensile Strength	ASTM D 5035	kN/m (ppi)	8.4 (48)	MARV	50,000 SF
• Thickness	ASTM D 5199	mm (in)	5.0 (200)	MARV	
• Carbon Black	ASTM D 4218 (OR ASTM D 1603)	(% weight)	(2.0)	MARV	
FINISHED GEOCOMPOSITE PROPERTIES					
• Peel Adhesion	ASTM F 904 (modified)[2]	g/in	454	MARV	50,000 SF
• Transmissivity[3] normal pressure = 15000psf; i=1.0	ASTM D 4716 metal plate/composite/metal plate	m^2/sec (E-04)	1.0	MARV	200,000 SF (or per project req.)
• Geotextile overlap at edges and unbonded area		mm (in)	75 (3.0)	minimum	Each roll
• Roll Length		m (ft)	68.6 (225)	minimum	
• Roll Width		m (ft)	4.0 (13.0)	minimum	
PRODUCT INFORMATION					
• Roll Weight		kg (lbs)	370 (815)	typical	N/A
• Roll Diameter		m (in)	0.8 (31)	typical	
• Core I.D.		mm (in)	100 (4)	nominal	
Packaging and Labeling					
• *Black polyethylene bag secured with nylon ties. Bag thickness*		*mm* (mils)	*0.12* (6.0)	*nominal*	
• Labeling: Product code,geotextile type, roll dimensions, finished product Lot and roll number.					

Notes

[1] MARV is defined as the one-sided 97.5% confidence limit obtained through long-term production data (mean - 2*standard deviation). It represents the minimum allowable sample roll average for each specific test.

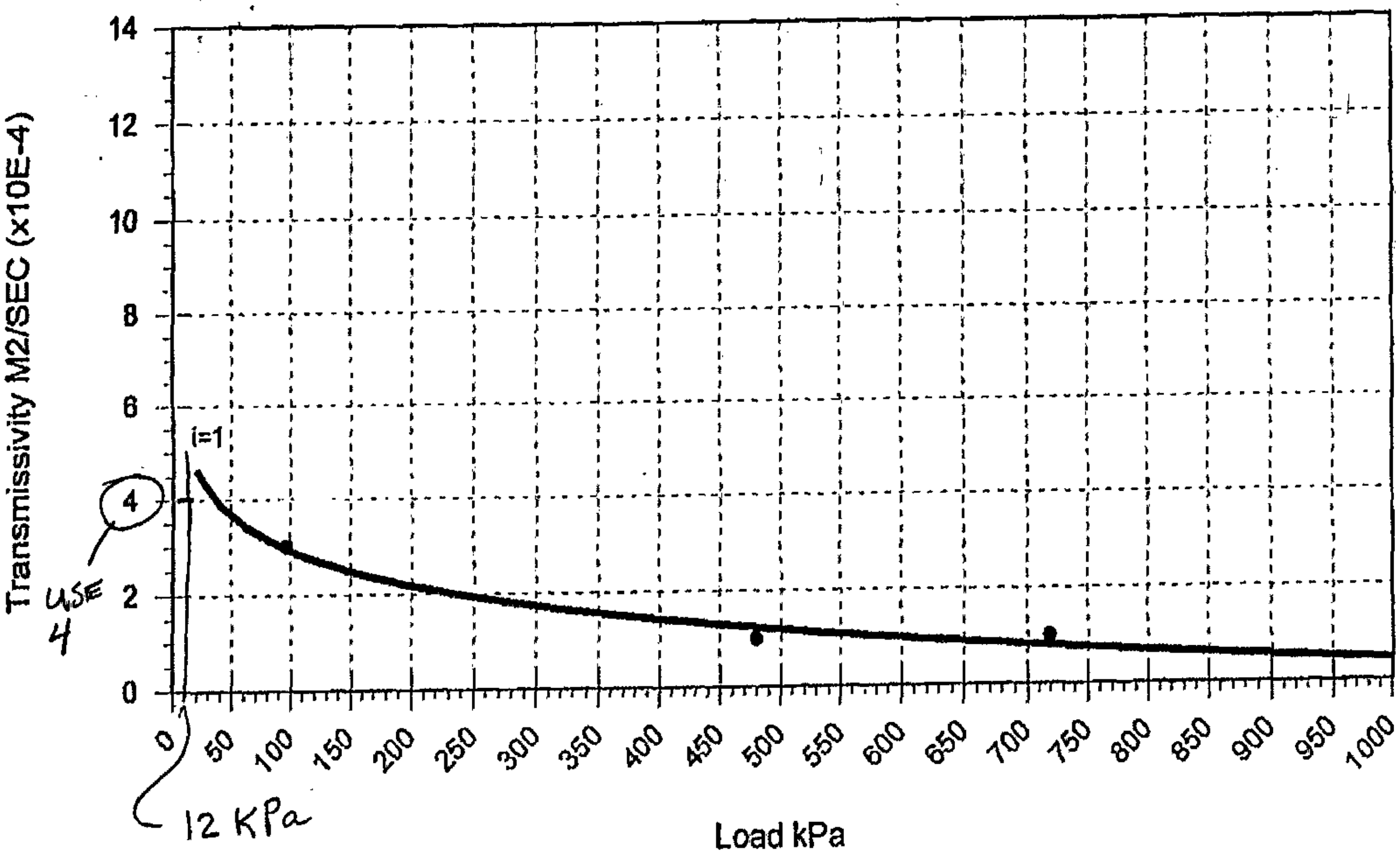
[2] Peel adhesion ASTM F 904; 2 inch wide strip. Reported value per specimen is average of 5 highest peaks.

[3] Transmissivity: Results reported by ETI are based on standard index test conditions. Actual performance is dependent upon site specific conditions. Please contact Evergreen Technologies, Inc.for site specific transmissivity testing.

Evergreen Technologies, Inc.
5775B Glenridge Drive, Suite 450
Atlanta, Georgia 30328-5363
1-800-984-9784

Transmissivity Curves
The Tensar Corporation

Geocomposite
Product: DC4205E88
Net: 1405
Geotextile - Top: 8 oz, Bottom:8 oz



6 July 1997

1000 PSF = 48 kPa

WESTERN EMBANKMENT SLOPE

GCL STABILITY CALCULATIONS

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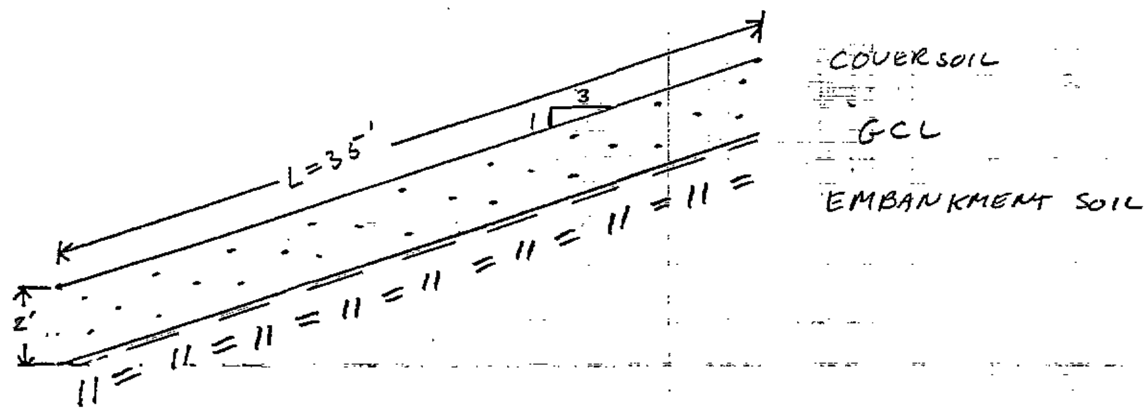
ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

NORTH & WEST EMBANKMENT GCL STABILITY CACULATIONS.

THE FOLLOWING CACULATIONS DETERMINE THE FACTOR OF SAFETY FOR INTERFACE FRICTION BETWEEN THE GCL AND THE UNDERLYING SOIL WITHOUT PUTTING THE GCL IN TENSION.



- COVER SOIL.

PARAMETERS	TYPICAL VALUES	VALUES USED
WET DENSITY, γ	110-130 pcf	130 pcf
COHESION, C SAT	300 psf	
C DRY	1050 psf	200 psf
∠ INTERNAL FRICTION φ	26-33°	26°
INTERFACE FRICTION		27.6° (ACTUAL TEST DATA)

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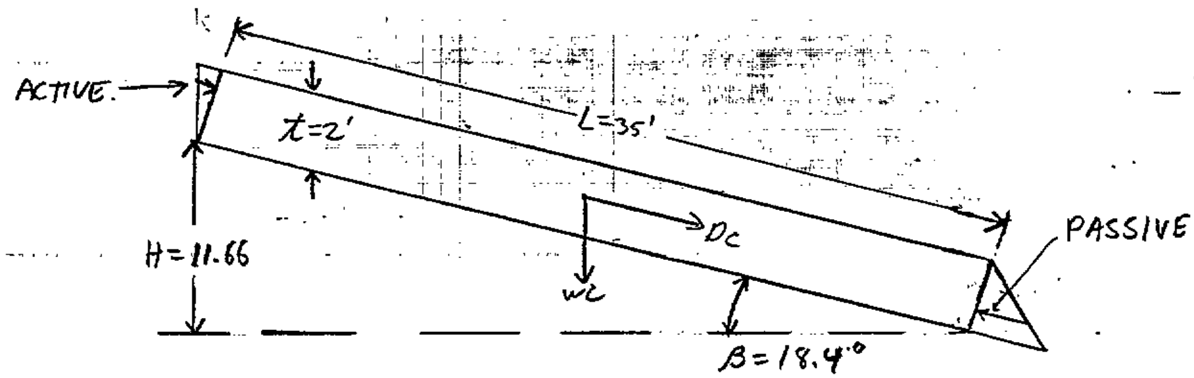
BY CJN DATE 7-23-98 DESCRIPTION W. EMBK. GCL STABILITY.
CHK. BY TML DATE 8-5-98

TYPICAL CROSS-SECTION



WEIGHT OF CENTRAL ZONE (wc)

$$W_c = L \cdot t \cdot \gamma = 35'(2')(130\,pcf) = 9{,}100\ lb/ft.\ width$$

DRIVING FORCE (Dc)

$$D_c = W_c \sin\beta = 9100\ lb/ft\ width\ (\sin 18.4°) = 2873\ lb/ft.\ width$$

ACTIVE FORCE $= F_A$.

$$F_A = \tfrac{1}{2} k_a \gamma H^2 \qquad k_a = \frac{1-\sin\phi}{1+\sin\phi} \Rightarrow \frac{1-\sin 26°}{1+\sin 26°} = .39$$

$$F_A = .5(.39)130(2')^2 = 101.4\ lb/ft\ width$$

PASSIVE FORCE $= F_P$

$$F_P = \tfrac{1}{2} k_p \gamma H^2 \qquad k_p = \frac{1+\sin\phi}{1-\sin\phi} \Rightarrow \frac{1+\sin 26°}{1-\sin 26°} = 2.56$$

$$F_P = .5(2.56)\,130\,pcf(2')^2 = 665.6\ lb/ft\ width$$

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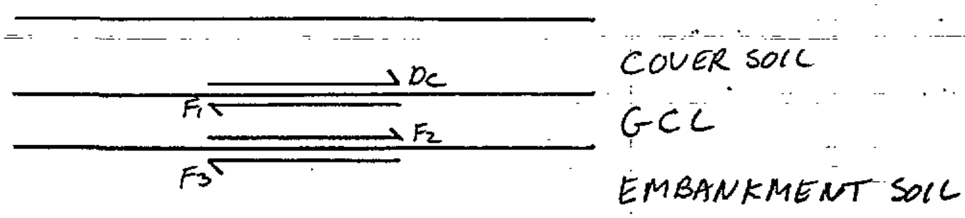
"Engineering for the Environment"™

GCL AND SOIL INTERFACES

$\beta = 18.4°$

$\delta_1 = ?$

$\delta_2 = \delta_1 = ?$



FOR A FACTOR OF SAFETY OF 1.5 FOR THE INTERFACE BETWEEN THE GEOSYNTHETIC CLAY LINER WITH THE COVER SOIL OR THE EMBANKMENT SOIL AN INTERFACE FRICTION ANGLE, δ_1 & δ_2 MUST BE DETERMINED

THE EQUATION FOR FACTOR OF SAFETY AT THE INTERFACE IS:

$$FS = \frac{R}{D}$$

WHERE: (SEE SHEET 2)

DRIVING FORCES (D) = $D_c + F_A \Rightarrow 2873 + 101.4 = 2974.4$ lb/ft width

RESISTING FORCES (R) = $F_1 + F_P \Rightarrow W_c(\cos\beta)\tan\delta_1 + 665.6$

$$9{,}100(\cos 18.4)\tan\delta_1 + 665.6$$

$$R = 8{,}634.7(\tan 27.6) + 665.6$$

δ_1 IS INTERFACE FRICTION ANGLE BETWEEN GCL AND COVER SOIL.

$$R = 5180 \text{ lb/FT}$$

$$FS = \frac{R}{D} = \frac{5180}{2974} = 1.7 \Rightarrow OK$$

 PROJECT NO. 96-248-79 PROJECT NAME TONOLLI

BY CJN DATE 7-23-98 DESCRIPTION W. EMBK GCL STABILITY

CHK. BY TML DATE 8-5-98

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"Engineering for the Environment"™

CHECKING INTERNAL SHEAR STRENGTH OF GCL

Driving Force = D = 2,974.4 $^{lb}/_{ft.}$ width

The internal shear strength of the GCL is 500 $^{lb}/_{ft^2}$

∴ The internal resistance = 500 $^{lb}/_{ft^2}$ x 35 ft slope length

= 17,500 lb/ft. width

2,974.4 << 17,500

∴ GCL will carry load through to underlying soil

SHEET 1 OF 1 PROJECT NO. 96-248-79 PROJECT NAME Tonolli
BY TDT DATE 8-5-98 DESCRIPTION Cap Stability
CHK. BY ______ DATE ______

SLOPE STABILITY CALCULATIONS

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SLOPE STABILITY ANALYSIS

1.0 INTRODUCTION

Presented herein is the slope stability analyses performed on the 3:1 (horizontal:vertical) final embankment slopes proposed for the closure of the existing landfill at a maximum cap slope of 20%. This analysis includes both the static and dynamic loading conditions. Slope stability calculations included in the Final Design demonstrated that the proposed 3:1 embankment slopes and the gabion wall with a maximum cap slope of 10% achieved the require 1.5 factor of safety for both the static and dynamic loading conditions. These analyses were modified to include a 20% cap slope.

Previous slope stability analyses were performed as part of the Technical Memorandum and Preliminary Design Report which were submitted to the Regulatory Agencies for review and comment. The results of these previous analyses are discussed briefly below.

1.1 Technical Memorandum

An analysis of the stability of the existing conditions of the landfill embankments was included in the Technical Memorandum submitted as a draft on October 4, 1996. This analysis concluded that the existing exterior slopes of the landfill along the east side, the south side, and the southern portion of the west side will require modifications to comply with the minimum factors of safety (FS = 1.5 Static and Dynamic) required by the Pennsylvania hazardous waste regulations. The existing slopes are as steep as 1.5:1 (horizontal:vertical).

1.2 Preliminary Design

Exterior embankment slopes of 4:1 (horizontal:vertical) with geogrid reinforcement were proposed for the closure of the landfill in the Preliminary Design Report, dated February 3, 1997. The slope stability analysis of this embankment design was performed for both the static and dynamic loading conditions considering the highest exterior slope which occurs at the south end of the landfill. In addition, an analysis was performed to determine the effect of a hypothetical high groundwater condition on embankment stability. The proposed 4:1 slope (with and without the high groundwater condition) achieved the minimum factors of safety for both Static and Dynamic Conditions as required by the Pennsylvania hazardous waste regulations, but would encroach on the existing railroad right-of-way along the southern portion of the Site.

During a series of meetings with the USEPA, PADEP, USACE, Tonolli Site RD/RA Steering Committee, and AGC, the issues of slope stability and the feasibility of filling within the existing railroad right-of-way were discussed. The 4:1 embankment slope proposed in the Preliminary Design was primarily the result of shallow surface failures calculated during the slope stability analysis of various embankment slopes (2:1 to 3.5:1) under the dynamic loading condition. It was agreed that the shallow surface failures will not pose a threat to the overall embankment stability; and therefore, the design of steeper slopes would be investigated during the subsequent design submissions in order to avoid filling within the existing railroad right-of-way. This would be done by defining failure as deep soil movement penetrating the "critical zone".

2.0 DEFINITION OF CRITICAL ZONE OF SLOPE FAILURE

During the June 24, 1997 meeting with the USEPA, PADEP, USACE, Tonolli Site RD/RA Steering Committee, and AGC, AGC presented a definition of a "critical zone" to be used for the slope stability analysis to be performed for the Final Design. This "critical zone" was defined as the soil mass, either embankment fill or natural subsurface soils, situated deeper than four feet beneath the embankment surface. This "critical zone" was approved in concept by the USEPA, PADEP, and USACE.

The minimum four foot depth limit of the "critical zone" was selected because slip surfaces located below this depth could result in significant embankment reconstruction if they occur. A failure of this depth could also potentially expose the landfill materials. However, this is considered unlikely, since a failure, if it was to occur, will most likely be a slow gradual movement of soil (creep) extending from the face of the slope to the liner, and not a sudden catastrophic failure. Catastrophic failures are typically associated with landslides associated with a build-up of excess pore water pressures from excessive precipitation or a high groundwater table, from a unique geologic condition, or from liquefaction of sands during an earthquake. These conditions are not present or expected within the Tonolli landfill embankment.

A slope failure located above the "critical zone" will cause sloughage of the soil cap materials (i.e., topsoil and select fill), and will only stress the liner and geosynthetic cap components within the anchorage area. The resulting damage of such a failure will be minor and can be readily corrected when detected.

3.0 METHOD OF ANALYSIS

The slope stability analysis was performed using:

> PC-Slope, SLOPE/W Software (Version 3.02)
> Copyright 1991, 1995
> Geo-Slope International Ltd.
> Calgary, Alberta, Canada

PC-Slope, SLOPE/W is a software product that uses the limit equilibrium theory to solve for the factor of safety of earth and rock slopes against failure. The limit equilibrium theory involves the cutting of slip surfaces (i.e., wedges of soil and/or rock) through an earth and/or rock slope and determining the resisting and overturning forces, and moments on that wedge of soil/rock. These moments and forces are compared to find factors of safety against failure.

Both the Bishop's Simplified and the Janbu's Simplified method were used for these analyses. Both methods consider normal forces but no shear forces between soil slices. The Bishop's Simplified calculates only moment equilibrium and the Janbu's Simplified calculates only force equilibrium. The results of these analyses were similar, and for simplicity only, the results of the Bishop's analysis are reported.

The PC-Slope software requires the following data for the analysis of slope stability:

- Slope geometry.
- Soil properties of the slope (i.e., total unit weight, internal friction angle, and cohesion).

Other factors such as the groundwater table, pore water pressures, seismic loads, anchor loads (such as geogrid reinforcement), and applied loads can also be entered into the software to model the subsurface conditions.

4.0 CAP AND EMBANKMENT GEOMETRY ASSUMPTIONS

The slope stability analyses were performed on the proposed 3:1 (horizontal:vertical) outer slope embankment geometry where the proposed slope will be the highest. The existing embankment slope is approximately 1.5:1 (horizontal:vertical). Filling will be performed along the outer slope to achieve the proposed 3:1 slope. The construction of a gabion wall will be performed along a portion of the southern embankment to prevent filling within the existing railroad right-of-way.

Based on available topographical data and the proposed design, this critical slope will occur at the southeast corner of the existing landfill. Summarized below are assumptions used in the analyses regarding the landfill geometry. A cross-section of the existing landfill and proposed cap are shown on Figure 1 located in Attachment 1.

1. The as-built construction drawings indicate that the interior buried slopes of the southern embankment are 3 (horizontal): 1 (vertical). A sensitivity analysis was performed using the PC-Slope software by varying the slope of this interior face between a 3:1 and 1:1 in order to determine the effect of this slope on the stability calculations. No effect was observed. This is reasonable since the interior face is no longer acting as a slope, because the landfill is filled to about elevation 1022.

2. The southern embankment is composed of two distinct fill materials: a reddish brown

silty coarse to fine sand with gravel, cobbles, and trace to some clay; and a black silty coarse to fine sand with gravel (mine spoils) underlain by natural soils. The delineation of these soil strata in and beneath the southern embankment (i.e., fill, mine spoil fill, and natural soils) is based on the test borings performed on top, and around the base of the landfill embankment during the Pre-Design Investigation. The presence of these two fill materials is consistent with the fact that the landfill was constructed in two phases. The test borings suggest that the liner at the bottom of the landfill is supported on less than 3 feet of mine spoil fill. However, to simplify modeling, it was assumed that the bottom of the landfill is supported on natural soils. This simplifying assumption does not affect the slope stability calculations since the critical failure plane does not extend to the bottom of the landfill.

3. To simplify the computer modeling, the future fill which will be placed along the exterior slopes was assumed to be the same as the existing embankment materials (i.e., the two fill strata were extended outward until a 3:1 slope was achieved). The minimum physical properties that were specified in the Contract Documents for the future fill material were, at a minimum, similar to those of the existing embankment fill (described below). The future fill will be benched into the existing slope, will be placed in a controlled/compacted condition, and will likely have strength parameters in excess of those assigned to the existing embankment materials.

4. The final landfill cap will be graded down toward the embankments with a maximum slope of 20%.

5. The groundwater table in the analytical model are the elevations of observed groundwater in December, 1996.

5.0 SOIL PROPERTIES

The following physical properties were assigned to the five soils shown on Figure 1 included in

Attachment 1. These physical properties are based on correlations of grain-size analysis, moisture content, Standard Penetration Resistance (SPR) data from the test boring sampling, and accepted engineering references. The test boring logs and the result of the grain size analyses are included in Attachment 2.

The pre-design field investigation program originally proposed in the RD Work Plan included the retrieval of relatively undisturbed samples of the embankment materials via shelby tubes, if possible. Triaxial tests were proposed to be performed on these undisturbed samples to determine unit weight and shear strength parameters (i.e., internal friction angle and cohesion) for input into the PC-Slope software. However, due to the cobbles and boulders in the upper embankment material and the granular nature of the mine spoils, undisturbed samples could not be obtained.

5.1 Cap and Additional Waste Materials

Soil Classification:		SM - SC
Total Unit Weight	=	120 pcf
Internal Friction Angle	=	30°
Cohesion	=	0

(from Navac, DM-7.2, 1982 and Simplified Design of Building Foundations included in Attachment 3).

SM - SC soils are commonly specified for cap construction and the on-site soils to be placed in the landfill have been classified as SM soils based on grain size analysis performed during the Pre-Design Investigation. The cap and additional waste soils will be placed in a controlled compacted manner; therefore the unit weight and friction angle of the dense fill will be on the higher end of the range shown on Table 1, Typical Properties of Compacted Soils (Navac, DM-7.2, 1982, page 7.2-39) and in Table 2.5 of Simplified Design of Building Foundations.

5.2 Existing Waste

Soil Properties:

Total Unit Weight	=	110 pcf
Internal Friction Angle	=	18°
Cohesion	=	0

These physical properties were assigned based on geotechnical knowledge of similar waste materials. They represent relatively conservative properties.

5.3 Embankment Fill

Soil Classification: SM - SC with cobbles and boulders

Total Unit Weight	=	125 pcf
Internal Friction Angle	=	32°
Cohesion	=	200 psf

(from Navac, DM-7.2, 1982 and Simplified Design of Building Foundations included in Attachment 3).

The above properties are based on Standard Penetration Resistance (SPR) values in conjunction with field classification, moisture content, and grain size analysis. These soils were placed in a controlled compacted manner; therefore the unit weight and friction angle will be on the higher end of the range as shown on Table 1, Typical Properties of Compacted Soils (Navac, DM-7.2, 1982, page 7.2-39). The SPR values of this material are generally between 10 bpf and 30 bpf. Based on the Simplified Design of Building Foundations, the dry unit weight of SM - SC soil at this consistency is typically on the order of 115 pcf. Laboratory testing performed during the Pre-Design Investigation indicates that the moisture content of this material is on the order of 10 percent. Therefore the total unit weight is on the order of 125 pcf.

5.4 Existing Mine Spoil Fill

Soil Classification: GM

Total Unit Weight	=	135 pcf
Internal Friction Angle	=	34°
Cohesion	=	0

(from Navac, DM-7.2, 1982 and Simplified Design of Building Foundations included in Attachment 3).

The above properties are based on Standard Penetration Resistance (SPR) values in conjunction with field classification, moisture content, and grain size analysis. These soils were placed in a controlled compacted manner; therefore the unit weight and friction angle will be on the higher end of the range as shown on Table 1, Typical Properties of Compacted Soils (Navac, DM-7.2, 1982, page 7.2-39). The SPR values of this material are generally between 10 bpf and 30 bpf. Based on the Simplified Design of Building Foundations, the dry unit weight of GM soil at this consistency is typically on the order of 115 pcf. Laboratory testing performed during the Pre-Design Investigation indicates that the moisture content of this material is on the order of 17 percent. Therefore the total unit weight is on the order of 135 pcf.

5.5 Natural Soils

Soil Classification: SM - SC with cobbles and boulders

Total Unit Weight	=	132 pcf
Internal Friction Angle	=	33°
Cohesion	=	200 psf

(Simplified Design of Building Foundations included in Attachment 3).

These properties are based on Standard Penetration Resistance (SPR) values in conjunction with field classification and grain size analysis. The SPR values of this material are well over 30 bpf. Based on the Simplified Design of Building Foundations, the dry unit weight of SM - SC soil at this consistency is typically on the order of 120 pcf. Due to the groundwater table, the moisture content of this material will likely vary. Assuming a natural moisture content of 10 percent, (same moisture content of these soils which were used for embankment fill), the total unit weight of this material is likely about 132 pcf.

6.0 DYNAMIC LOADING

The dynamic condition evaluates slope stability under a horizontal force created by seismic or earthquake accelerations. The PC-Slope software models these effects by defining a seismic coefficient. The software applies a horizontal force at the centroid of each slip surface equal to the slice weight multiplied by the user-defined seismic coefficient.

The seismic coefficient entered into PC-Slope is analogous to the Effective Peak Velocity-Related Acceleration (A_v) assigned to seismic zones in the United States. A seismic coefficient of 0.1 was used for the stability analysis and was obtained from the BOCA National Building Code/1990. The map of seismic zones and Effective Peak Velocity-Related Acceleration (A_v) for the contiguous 48 states is provided in Attachment 3.

7.0 FINDINGS AND CONCLUSIONS

Results for the static and dynamic loading condition are provided on Figures 2 through 9 of Attachment 1. For each condition, multiple radii within the landfill embankment and natural soils were analyzed at each grid node (a total of 4,096 radii) to determine the slip surface with the lowest factor of safety. The radius with the minimum factor of safety for each grid node was determined. The minimum slip surface (identified by node coordinate and radii) are shown in the figures provided to aid in our discussion of results which is provided below. Additional slip surfaces through the proposed cap are also provided for the landfill embankment and gabion wall.

7.1 3:1 Embankment Slope

7.1.1 Static Condition

A minimum Factor of Safety of 2.1 (2.124 rounded off to the nearest tenth) was calculated for static condition as shown on Figure 2.

7.1.2 Dynamic Condition

A minimum Factor of Safety of 1.6 (1.584 rounded off to the nearest tenth) was calculated for the dynamic condition as shown on Figure 3. Additional slip surfaces through the landfill are provided for the dynamic condition on Figures 4 and 5.

7.2 Gabion Wall Design

A global slope stability analysis using PC-Slope, SLOPEW Software for the proposed landfill embankment that includes the proposed 3:1 slope and the highest proposed section of the wall was performed. As recommended in the October 15, 1997 comments to the Pre-Final Design Report, the soil parameters for the mine spoil fill located beneath the proposed gabion wall were reduced to more conservative values. The following soil parameters were used for the mine spoil fill beneath the gabion wall. The physical property parameters used in the analysis for all other soils are those described in Section 5.0, "Soil Properties".

Total Unit Weight	=	130 pcf
Internal Friction Angle	=	30
Cohesion	=	0

The results of these analyses are described briefly below.

7.2.1 Static Condition

A minimum Factor of Safety of 1.9 (1.865 rounded off to the nearest tenth) was calculated for static condition as shown on Figure 6.

7.2.2 Dynamic Condition

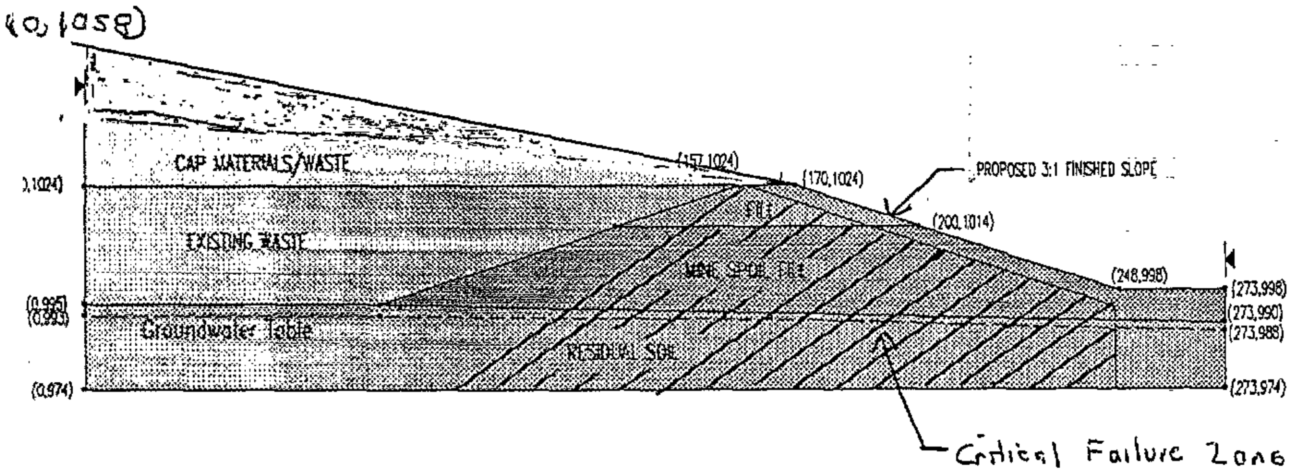
A minimum Factor of Safety of 1.5 (1.474 rounded off to the nearest tenth) was calculated for the dynamic condition as shown on Figure 7. Additional slip surfaces through the landfill are provided for the dynamic condition on Figures 8 and 9.

ATTACHMENT 1

TO

SLOPE STABILITY CALCULATION

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)



LEGEND

(0,1041) = (Horizontal Station, Elevation)

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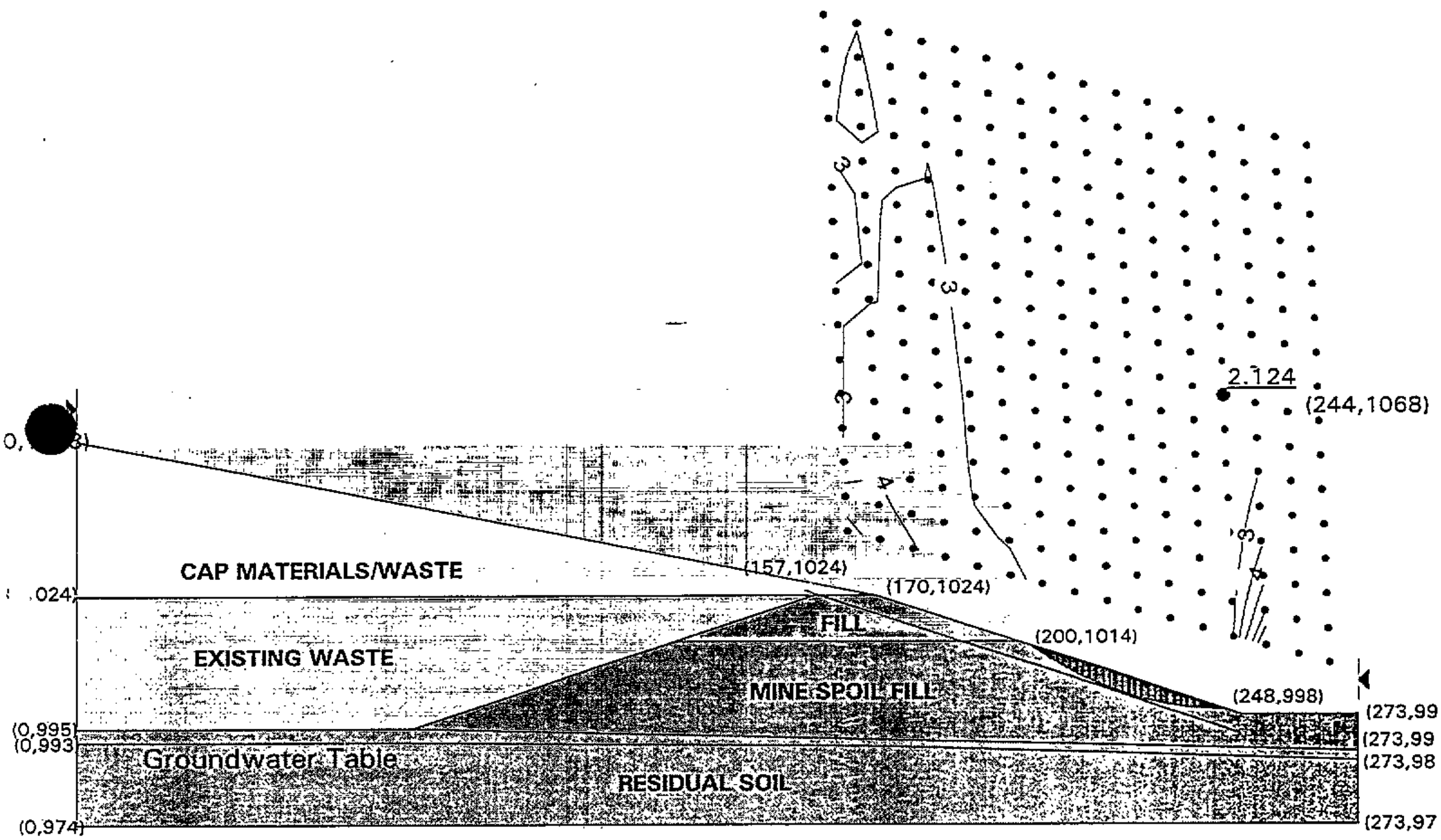
Figure 1

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)
STATIC CONDITION

(Minimum Factor of Safety)



LEGEND

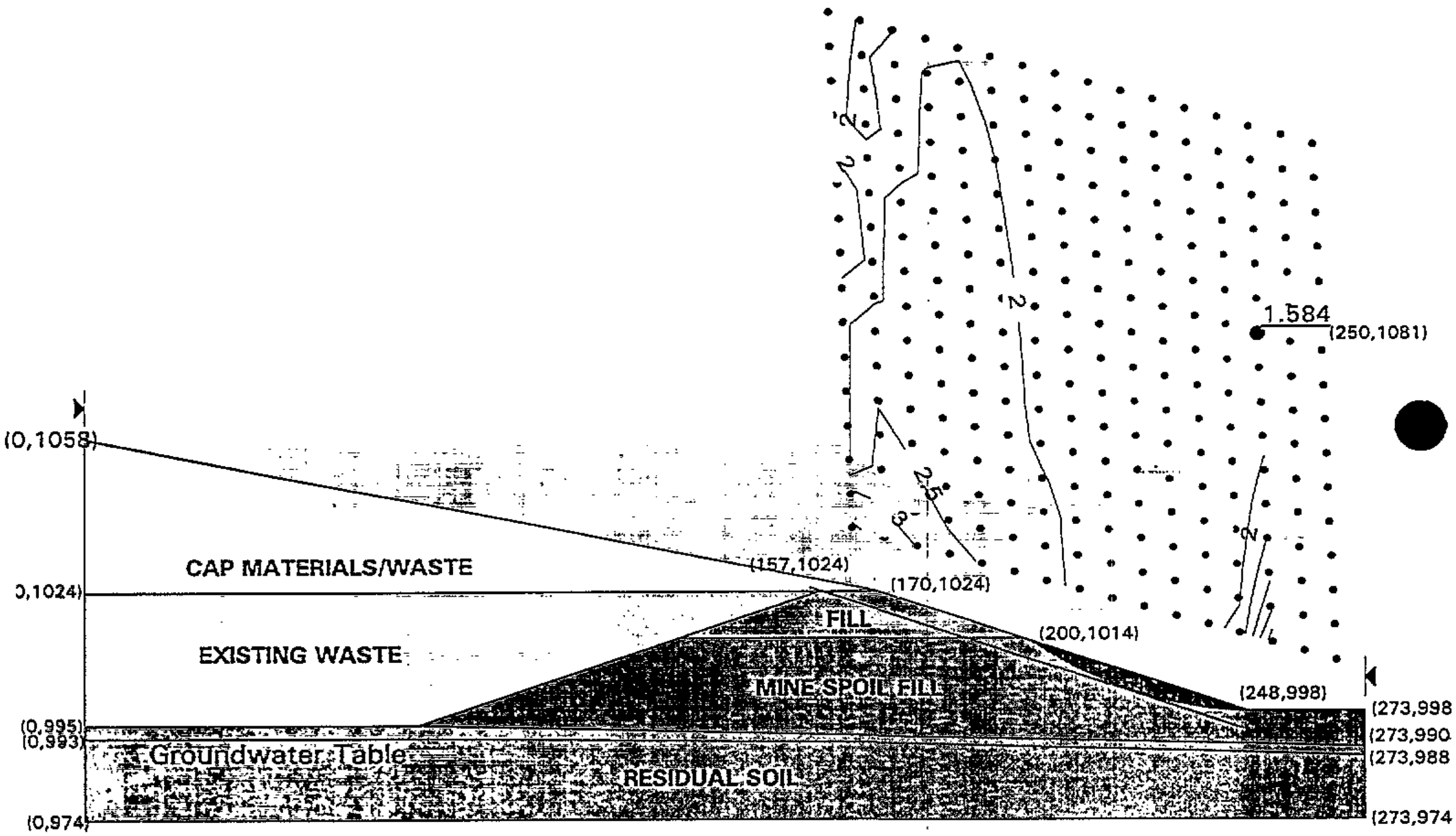
(0,1041) = (Horizontal Station, Elevation)

AR305379

Figure 2

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPE) DYNAMIC CONDITION

(Minimum Factor of Safety)



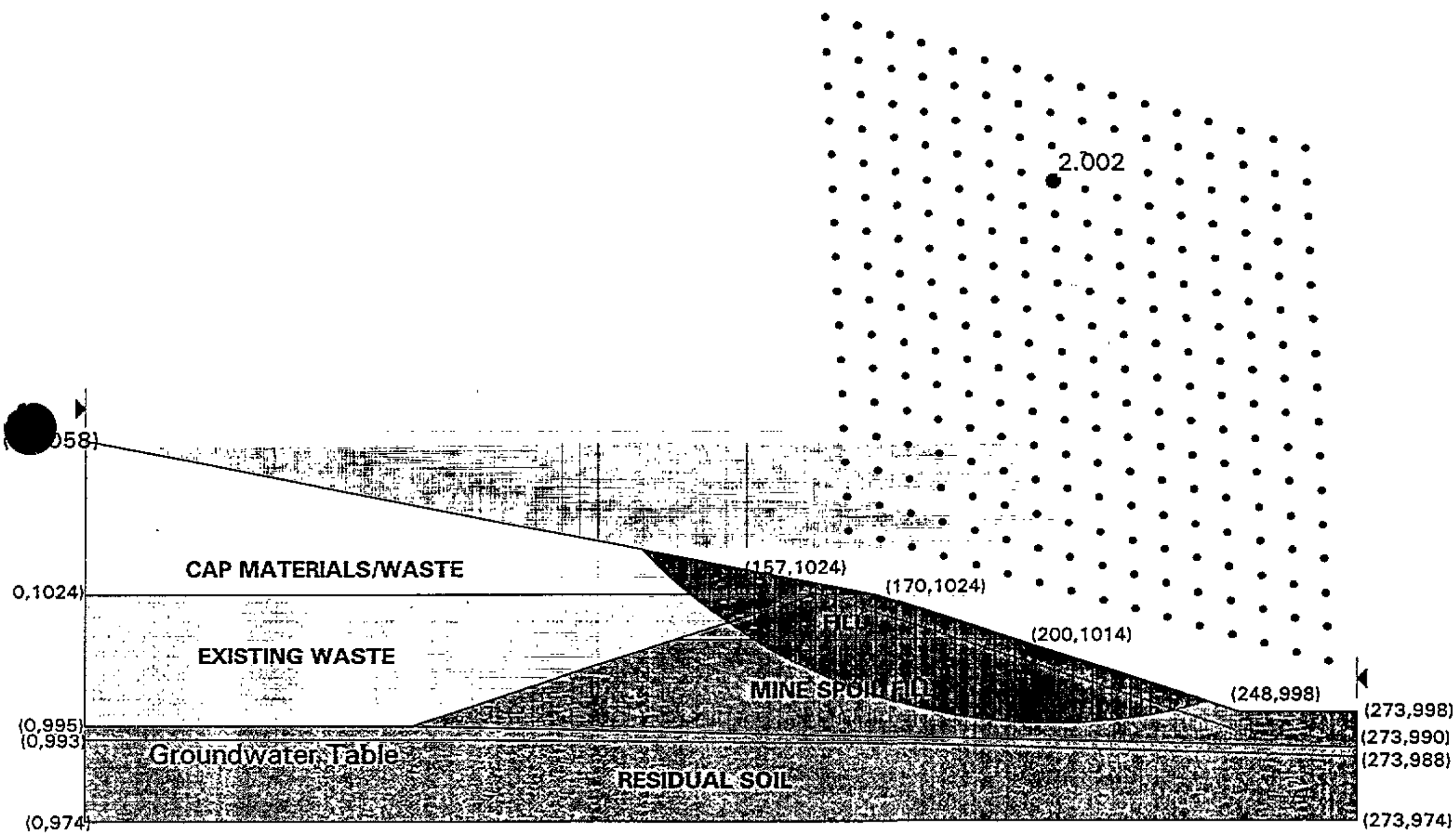
LEGEND

(0,1041) = (Horizontal Station, Elevation)

AR305380

Figure 3

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)
DYNAMIC CONDITION



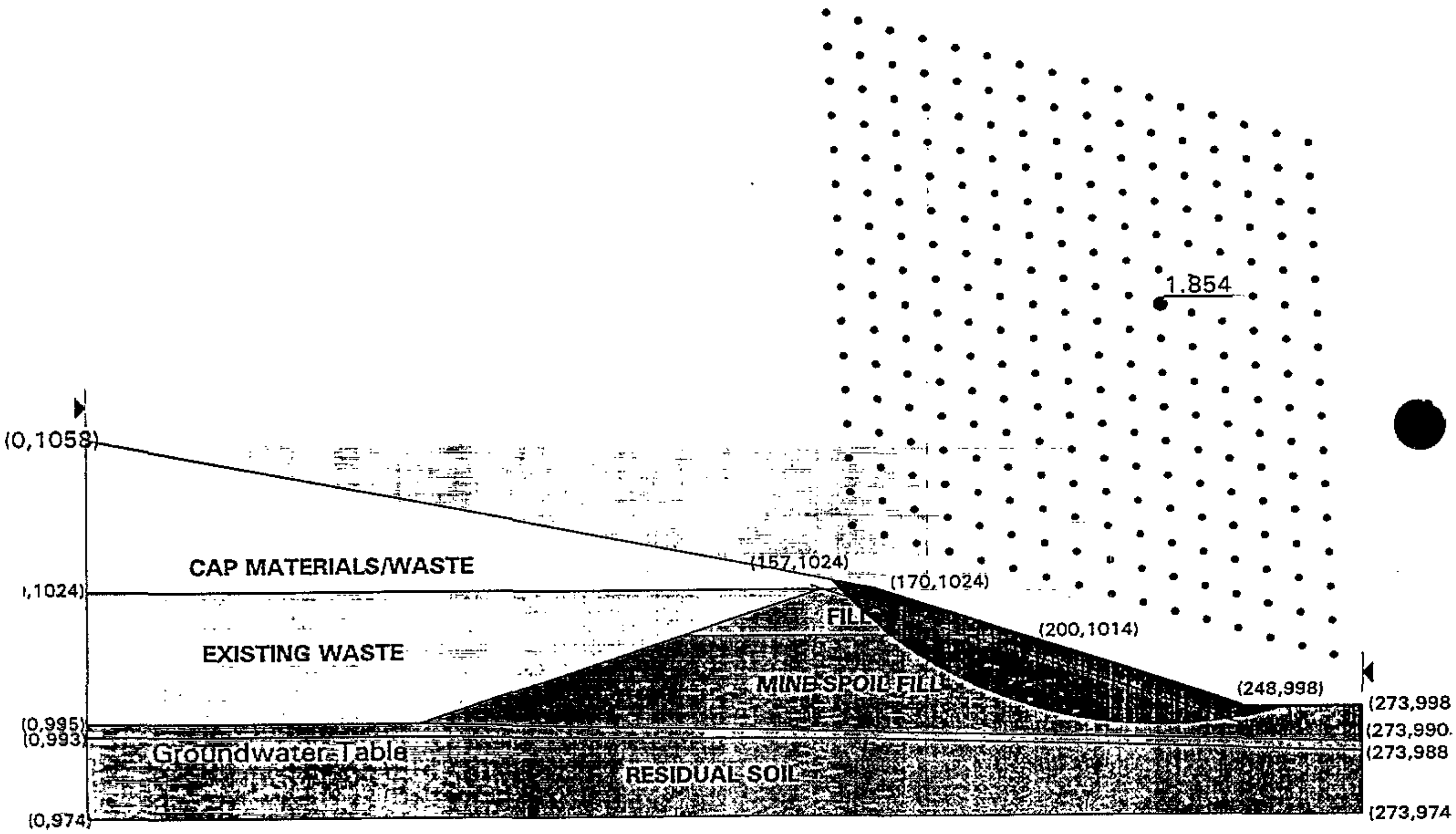
LEGEND

(0,1041) = (Horizontal Station, Elevation)

AR305381

Figure 4

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES) DYNAMIC CONDITION



LEGEND

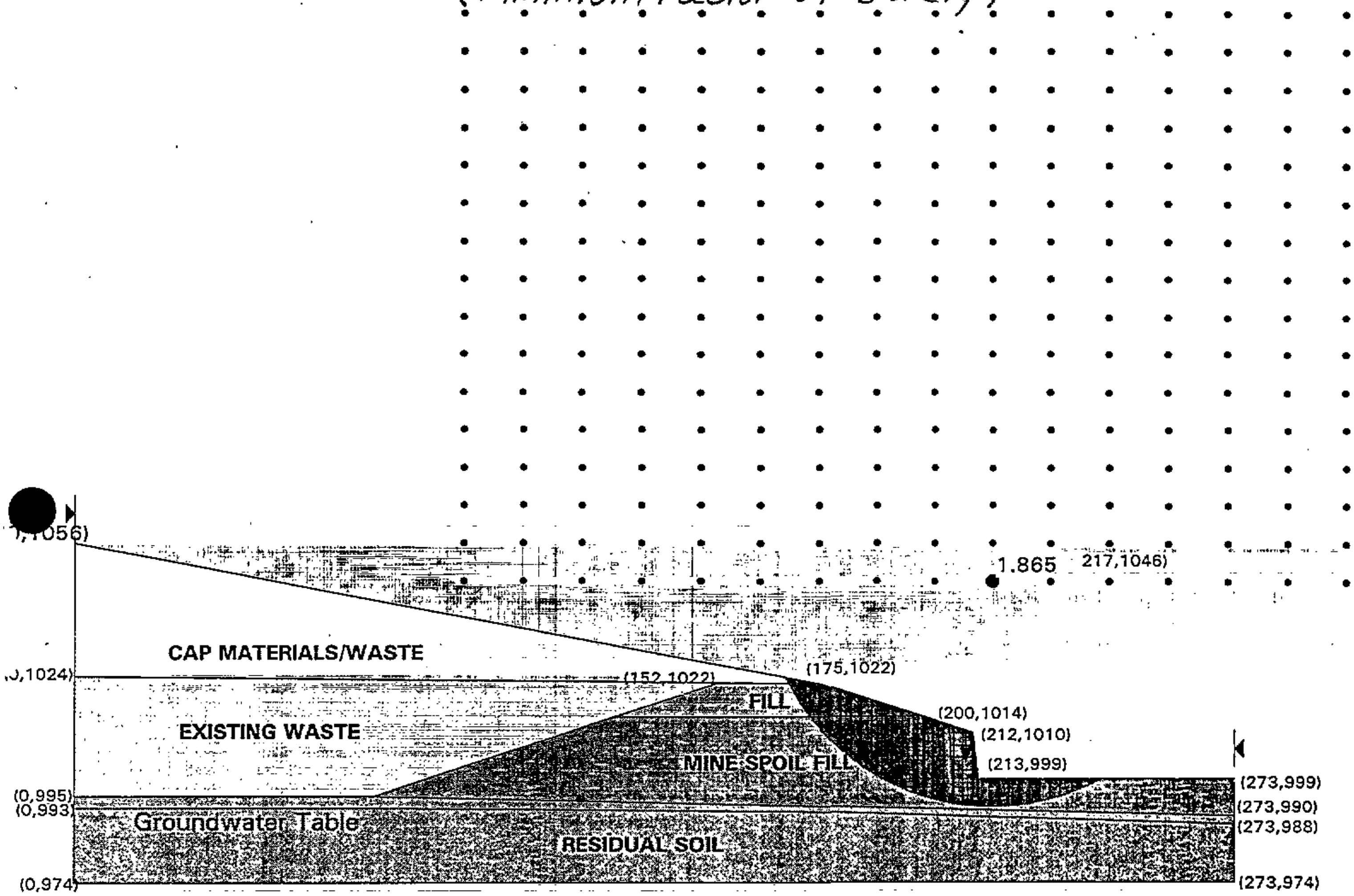
(0,1041) = (Horizontal Station, Elevation)

AR305382

Figure 5

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)

GABION WALL STATIC CONDITION
(Minimum Factor of Safety)



LEGEND

(0,1041) = (Horizontal Station, Elevation)

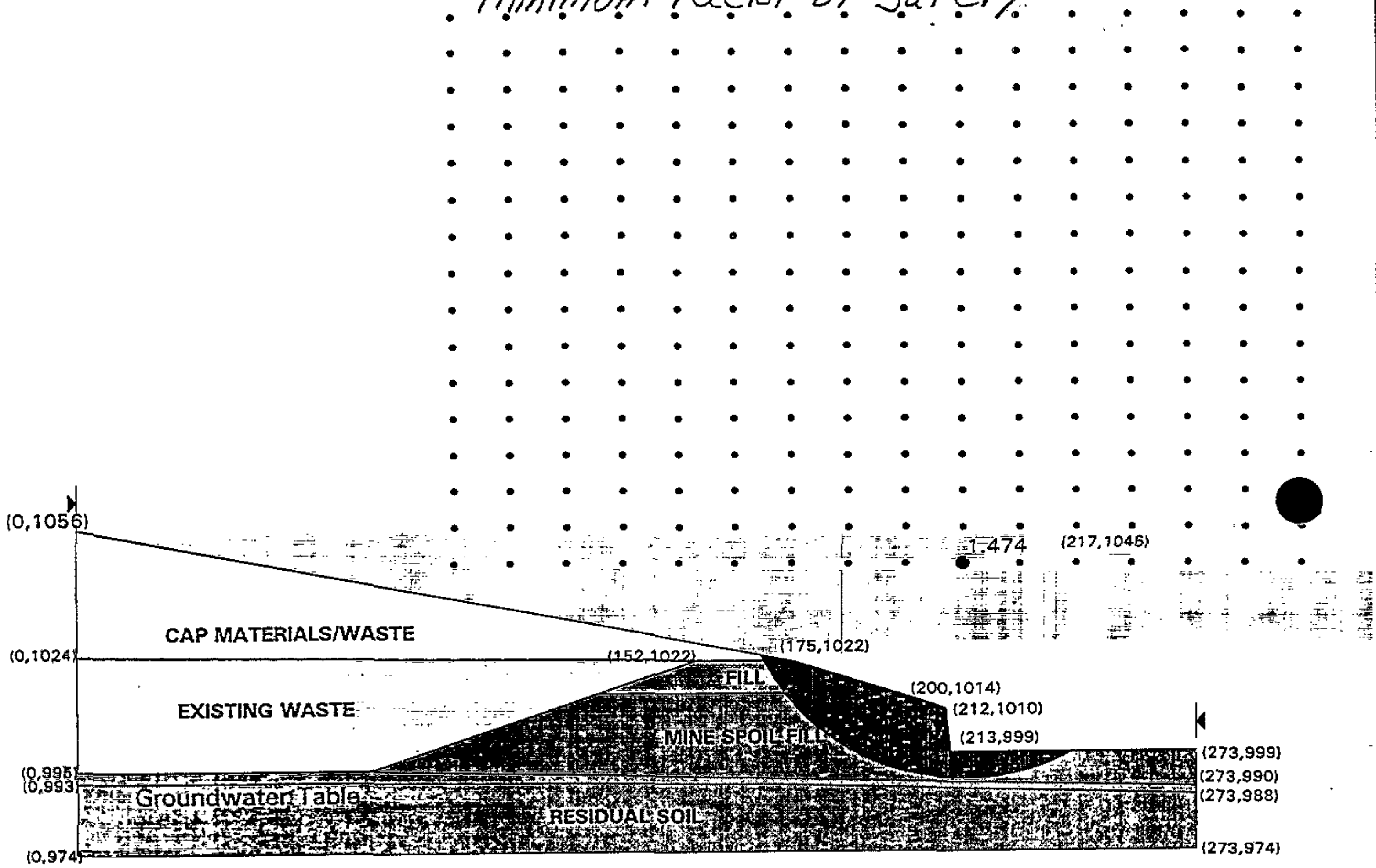
AR305383

Figure 6

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)

GABION WALL DYNAMIC CONDITION

Minimum Factor of Safety



LEGEND

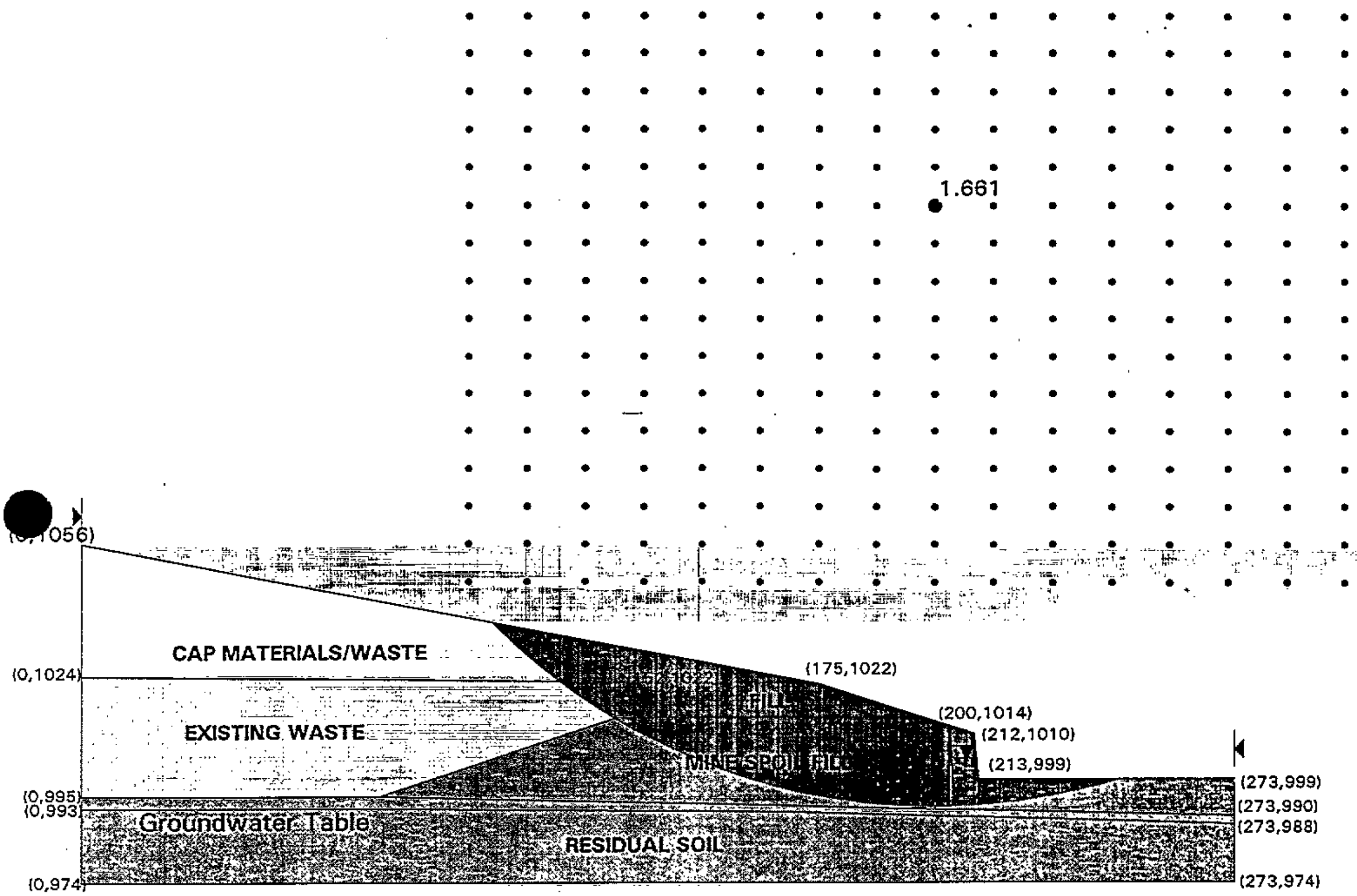
(0,1041) = (Horizontal Station, Elevation)

AR305384

Figure 7

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)

GABION WALL DYNAMIC CONDITION



LEGEND

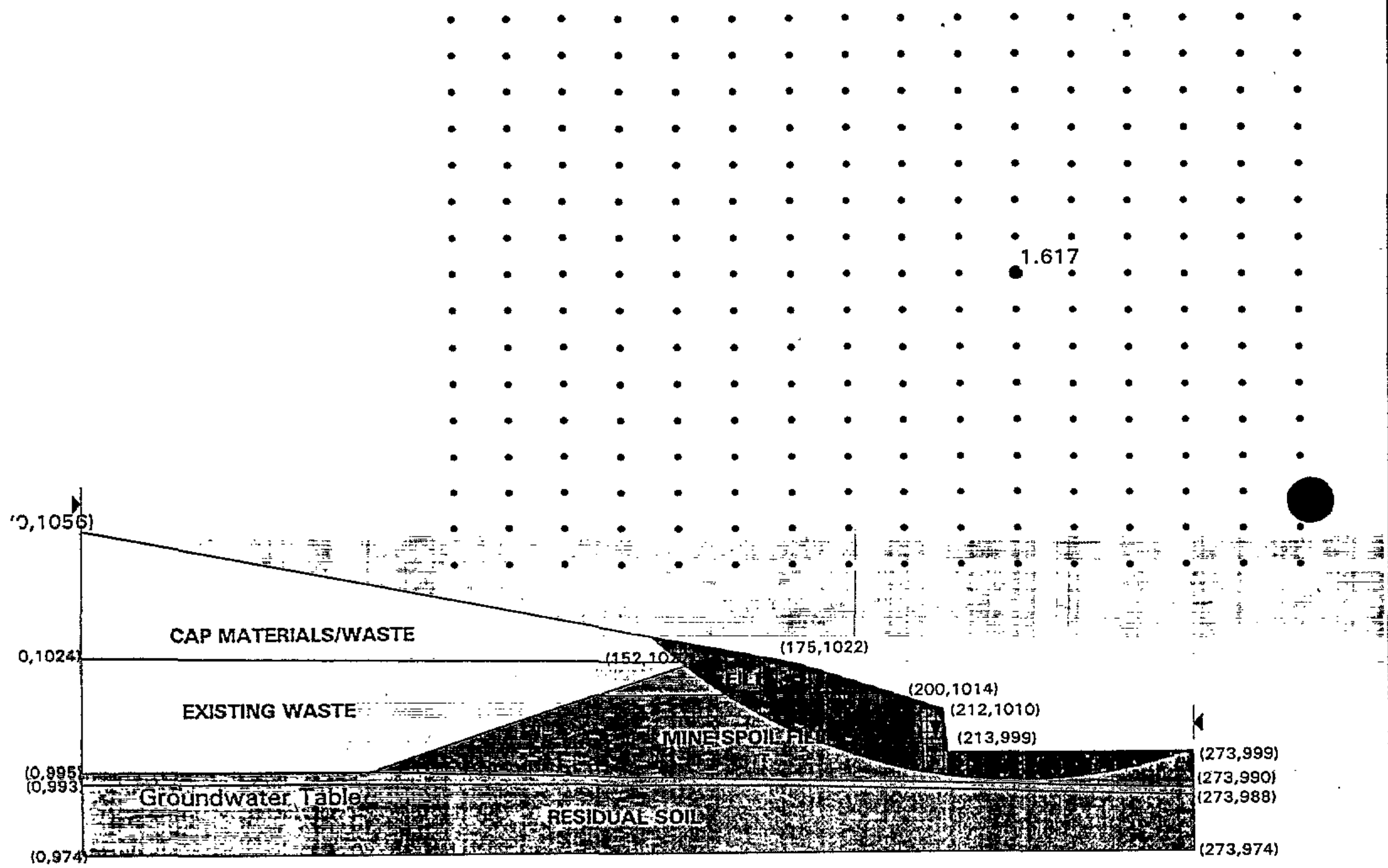
(0,1041) = (Horizontal Station, Elevation)

AR305385

Figure 8

GENERAL CROSS SECTION GEOMETRY (3:1 FINAL SLOPES)

GABION WALL DYNAMIC CONDITION



LEGEND

(0,1041) = (Horizontal Station, Elevation)

AR305386

Figure 9

ATTACHMENT 2

TO

SLOPE STABILITY CALCULATIONS

LOG OF TEST BORING
TEST BORING MW-20

DATE: 5/20/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: 15.0 ft.

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1007.6 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description		SPT (N)	Moisture (%)	Other Tests
0	2/6 2/6 2/6 3/6	Medium compact black silty coarse to fine SAND, some fine gravel. (MINE SPOIL)	1007.6	4		
1005	4/6 4/6 4/6 5/6			8		
5	5/6 4/6 3/6 3/6			7		
	3/6 4/6 4/6 5/6			8		
1000	4/6 5/6 6/6 9/6			11		
10	6/6 5/6 5/6 6/6			10		
995	7/6 5/6 5/6 7/6			10		
15	4/6 3/6 2/6 3/6			5		
990	6/6 5/6 3/6 4/6			8		
	5/6 7/6 6/6 5/6			13		
20	9/6 8/6 7/6 9/6	Compact orange to gray clayey SILT, some coarse to fine sand and gravel. (RESIDUAL SOIL)	19.2 / 988.4	15		
985	9/6 17/6 32/6 40/6			49		
25	17/6 40/6 100/0.3	Weathered sandstone.	24.3 / 983.3	140		
980 / 30						
975		END OF BORING, WATER AT 15.0 FT., NO PID HITS, WELL SET AT 32 FT. Completion Depth = 32 feet	32.0 / 975.6			
35 / 970						

LOG OF TEST BORING

TEST BORING TB-1

DATE: 5/21/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: 10.0 ft.

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 997.6 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

ELEVATION / DEPTH	Soil Description	SPT (N)	Moisture (%)	Other Tests
0 / 997.6	*Medium compact to compact black silty coarse to fine SAND, some fine gravel. (MINE SPOIL)*	26, 28, 17, 15, 13, 10		
13.0 / 984.6	*Brown organic clayey SILT. (FORMER TOPSOIL)*	11		
13.5 / 984.1	*Very compact SAND and GRAVEL. (RESIDUAL SOIL)*	51, 150		
16.5 / 981.1	*END OF BORING, WATER AT 10.0 FT., NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS* Completion Depth = 16.5 feet			

SOIL SYMBOLS / SAMPLER SYMBOLS / BLOWS PER 6 INCHES:
16/6, 12/6, 14/6, 19/6, 20/6, 15/6, 13/6, 10/6, 12/6, 9/6, 8/6, 6/6, 7/6, 8/6, 7/6, 9/6, 9/6, 8/6, 5/6, 5/6, 7/6, 5/6, 5/6, 6/6, 7/6, 6/6, 5/6, 4/6, 17/6, 22/6, 29/6, 40/6, 150/5

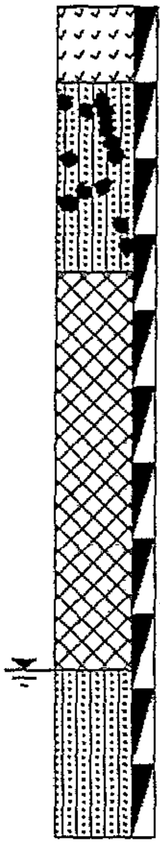
LOG OF TEST BORING

TEST BORING TB-2

DATE: 5/21/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See : Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: 17.5 ft.

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1010.8 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description	SPT (N)	Moisture (%)	Other Tests
1010 / 0	2/6 2/6 5/6 10/6	*Light brown SILT. (TOPSOIL)* 1010.8	7		
	15/6 17/6 17/6 30/6	2.0 1008.8 *Compact to very compact black silty coarse to fine SAND with sandstone fragments. (FILL)*	34		
1005 / 5	15/6 11/6 15/6 17/6		26		
	29/6 15/6 11/6 10/6		26		
	15/6 9/6 9/6 7/6	7.0 1003.8 *Medium compact black silty coarse to fine SAND, some fine gravel. (MINE SPOIL)*	18	9.5	
1000 / 10	17/6 7/6 6/6 5/6		13	16.4	*Grain Size*
	7/6 6/6 5/6 4/6		11	17.3	
995 / 15	2/6 2/6 1/6 2/6		3	18.0	
	2/6 5/6 7/6 8/6		12		
	5/6 7/6 7/6 10/6	17.5 993.3 *Medium compact to compact light brown silty coarse to fine SAND, little fine gravel and clay. (RESIDUAL SOIL)*	14		
990 / 20	26/6 38/6 38/6 40/6		76		
985 / 25		22.0 988.8 *END OF BORING, WATER AT 17.5 FT., NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS* Completion Depth = 22 feet			
980 / 30					
975 / 35					



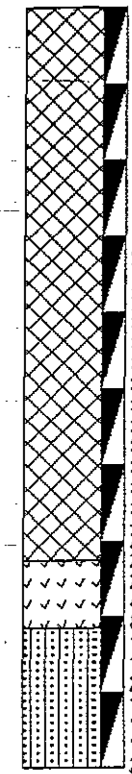
ADVANCED GEOSERVICES CORP.

AR305390

LOG OF TEST BORING
TEST BORING TB-3

DATE: 5/22/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: None Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1020.3 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci



ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description		SPT (N)	Moisture (%)	Other Tests
1020 / 0	12/6 15/6 20/6 29/6 18/6 15/6 15/6 15/6 7/6 8/6 7/6 8/6 7/6 5/6 5/6 3/6 3/6 3/6 3/6 3/6 2/6 3/6 3/6 3/6 3/6 2/6	*Medium compact black silty coarse to fine SAND, some coarse to fine gravel. (MINE SPOIL)*	1020.3	35, 30, 15, 10, 6, 6, 4, 20		
1015 / 5; 1010 / 10			14.5			
1005 / 15	2/6 2/6 4/6 8/6	*Orange silty CLAY. (FORMER TOPSOIL)*	1005.8 / 16.3	31		
	12/6 15/6 20/6 14/6 17/6 19/6 24/6	*Compact to very compact light brown silty coarse to fine SAND, some coarse fine gravel. (RESIDUAL SOIL)*	1004 / 20.0	138		
1000 / 20	60/6 78/6 110/6	*END OF TEST BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS* Completion Depth = 20 feet	1000.3			
995 / 25; 990 / 30; 985 / 35						

LOG OF TEST BORING

TEST BORING TB-6

DATE: 5/22/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: None Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1024.9 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

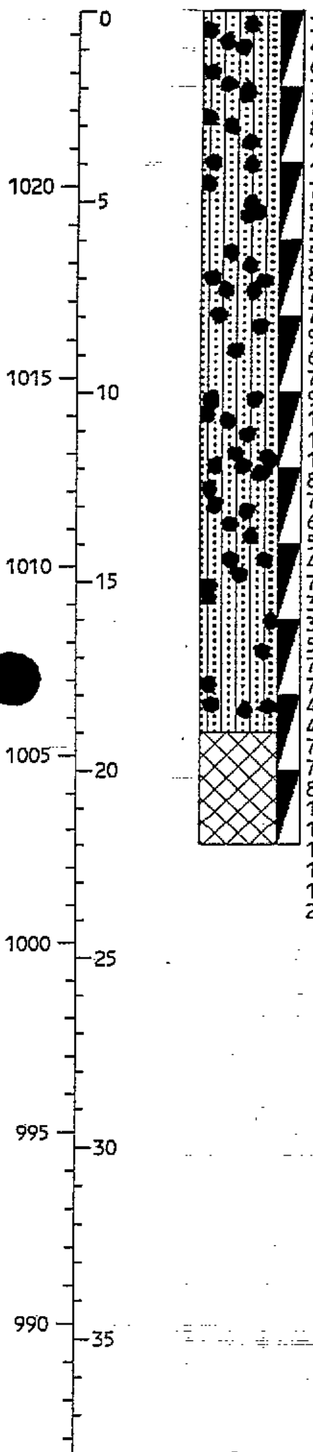
ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description		SPT (N)	Moisture (%)	Other Tests
0	10/6, 10/6	*Medium compact reddish brown silty medium to fine SAND, some coarse to fine gravel, trace to little clay with boulders. (FILL)*	1024.9	17		
	7/6, 6/6, 17/6			16		
	8/6, 8/6, 9/6			15		
1020 — 5	6/6, 4/6, 11/6			65		
	13/6, 65/.5	*Boulder encountered at 6.5', boring offset 5' to the east.*				
	19/6, 81/6			88		
1015 — 10	7/6, 6/6, 7/6		10.5	35		
	21/6	*Battery casing fragments, plastic, ash. (FILL)*	1014.4			
	14/6, 14/6, 10/6			11		
	6/6		13.5			
	5/6, 4/6	*Loose to medium compact black silty coarse to fine SAND, little fine gravel. (MINE SPOIL)*	1011.4	5		
1010 — 15	2/6, 2/6, 3/6					
	5/6, 9/6, 10/6			19		
	9/6, 9/6		18.0			
		END OF BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS	1006.9			
1005 — 20		Completion Depth = 18 feet				
1000 — 25						
995 — 30						
990 — 35						

LOG OF TEST BORING

TEST BORING TB-7

DATE: 5/22/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED/AT: Not Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1024.6 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci



ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description	SPT (N)	Moisture (%)	Other Tests
1024.6 / 0	3/6 4/6 6/6 7/6	*Compact reddish brown silty coarse to fine SAND, some coarse to find gravel, little clay. (FILL)*	10		
	7/6 8/6 12/6 14/6		20		
	7/6 5/6 5/6 5/6		10		
	8/6 7/6 7/6 9/6		14		
	6/6 7/6 9/6 11/6		16		
	10/6 11/6 8/6 7/6		19		
	6/6 5/6 4/6 7/6		9		
	3/6 3/6 5/6 7/6		8		
	7/6 4/6 4/6 7/6		8		
	7/6 8/6 12/6 14/6		20		
1005.6 / 19.0	12/6 17/6 17/6 21/6	*Compact black silty coarse to fine SAND, some fine gravel. (MINE SPOIL)*	34		
1002.6 / 22.0		*END OF BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS* Completion Depth = 22 feet			

LOG OF TEST BORING

TEST BORING TB-8

DATE: 5/23/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: Not Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1024.4 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description	SPT (N)	Moisture (%)	Other Tests
0	10/6 12/6 10/6 12/6 11/6 16/6 22/6 7/6 8/6 6/6 7/6 8/6 23/6 7/6 9/6 9/6 16/6 17/6	*Compact to very compact reddish brown clayey SILT, some coarse to fine sand and gravel, trace cobbles and boulders. (FILL)* 1024.4	22 38 13 16 34		
10.0 / 1014.4	17/6 23/6 4/6 7/6 7/6 5/6 9/6 5/6 5/6 5/6 16/6 4/6 3/6 4/6 5/6 3/6 7/6 8/6 5/6 3/6 2/6 3/6 27/6 18/6 10/6 9/6 5/6 3/6 3/6 3/6 7/6 5/6 4/6 3/6 17/6 9/6	*Medium compact black silty coarse to fine SAND, little fine gravel. (MINE SPOIL)*	14 10 7 10 5 28 6 9 17 170		
29.8 / 994.6	8/6 20/6 27/6 70/6 100/.2	*No sample recovery. (RESIDUAL SOIL)*	57		
32.0 / 992.4	30/6 40/6 17/6 17/6	END OF BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS Completion Depth = 32 feet			

LOG OF TEST BORING

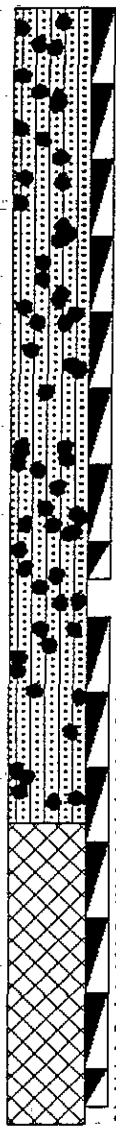
TEST BORING TB-9

DATE: 5/23/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: Not Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1023.3 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

ELEVATION / DEPTH: 0, 1020, 5, 1015, 10, 1010, 15, 1005, 20, 1000, 25, 995, 30, 990, 35, 985

SOIL SYMBOLS / SAMPLER SYMBOLS / BLOWS PER 6 INCHES



4/6, 6/6, 4/6, 4/6, 4/6, 4/6, 4/6, 4/6, 2/6, 3/6, 2/6, 4/6, 3/6, 4/6, 3/6, 5/6, 3/6, 6/6, 8/6, 12/6, 8/6, 7/6, 3/6, 3/6, 8/6, 7/6, 4/6, 4/6, 15/6, 65/6, 30/.1, 13/6, 13/6, 11/6, 9/6, 11/6, 15/6, 7/6, 7/6, 3/6, 4/6, 4/6, 5/6, 7/6, 9/6, 23/6, 18/6, 17/6, 7/6, 9/6, 15/6, 33/6, 17/6, 28/6, 40/6, 32/6, 60/6, 150/.3

Soil Description	SPT (N)	Moisture (%)	Other Tests
Medium compact dark reddish brown clayey SILT, some coarse to fine sand and gravel. (Intermittent Seams of Coarse Gravel) (FILL) 1023.3	10		
	8	9.9	
	5	10.2	
	7	10.5	*Grain Size*
	14		
	10		
	11	7.8	
	95		
(Concrete rubble encountered at 15.5'. Boring offset 10' to the west). (FILL)	24		
	22		
21.5	8		
Compact to very compact black silty coarse to fine SAND, some fine gravel. (MINE SPOIL) 1001.8	32		
	16		
	45		
29.5	210		
END OF BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS 993.8 Completion Depth = 29.5 feet			

LOG OF TEST BORING

TEST BORING TB-10

DATE: 5/23/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: Not Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1025.1 ft.
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci

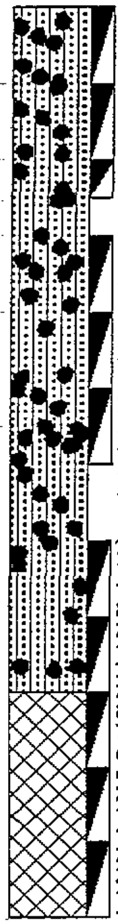
ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS BLOWS PER 6 INCHES	Soil Description		SPT (N)	Moisture (%)	Other Tests
1025 — 0	10/6 17/6 23/6 36/6	*Medium compact to very compact light brown to light reddish brown silty coarse to fine SAND, some coarse to fine gravel, trace clay. (Intermittent Very Compact Gravel Seams) (FILL)*	1025.1	40		
	24/6 32/6 38/6 50/6			70		
1020 — 5	17/6 24/6 32/6 30/6			56		
	19/6 26/6 20/6 38/6			46		
	27/6 23/6 22/6 17/6			45		
1015 — 10	15/6 15/6 8/6 8/6			23		
	14/6 14/6 8/6 8/6			22		
	5/6 8/6 9/6 8/6		15.0	17		
1010 — 15	12/6 14/6 19/6 22/6	*Medium compact to compact black silty coarse to fine SAND, some fine gravel. (MINE SPOIL)*	1010.1	33		
	11/6 10/6 9/6 8/6			19		
1005 — 20	11/6 12/6 15/6 12/6			27		
	24/6 20/6 11/6 8/6			31		
1000 — 25	3/6 4/6 4/6 5/6			8		
	8/6 11/6 17/6 20/6			28		
	19/6 50/6 60/6 79/6		30.0	110		
995 — 30		*END OF BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS* Completion Depth = 30 feet	995.1			
990 — 35						

LOG OF TEST BORING

TEST BORING TB-11

DATE: 5/24/96
PROJECT: Tonolli Superfund Site
BORING LOCATION: See Sheet 3
DRILLING METHOD: 6" OD Hollow Stem Auger
DRILLING COMPANY: Advanced Drilling, Inc.
WATER ENCOUNTERED AT: Not Encountered

PROJECT NO.: 96-248-25
SURFACE ELEVATION: 1025.04 ft
CHECKED BY: TDT
DRILLER: Jerry Malack
INSPECTOR: C. Voci



ELEVATION / DEPTH	Soil Description	SPT (N)	Moisture (%)	Other Tests
1025.04 / 0 – 1007.04 / 18.0	*Compact reddish brown clayey SILT, some coarse to fine sand and gravel. (Intermittent Seams of Gravel-Size Sandstone Fragments) (FILL)*	19 10 81 17 34 18 29 53		
1007.04 / 18.0 – 1001.04 / 24.0	*Loose to medium compact black silty medium to fine SAND. (MINE SPOIL)*	13 2 2		
1001.04 / 24.0	*END OF TEST BORING, DRY AT COMPLETION, NO PID HITS, BORING BACKFILLED WITH DRILLING SPOILS* Completion Depth = 24 feet			

Elevation scale: 1025, 1020, 1015, 1010, 1005, 1000, 995, 990; Depth scale: 0, 5, 10, 15, 20, 25, 30, 35

VALLEY FORGE LABORATORIES, INC.

Engineering Consultants Si[illegible] 19

SOIL LABORATORY TEST REPORT 6-2

Project No. 94104
June 18, 1996

Geotechnical Engineering

Attention: Mr. Todd D. Trotman, P.E.
Advanced GeoServices Corp.
Chadds Ford Business Campus
Rts. 202 & 1, Brandywine One - Suite 202
Chadds Ford, PA 19317

Construction Quality Control

Re: Tonolli Superfund Site, AG #96-248-25
Soil Samples for Laboratory Analysis

Samples Received: 10 Jars delivered on 6/11/96.

Laboratory Testing

Testing Completed:

Test	Standard
Natural Water Content	D2216
Particle-Size Analysis (Sieve Only)	D422

NDT and Related Services

Results:

The results of the moisture contents are shown in Table 1. The results of the particle-size analysis are graphically depicted on the attached Grain Size Distribution Curves. If you have any questions about this test report, please call.

Research and Special Studies

Sincerely,

Jeffrey W. Rosengarten
Geotechnical Engineer

Environmental Engineering

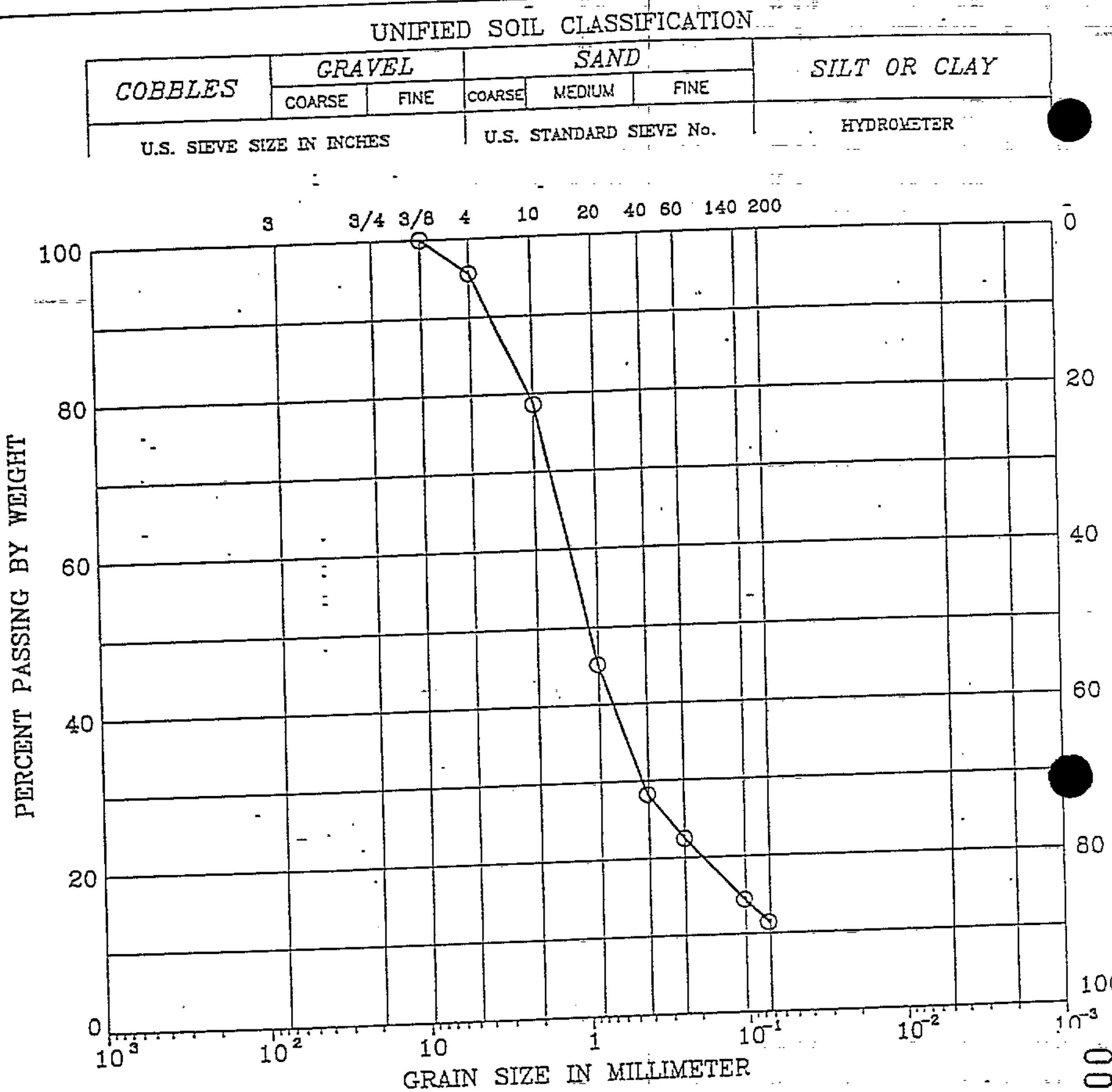
JWR:lcw

Transportation and Traffic Engineering

AR305398

TABLE 1

Sample	Depth (ft)	Moisture Content (%)
TB-2/S-5	8.0	9.49
TB-2/S-6	10.0	16.4
TB-2/S-7	12.0	17.3
TB-2/S-8	14.0	18.0
TB-9/S-2	2.0	9.92
TB-9/S-3	4.0	10.2
TB-9/S-4	6.0	10.5
TB-9/S-6	10.0	7.76



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	TB-2/S-6	10.0			BLACK POORLY-GRADED SAND WITH SILT (SP-SM)

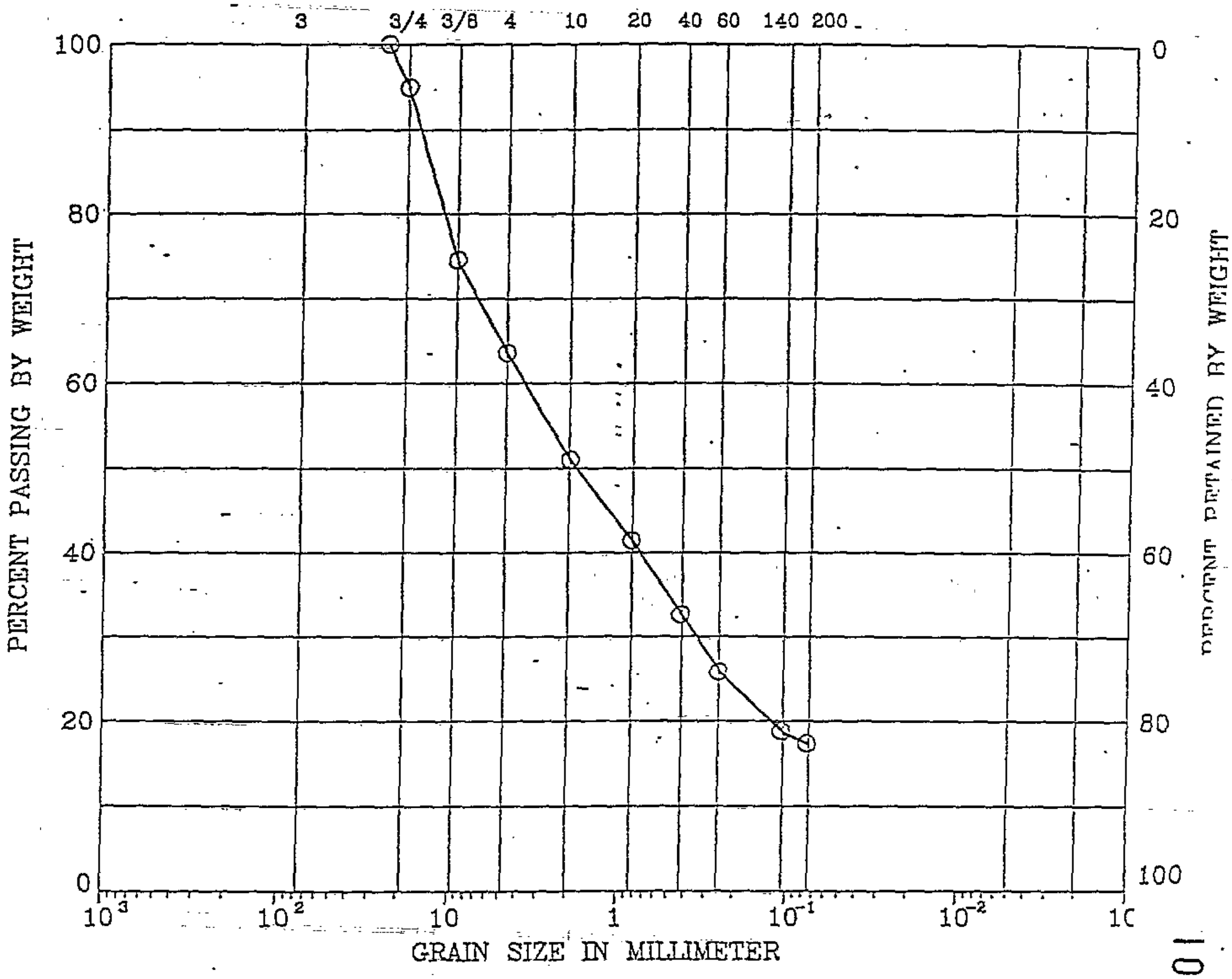
AR305400

Remark : NAT. WATER CONTENT 16.4

Project No. 94104	TONOLLI SUPERFUND SITE
Valley Forge Laboratories, Inc.	GRAIN SIZE DISTRIBUTION 6/19/96

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
U.S. SIEVE SIZE IN INCHES			U.S. STANDARD SIEVE No.			HYDROMETER



AR305401

SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	TB-9/S-4	6.0			TAN SILTY, CLAYEY SAND WITH GRAVEL (SM-SC)

Remark : NAT. WATER CONTENT 10.5

Project No. 94104	TONOLLI SUPERFUND SITE
Valley Forge Laboratories, Inc.	GRAIN SIZE DISTRIBUTION 6/19/96

ATTACHMENT 3

TO

SLOPE STABILITY CALCULATIONS

SIMPLIFIED DESIGN OF BUILDING FOUNDATIONS

JAMES AMBROSE
Professor of Architecture
University of Southern California
Los Angeles, California

SECOND EDITION

WILEY

A Wiley-Interscience Publication
JOHN WILEY & SONS
New York Chichester Brisbane Toronto Singapore

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TABLE 2.5 Summary of Properties and Recommended Design Values for Soils Classified by the Unified System (ASTM Designation D-2487)

Description	Gravel, well-graded; little or no fines			Gravel, poorly graded, little or no fines			*Silty gravel and gravel-sand-silt mixes*		
ASTM classification (See Figure 2.7)	GW			GP			GM		
Significant properties	≧95% *retained* on No. 200 sieve (0.003 in.) ≧50% of coarse fraction retained on No. 4 sieve ($\frac{3}{16}$ in.) $C_u > 4$ $1 < C_z < 3$			≧95% retained on No. 200 sieve (0.003 in.) ≧50% of coarse fraction retained on No. 4 sieve ($\frac{3}{16}$ in.) Does not meet C_u and/or C_z requirements for well-graded (GW)			50–88% retained on No. 200 sieve (0.003 in.) ≧50% of coarse fraction retained on No. 4 sieve ($\frac{3}{16}$ in.) Atterberg plot below A line or $I_p < 4$		
Field identification	Significant amounts of coarse rock fragments; easily pulverized; fast draining; wide range of grain sizes			Significant amounts of coarse rock fragments; easily pulverized; fast draining; has narrow range of sizes or is *gap-graded*			Gravely but forms clumps that pulverize with moderate effort; wet sample takes little or no remolding before disintegrating; slow draining		
Average properties	Loose ($N < 10$)	Medium ($10 < N < 30$)	Dense ($N > 30$)	Loose ($N < 10$)	Medium ($10 < N < 30$)	Dense ($N > 30$)	Loose ($N < 10$)	Medium ($10 < N < 30$)	Dense ($N > 30$)
Allowable bearing (lb/ft²) with minimum of one ft surcharge	1300	1500	2000	1300	1500	2000	1000	1500	2000
increase for surcharge (%/ft)	20	20	20	20	20	20	20	20	20
maximum total	8000	8000	8000	8000	8000	8000	8000	8000	8000
Lateral pressure									
active *coefficient*	0.25	0.25	0.25	0.25	0.25	0.25	0.30	0.30	0.30
passive (lb/ft² per ft depth)	200	300	400	200	300	400	167	250	333
Friction (coefficient or lb/ft²)	0.50	0.60	0.60	0.50	0.60	0.60	0.40	0.50	0.50
Weight (lb/ft³)									
dry	100	110	115	90	100	110	100	115	130
saturated	125	130	135	120	125	130	125	135	145
Compressibility	Medium	Low	Very low	Medium	Low	Very low	Medium	Low	Very low

(Continued)

Weight (lb/ft^3)									
dry	110	120	125	100	110	115	90	100	110
saturated	125	130	135	125	130	135	120	125	130
Compressibility	Medium	Low	Very low	Medium high	Medium	Low	Medium high	Medium	Low

Description	*Silty sand and sand-silt mixes*			*Clayey sand and sand-clay mixes*			*Inorganic silt, very find sand, rock flour, silty or clayey fine sand*		
ASTM classification (See Figure 2.7)	SM			SC			ML		
Significant properties	50–80% retained on No. 200 sieve (0.003 in.) >50% of coarse fraction passes No. 4 sieve ($\frac{3}{16}$ in.) Atterberg plot below A line, or $I_p < 4$			50–80% retained on No. 200 sieve (0.003 in.) >50% passes No. 4 sieve ($\frac{3}{16}$ in.) Atterberg plot above A line or $I_p > 7$			≧50% passes No. 200 sieve (0.003 in.) $w_L \leqq 50\%$ Atterberg plot below A line $I_p < 20$		
Field identification	Sandy soil; forms clumps that can be pulverized with moderate effort; wet sample takes little remolding before disintegrating; slow draining			Sandy soil; forms clumps that offer some resistance to being pulverized; wet sample takes some remolding before disintegrating; very slow draining			Fine-grained soils of low plasticity; slow draining; dry clumps easily pulverized; won't form thin thread when molded		
Average properties	Loose ($N < 10$)	Medium ($10 < N < 30$)	Dense ($N > 30$)	Loose or soft ($N < 10$)	Medium ($10 < N < 30$)	Dense or stiff ($N > 30$)	Loose or soft ($N < 10$)	Medium ($10 < N < 30$)	Dense or stiff ($N < 30$)
Allowable bearing (lb/ft^2) with minimum of one ft surcharge	500	1000	1500	1000	1500	2000	500	750	1000
increase for surcharge (%/ft)	20	20	20	20	20	20	20	20	20
maximum total	4000	4000	4000	4000	4000	4000	3000	3000	3000
Lateral pressure									
active coefficient	0.30	0.30	0.30	0.30	0.30	0.30	0.35	0.35	0.35
passive (lb/ft^2 per ft depth)	100	167	233	133	217	300	67	100	133
Friction (coefficient or lb/ft^2)	0.35	0.40	0.40	0.35	0.40	0.40	0.35 or 250	0.40 or 375	0.40 or 500
Weight (lb/ft^3)									
dry	105	115	120	105	115	120	105	115	120
saturated	125	130	135	125	130	135	125	130	135
Compressibility	Medium	Low	Low	Medium	Low	Low	Medium high	Medium	Low

(Continued)

NAVFAC DM-7.2
MAY 1982

APPROVED FOR PUBLIC RELEASE

PAUL MARANO
2610 ACADEMY AVE.
HOLMES, PA. 19043

FOUNDATIONS AND EARTH STRUCTURES

DESIGN MANUAL 7.2

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
200 STOVALL STREET
ALEXANDRIA, VA. 22332

AR305406

TABLE 1

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi)	Typical Value of Compression: At 3.6 tsf (50 psi)	Typical Strength Characteristics: Cohesion (as compacted) psf	Typical Strength Characteristics: Cohesion (saturated) psf	Typical Strength Characteristics: Ø (Effective Stress Envelope Degrees)	Typical Strength Characteristics: Tan Ø	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				Percent of Original Height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) Indicates that typical property is greater than the value shown.
   (..) Indicates insufficient data available for an estimate.

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Telephone 215/638-0554

Towne Centre Complex
10830 E. 45th Place, Ste. 200
Tulsa, OK 74146
Telephone 918/664-4434

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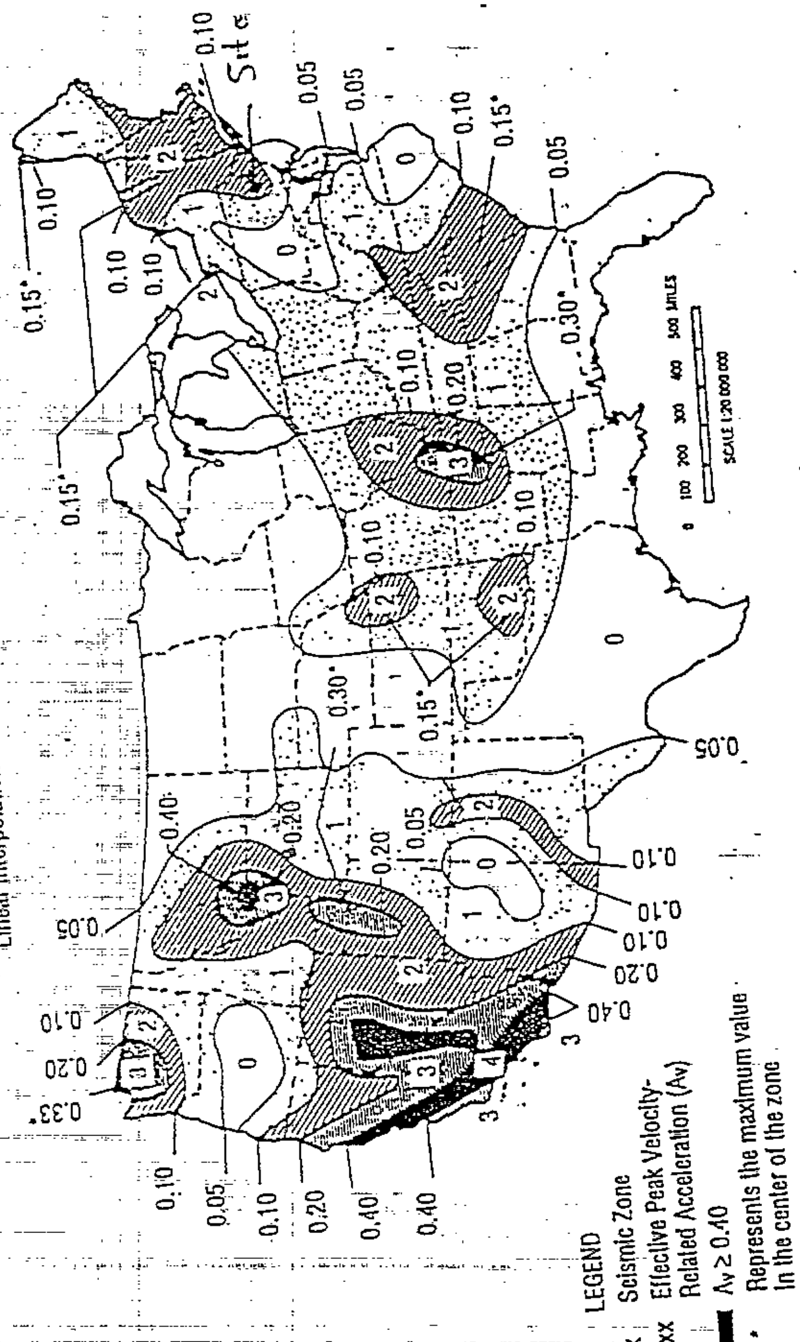
Figure 1113.1
MAP OF SEISMIC ZONES AND EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION (Av) CONTIGUOUS 48 STATES

Linear interpolation between contours is acceptable



LANDFILL SETTLEMENT CALCULATIONS

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

SETTLEMENT CALCULATIONS FOR ADDITIONAL FILL MATERIAL

1. PURPOSE- THE PURPOSE OF THIS CALCULATION IS TO ESTIMATE THE TOTAL SETTLEMENT WHICH WILL OCCUR AT THE TONOLLI SITE FOLLOWING PLACEMENT OF 37 FEET OF FILL (THIS INCLUDES THE CAP) ON THE LANDFILL. THIS CALCULATION IS DONE USING THE SCHMERTMANNS METHOD AS DESCRIBED IN DESIGN MANUAL 7.01. THIS METHOD IS APPROPRIATE FOR DETERMINING SETTLEMENT OF GRANNUAL MATERIAL

2. METHOD- TOTAL SETTLEMENT IS ESTIMATED USING THE FOLLOWING FORMULA.

$$\Delta H = C_1 C_2 \Delta p \sum_0^{2B} \frac{I_z}{E_s} \Delta z$$

WHERE:

$C_1 = 1 - 0.5(p_o/\Delta p)$; $C_1 \geq 0.5$

$C_2 = 1 + 0.2 \log(10t)$

p_o = OVERBURDEN PRESSURE @ FOUNDATION LEVEL

Δp = NET FOUNDATION PRESSURE INCREASE

t = ELAPSED TIME IN YEARS.

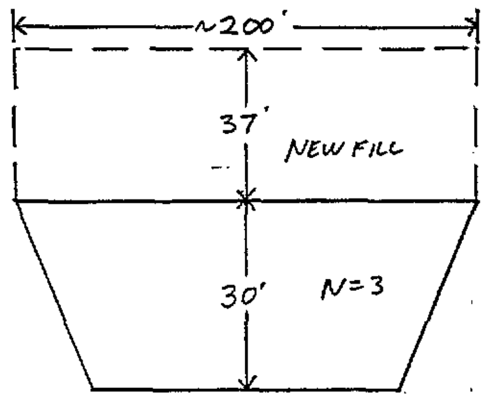
SHEET 1 OF 3 PROJECT NO. 96-248-01 PROJECT NAME TONOLLI
BY CTR DATE 10-2-96 DESCRIPTION SETTLEMENT CALC
CHK. BY TML DATE 10-4-96

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

3. SITE CONDITIONS -

LANDFILL MATERIALS ARE KNOWN TO BE BATTERY CASINGS, SLUDGE, AND SAND. INFORMATION REGARDING THE INSITU DENSITY OR BLOW COUNTS OF THESE MATERIALS WAS NOT AVAILABLE FROM SITE INVESTIGATIONS. HOWEVER, IT IS KNOWN THESE MATERIALS WERE PLACED AND "TRACKED-IN" WITH A BULLDOZER, FOR PURPOSES OF THIS CALCULATION A BLOW COUNT PER FOOT (N VALUE) OF "3" HAS BEEN ASSUMED. MATERIALS BENEATH THE LANDFILL ARE BELIEVED TO BE RESIDUAL SOIL AND ROCK.



B = 200'
2B = 400'
B/2 = 100'

N.T.S.

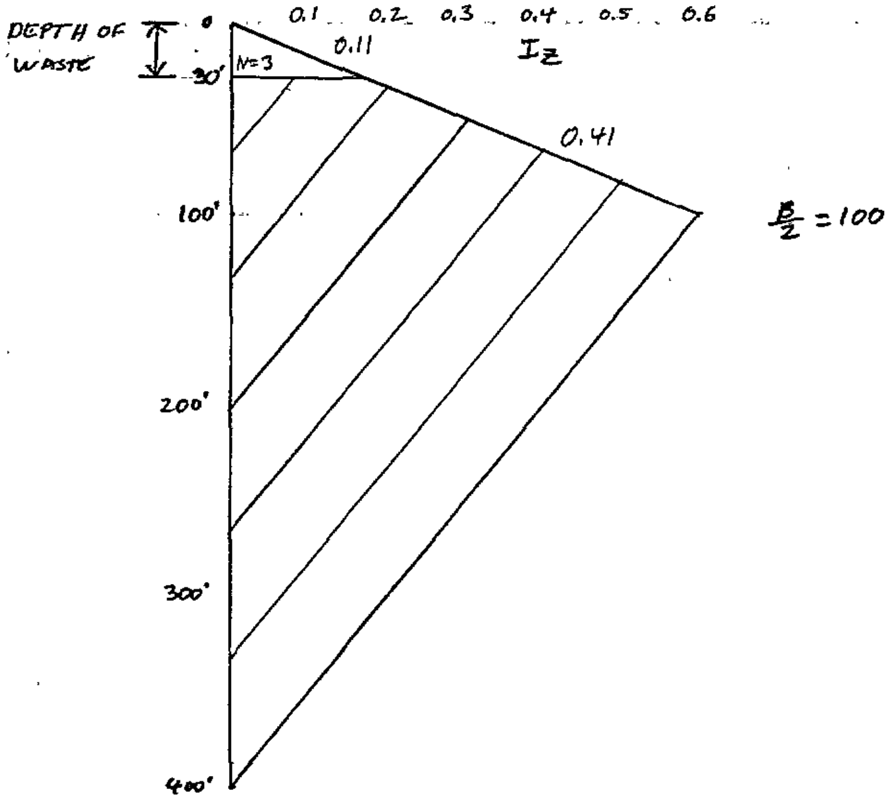
$P_0 = 0$ (NEW FILL)

$\Delta P = 37'(120\,lb/ft^3) = 4{,}440\,psf$ or $2.2\,TSF$ (ASSUME SOIL $\rho = 120\,lb/ft^3$)

$c_1 = 1 - 0.5\left(\frac{P_0}{\Delta P}\right)^{0} = 1$

$c_2 = 1 + 0.2\log(10t) = 1.6$ (ASSUME $t = 100$ yrs)

SHEET 2 OF 3 PROJECT NO. 96-248-01 PROJECT NAME TONOLLI
BY CTR DATE 10-2-96 DESCRIPTION SETTLEMENT CALC.
CHK. BY TML DATE 10-4-96



(1) LAYER	(2) Δz (IN)	(3) N	(4) ES/N	(5) Es (TSF)	(6) Zc	(7) I_z	(8) $\frac{I_z}{E_s}\Delta z$
1	30'=360"	3	4	12	180"	~0.11	3.3"

$$\Delta H = C_1 C_2 \Delta p \sum_{0}^{2B} \frac{I_z}{E_s} \Delta z$$

$$1(1.6)\, 2.2\ \text{TSF}\, (3.3) = 11.6'' \approx 1.0'$$

SETTLEMENT DUE TO THE PLACEMENT OF ADDITIONAL WASTE MAY VARY FROM 0" TO 12".

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

SETTLEMENT CALCULATIONS — REMOVAL OF LANDFILL LIQUID

1. PURPOSE – THE PURPOSE OF THIS CALCULATION IS TO ESTIMATE THE SETTLEMENT WHICH WILL OCCUR IN THE EXISTING LANDFILL WASTE DURING THE REMOVAL OF THE IMPOUNDED LIQUID. THIS CALCULATION WAS PERFORMED USING THE SCHMERTMANN'S METHOD AS DESCRIBED IN DESIGN MANUAL 7.01. THIS METHOD IS APPROPRIATE FOR DETERMINING SETTLEMENT OF GRANULAR MATERIAL.

2. FROM THE FORMULA ESTABLISHED FROM THE DETERMINATION OF SETTLEMENT DURING THE PLACEMENT OF ADDITIONAL MATERIAL:

$$\Delta H = C_1 C_2 \Delta p \sum_{0}^{2B} \frac{I_z}{E_s} \Delta z$$

WHERE:

$C_1 = 1$

$C_2 = 1.6$

$\Delta p = (30' \text{ OF WATER})(62.4\,pcf) = 1{,}872\,psf = 0.94\,TSF$

$\frac{I_z}{E_s}\Delta z = 3.3''$

$$\Delta H = (1)(1.6)(0.94\,TSF)(3.3'') = 5.0'' \text{ or } 0.4'$$

SETTLEMENT DUE TO WATER REMOVAL MAY VARY FROM 0" TO 5".

TOTAL SETTLEMENT OF EXISTING WASTE MAY RANGE FROM 0" TO 17" DUE TO BOTH THE INCREASE IN LOAD AND REMOVAL OF WATER.

DATA REQUIRED:

1. A profile of standard penetration resistance N (blows/ft) versus depth, from the proposed foundation level to a depth of 2B, or to boundary of an incompressible layer, whichever occurs first. Value of soil modulus E_s is established using the following relationships.

Soil Type	E_s/N
Silts, sands silts, slightly cohesive silt-sand mixtures	4
Clean, fine to med, sands & slightly silty sands	7
Coarse sands & sands with little gravel	10
Sandy gravels and gravel	12

2. Least width of foundation = B, depth of embedment = D, and proposedaverage contact pressure = P.

3. Approximate unit weights of surcharge soils, and position of water table if within D.

4. If the static cone bearing value q_c is measured compute E_s based on $E_s = 2\ q_c$.

ANALYSIS PROCEDURE:

Refer to table in example problem for column numbers referred to by parenthesis:

1. Divide the subsurface soil profile into a convenient number of layers of any thickness, each with constant N over the depth interval 0 to 2B below the foundation.

2. Prepare a table as illustrated in the example problem, using the indicated column headings. Fill in columns 1, 2, 3 and 4 with the layering assigned in Step 1.

3. Multiply N values in column 3 by the appropriate factor E_s/N (col. 4) to obtain values of E_s; place values in column 5.

4. Draw an assumed 2B-0.6 triangular distribution for the strain influence factor I_z, along a scaled depth of 0 to 2B below the foundation. Locate the depth of the mid-height of each of the layers assumed in Step 2, and place in column 6. From this construction, determine the I_z value at the mid-height of each layer, and place in column 7.

FIGURE 7

Settlement of Footings Over Granular Soils: Example Computation Using Schmertmann's Method

5. Calculate $(I_z/E_s)\ \Delta Z$, and place in column 8. Determine the sum of all values in column 8.

6. Total settlement = $\Delta H = C_1 C_2 \Delta p \sum_{0}^{2B} \left(\frac{I_z}{E_s}\right) \Delta Z$,

where $C_1 = 1 - 0.5\ (p_o/\Delta p)$; $C_1 \geq 0.5$ embedment correction factor

$C_2 = 1 + 0.2 \log (10t)$ creep correction factor

p_o = overburden pressure at foundation level

Δp = net foundation pressure increase

t = elapsed time in years.

EXAMPLE PROBLEM:

GIVEN THE FOLLOWING SOIL SYSTEM AND CORRESPONDING STANDARD PENETRATION TEST (SPT) DATA, DETERMINE THE AMOUNT OF ULTIMATE SETTLEMENT UNDER A GIVEN FOOTING AND FOOTING LOAD:

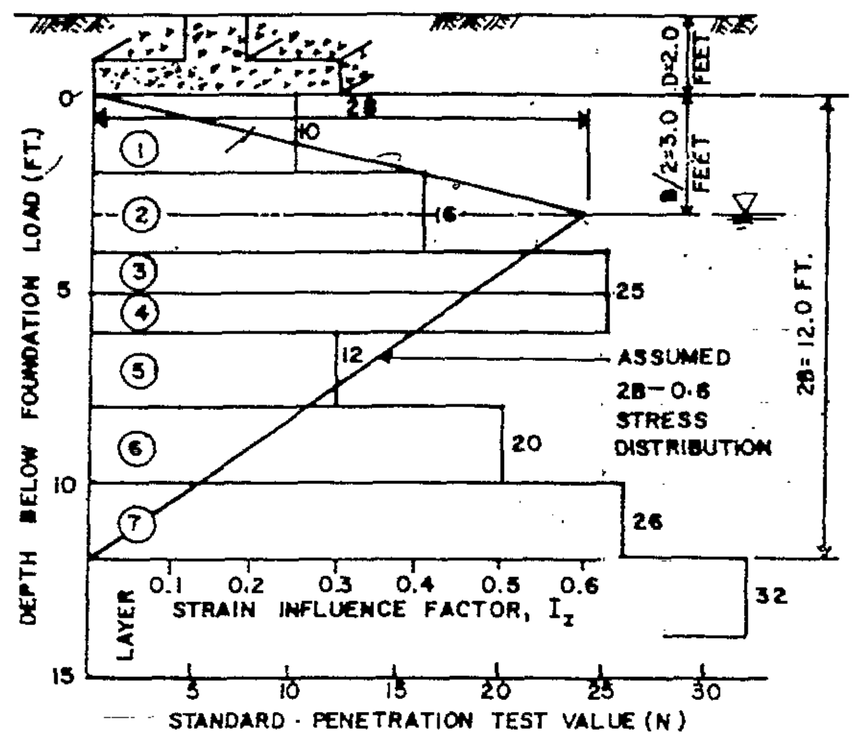


FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Footing Details:

Footing width: 6.0 ft. (min.) by 8.0 ft. (max.)

Depth of Embedment: 2.0 ft. Load (Dead + Live): 120 tons

Soil Properties:

Depth Below Surface (ft.)	Depth Below Base of Footing (ft.)	Unit Wt. (pcf) Moist	Sat.	Soil Description
0 - 5	<5	95	105	Fine sandy silt
5 - 10	3 - 8	105	120	Fine to medium sand
10 - 17	8 - 15	120	130	Coarse sand

Solution:

Layer (1)	ΔZ (in.) (2)	N (3)	E_s/N (4)	E_s (tsf) (5)	Z_c (in.) (6)	I_z (7)	$\frac{I_z}{E_s}\Delta Z$ (in./tsf) (8)
1	24 2	10	4	40	12	.20	0.120
2	24 2	16	4	64	36	.60	0.225
3	12 1	25	4	100	54	.50	0.060
4	12 1	25	7	175	66	.43	0.029
5	24 2	12	7	84	84	.33	0.094
6	24 2	20	7	140	108	.20	0.034
7	24 2	26	10	260	132	.07	0.006

$\Sigma = 0.568$

$p_o = (2.0 \text{ ft})(95 \text{ pcf}) = 190 \text{ psf} = 0.095 \text{ tsf}$
$\Delta p = 120 \text{ tons}/(6 \text{ ft.})(8 \text{ ft.}) = 2.50 \text{ tsf}$

At t = 1 yr,

$$C_1 = 1 - 0.5(.095/2.50) = 0.981$$

$$C_2 = 1 + 0.2 \log (10)(1) = 1.20$$

$$\Delta H = (0.981)(1.20)(2.50)(0.568) = \underline{1.67 \text{ in.}}$$

FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

STORMWATER CALCULATIONS

DRAINAGE AREA #3 (DA#3)

EXISTING CONDITIONS

- TOTAL AREA = 11.0 acres.

  IMPERVIOUS AREA = 7.65 acres.
  BARE SOIL AREA = 3.35 acres.

- RUNOFF COEFFICIENTS (c)

  IMPERVIOUS AREA: c = 0.90
  BARE SOIL AREA: c = 0.40

- COMPOSITE RUNOFF COEFFICIENT (C_{comp})

$$C_{comp} = \frac{(0.90)(7.65) + (0.40)(3.35)}{11.0}$$

$$C_{comp} = 0.75$$

- TIME OF CONCENTRATION (tc)

THE ASPHALT PORTION OF THIS DRAINAGE AREA MAKES UP THE LARGEST PORTION OF THIS DRAINAGE AREA. THEREFORE, THE TRAVEL TIME FROM THE FARTHEST POINT OF THE ASPHALT AREA TO THE NEAREST INLET (i.e. THE LONGEST PIPE TRAVEL TIME AND OVERLAND FLOW) WILL BE DETERMINED BY BOTH THE KIRPICH AND FAA METHODS.

$L = 460'$ at 1.0%

A TRAVEL TIME OF 2 MINUTES WILL BE ADDED FOR PIPE FLOW.

· KIRPICH :

$t_c = 4.1$ min

FAA :
USING $C = 0.90$

$t_c = 9.7$ min

TAKING THE AVERAGE OF THE TWO.

$t_c = 7$ MINUTES

· RATIONAL METHOD

$Q = C_{comp} I A = 0.75 \times 11.0\ Ac \times I$

$Q = 8.25\ I$ ($t_c = 7$ MINUTES)

POST REMEDIATION CONDITIONS

· TOTAL AREA = 13.68 acres

IMPERVIOUS AREA : 3.62 acres
BARE SOIL AREA : 0.98 acres
GRASS AREA : 4.31 acres
CRUSHED STONE/BACKFILL : 4.77 acres

- RUNOFF COEFFICIENTS.

IMPERVIOUS AREA : 0.90
BARE SOIL AREA : 0.40
GRASS AREA : 0.30 (ASCE)
CRUSHED STONE/BACKFILL : 0.50 (ASCE) ASSUMES CRUSHED STONE IS PACKED DURING CONSTRUCTION: HIGH VALUE USED DUE TO STEEP SLOPES.

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• COMPOSITE RUNOFF COEFFICIENT (C_{comp})

$$C_{comp} = \frac{(0.90)(3.62)+(0.40)(0.98)+(0.30)(4.31)+(0.50)(4.77)}{13.68}$$

$$C_{comp} = 0.54$$

• TIME OF CONCENTRATION (t_c)

THE TIME OF CONCENTRATION FOR POST-REMEDIATION CONDITIONS WAS CACULATED ASSUMING THAT THE LONGEST FLOW TIME WILL BE FROM THE CAP. FAA AND KIRPATCH WILL BE COMBINED.

$L = 150'$ @ $S = 33\%$ (FAA)

$L = 360'$ @ $S = 2\%$ (KIRPATCH)

USING $C = 0.30$

FAA:

$t_c = 5.5$ MIN.

KIRPATCH:

$t_c = 3.3$ MIN.

$t_c = 8.8$ MINUTES

• RATIONAL METHOD.

$Q = C_{comp} I A = 0.54 \times 13.68 \times I$

$Q = 7.39 I$ ($t_c = 8.8$ MINUTES)

SHEET 3 OF 22 PROJECT NO. 96-348-62 PROJECT NAME TONOLLI

BY TDT DATE 7/20/98 DESCRIPTION STORMWATER CALC

CHK. BY TML DATE 7/21/98

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

COMPARISON OF Q PEAK (DA #3)

PRE- AND POST REMEDIATION

RETURN PERIOD (YEARS)	PRE		POST		Δ (cfs)
	INTENSITY (I) (IN/hr)	Q PEAK (cfs)	INTENSITY (I) (IN/hr)	Q PEAK (cfs)	
1	3.7	30.5	3.1	22.9	-7.6
2	4.2	34.7	3.6	26.6	-8.1
5	5.0	41.3	4.3	31.8	-9.5
10	5.6	46.2	4.9	36.2	-10.0
25	6.2	51.2	5.4	39.9	-11.3
50	7.1	58.6	6.2	45.8	-12.8
100	8.0	66.0	7.1	52.5	-13.5

PRE: $Q = 8.25\,I$ (tc 7 MINUTES)

POST: $Q = 7.39\,I$ (tc 8.8 MINUTES)

NOTE: NEGATIVE Δ INDICATES DECREASE IN PEAK DISCHARGE

SHEET 4 OF 22 PROJECT NO. 96-248-62 PROJECT NAME TONOLLI
BY TPT DATE 7/20/98 DESCRIPTION STORMWATER C
CHK. BY TML DATE 7/21/98

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

DRAINAGE AREA #4 (DA#4) (DRAINAGE AREA FORMED AFTER CONSTRUCTION)

EXISTING CONDITIONS

- NO PRESENT RUNOFF

POST-REMEDIATIONS CONDITIONS.

- TOTAL AREA = 4.4 acres

  GRASS AREA: 4.4 acres

- RUNOFF COEFFICIENTS

  GRASS AREA: 0.30

- COMPOSITE RUNOFF COEFFICIENT (C_{comp})

  $C_{comp} = 0.30$

- TIME OF CONCENTRATION (t_c)

  USING BOTH THE KIRPATCH AND FAA METHODS

  L = 160' @ S = 20% FAA

  L = 620' @ S = 1% KIRPATCH

  FAA t_c = 6.7 MIN

  KIRPATCH t_c = 6.5 MIN

  13.2 MINUTES

SHEET 5 OF 22 PROJECT NO. 96-248-62 PROJECT NAME TONOLLI
BY TDT DATE 7/20/98 DESCRIPTION STORM WATER CALC.
CHK. BY TML DATE 7/21/98

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

- RATIONAL METHOD

$Q = C_{comp} I A = 0.30 \times 4.4\ Ac \times I$

$Q = 1.32\ I$ (t_c = 13.2 MINUTES).

PEAK DISCHARGE FOR DA#4

RETURN PERIOD (YEARS)	INTENSITY (in/hr)	Q PEAK (cfs)
1	2.6	3.4
2	3.1	4.1
5	3.8	5.0
10	4.3	5.7
25	5.0	6.6
50	5.8	7.7
100	6.3	8.3

BASIN ROUTING

BASIN #1 WILL SERVE AS A SEDIMENT BASIN DURING CONSTRUCTION ACTIVITIES AND WILL BE REMOVED. BASIN #2 AND BASIN #3 WILL REMAIN.

BASIN #2 AND BASIN #3 ARE SIZED TO BE ABLE HANDLE A 100 YEAR STORM WITHOUT OVERTOPPING. HYDRAFLOW (VERSION 5.0) WAS USED FOR THE ROUTING ANALYSIS. THE INPUT AND OUTPUT FOR BASIN #2 AND BASIN #3 ARE ATTACHED.

Reservoir No. 2

Basin#2

Culvert / Orifice Structures

		[A]	[B]	[C]
Rise (in)	=	24	0	0
Span (in)	=	24	0	0
No. Barrels	=	1	0	0
Invert El. (ft)	=	1004.50	0.00	0.00
Length (ft)	=	160.0	0.0	0.0
Slope (%)	=	9.00	0.00	0.00
N-Value	=	.013	.013	.013
Orif. Coeff.	=	0.60	0.60	0.60
Multi-Stage	=	—	No	No

Weir Structures

		[A]	[B]	[C]
Crest Len (ft)	=	25.0	0.0	0.0
Crest El. (ft)	=	1009.00	0.00	0.00
Weir Coeff.	=	3.00	3.00	3.00
Eqn. Exp.	=	1.50	1.50	1.50
Multi-Stage	=	No	No	No

Tailwater Elevation = 0.00 ft

Note: All outflows have been analyzed under inlet and outlet control.

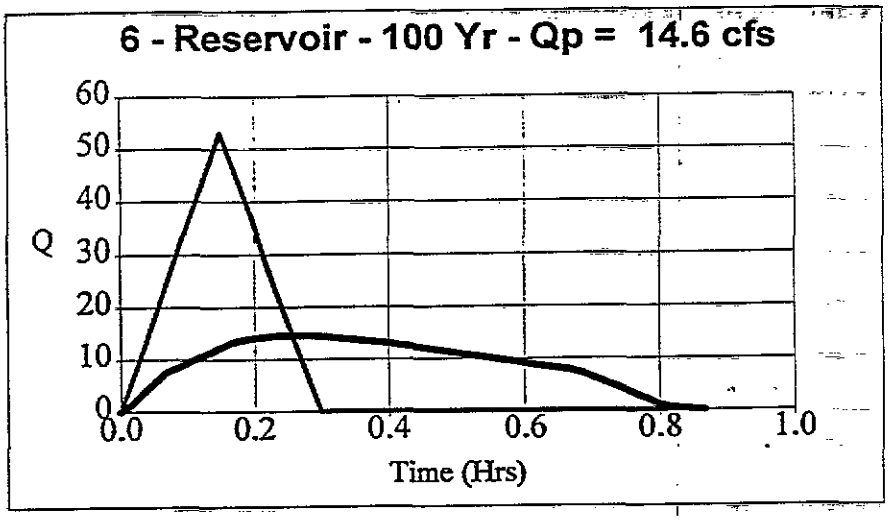
Stage / Storage / Discharge Table

Stage (ft)	Storage (cuft)	Elevation (ft)	Culv. A (cfs)	Culv. B (cfs)	Culv. C (cfs)	Weir A (cfs)	Weir B (cfs)	Weir C (cfs)	Discharge (cfs)
0.0	00	1004.50	0.00	—	—	0.00	—	—	0.00
0.1	207	1004.55	2.39	—	—	0.00	—	—	2.39
0.1	414	1004.60	3.38	—	—	0.00	—	—	3.38
0.2	621	1004.65	4.14	—	—	0.00	—	—	4.14
0.2	828	1004.70	4.78	—	—	0.00	—	—	4.78
0.3	1,035	1004.75	5.35	—	—	0.00	—	—	5.35
0.3	1,242	1004.80	5.86	—	—	0.00	—	—	5.86
0.4	1,449	1004.85	6.33	—	—	0.00	—	—	6.33
0.4	1,656	1004.90	6.76	—	—	0.00	—	—	6.76
0.5	1,863	1004.95	7.17	—	—	0.00	—	—	7.17
0.5	2,070	1005.00	7.56	—	—	0.00	—	—	7.56
0.6	3,089	1005.10	8.29	—	—	0.00	—	—	8.29
0.7	4,108	1005.20	8.95	—	—	0.00	—	—	8.95
0.8	5,126	1005.30	9.57	—	—	0.00	—	—	9.57
0.9	6,145	1005.40	10.15	—	—	0.00	—	—	10.15
1.0	7,164	1005.50	10.70	—	—	0.00	—	—	10.70
1.1	8,183	1005.60	11.22	—	—	0.00	—	—	11.22
1.2	9,202	1005.70	11.72	—	—	0.00	—	—	11.72
1.3	10,220	1005.80	12.19	—	—	0.00	—	—	12.19
1.4	11,239	1005.90	12.65	—	—	0.00	—	—	12.65
1.5	12,258	1006.00	13.10	—	—	0.00	—	—	13.10
1.7	15,520	1006.20	13.95	—	—	0.00	—	—	13.95
1.9	18,781	1006.40	14.74	—	—	0.00	—	—	14.74
2.1	22,043	1006.60	15.86	—	—	0.00	—	—	15.86
2.3	25,304	1006.80	17.25	—	—	0.00	—	—	17.25
2.5	28,566	1007.00	18.52	—	—	0.00	—	—	18.52

Continues on next page...

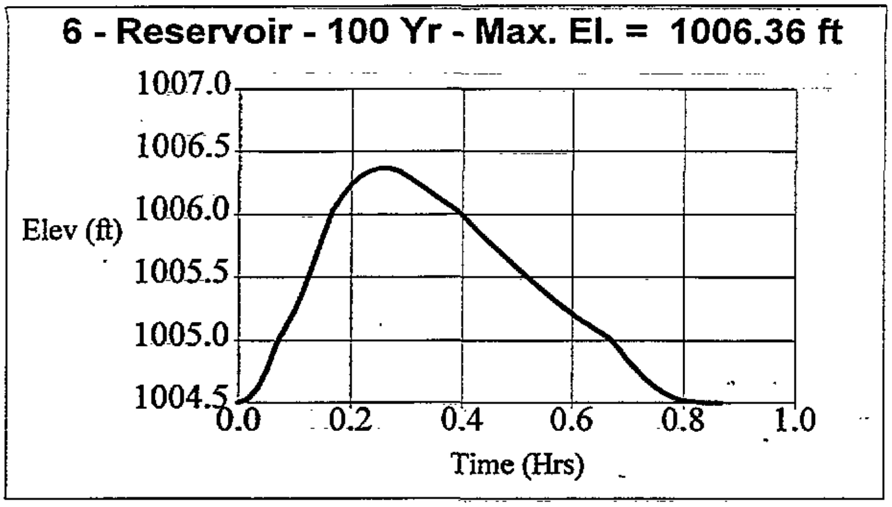
Stage / Storage / Discharge Table

Stage (ft)	Storage (cuft)	Elevation (ft)	Culv. A (cfs)	Culv. B (cfs)	Culv. C (cfs)	Weir A (cfs)	Weir B (cfs)	Weir C (cfs)	Discharge (cfs)
2.7	31,828	1007.20	19.72	—	—	0.00	—	—	19.72
2.9	35,089	1007.40	20.85	—	—	0.00	—	—	20.85
3.1	38,351	1007.60	21.92	—	—	0.00	—	—	21.92
3.3	41,612	1007.80	22.94	—	—	0.00	—	—	22.94
3.5	44,874	1008.00	23.91	—	—	0.00	—	—	23.91
3.7	46,926	1008.20	24.85	—	—	0.00	—	—	24.85
3.9	48,978	1008.40	25.76	—	—	0.00	—	—	25.76
4.1	51,030	1008.60	26.63	—	—	0.00	—	—	26.63
4.3	53,082	1008.80	27.48	—	—	0.00	—	—	27.48
4.5	55,134	1009.00	28.30	—	—	0.00	—	—	28.30
4.7	57,186	1009.20	29.09	—	—	6.71	—	—	35.81
4.9	59,238	1009.40	29.87	—	—	18.98	—	—	48.85
5.1	61,290	1009.60	30.63	—	—	34.87	—	—	65.49
5.3	63,342	1009.80	31.36	—	—	53.68	—	—	85.04
5.5	65,394	1010.00	32.08	—	—	75.00	—	—	107.08



Basin #2

Q vs Time



Basin # 2

Elev. vs Time

AR305429

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Reservoir Report

Reservoir No. 1

Basin#3

Culvert / Orifice Structures

		[A]	[B]	[C]
Rise (in)	=	24	0	0
Span (in)	=	24	0	0
No. Barrels	=	1	0	0
Invert El. (ft)	=	1004.60	0.00	0.00
Length (ft)	=	688.0	0.0	0.0
Slope (%)	=	2.10	0.00	0.00
N-Value	=	.013	.013	.013
Orif. Coeff.	=	0.60	0.60	0.60
Multi-Stage	=	—	No	No

Weir Structures

		[A]	[B]	[C]
Crest Len (ft)	=	0.0	0.0	0.0
Crest El. (ft)	=	0.00	0.00	0.00
Weir Coeff.	=	3.00	3.00	3.00
Eqn. Exp.	=	1.50	1.50	1.50
Multi-Stage	=	No	No	No

Tailwater Elevation = 0.00 ft

Note: All outflows have been analyzed under inlet and outlet control.

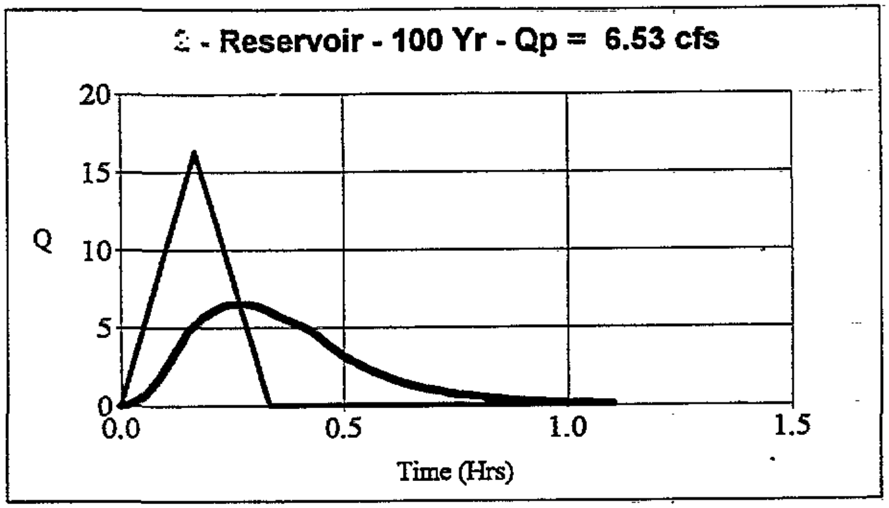
Stage / Storage / Discharge Table

Stage (ft)	Storage (cuft)	Elevation (ft)	Culv. A (cfs)	Culv. B (cfs)	Culv. C (cfs)	Weir A (cfs)	Weir B (cfs)	Weir C (cfs)	Discharge (cfs)
0.0	00	1004.60	0.00	—	—	—	—	—	0.00
0.2	2,951	1004.80	4.78	—	—	—	—	—	4.78
0.4	5,902	1005.00	6.77	—	—	—	—	—	6.77
0.6	8,853	1005.20	8.29	—	—	—	—	—	8.29
0.8	11,804	1005.40	9.57	—	—	—	—	—	9.57
1.0	14,755	1005.60	10.70	—	—	—	—	—	10.70
1.2	17,706	1005.80	11.72	—	—	—	—	—	11.72
1.4	20,657	1006.00	12.66	—	—	—	—	—	12.66
1.6	23,608	1006.20	13.53	—	—	—	—	—	13.53
1.8	26,559	1006.40	14.35	—	—	—	—	—	14.35
2.0	29,510	1006.60	15.12	—	—	—	—	—	15.12
2.2	37,331	1006.80	16.57	—	—	—	—	—	16.57
2.4	45,152	1007.00	17.90	—	—	—	—	—	17.90
2.6	52,973	1007.20	19.13	—	—	—	—	—	19.13
2.8	60,794	1007.40	20.29	—	—	—	—	—	20.29
3.0	68,615	1007.60	21.39	—	—	—	—	—	21.39
3.2	76,436	1007.80	22.43	—	—	—	—	—	22.43
3.4	84,257	1008.00	23.43	—	—	—	—	—	23.43
3.6	92,078	1008.20	24.39	—	—	—	—	—	24.39
3.8	99,899	1008.40	25.31	—	—	—	—	—	25.31
4.0	107,720	1008.60	26.20	—	—	—	—	—	26.20
4.2	119,447	1008.80	27.06	—	—	—	—	—	27.06
4.4	131,174	1009.00	27.89	—	—	—	—	—	27.89
4.6	142,901	1009.20	28.70	—	—	—	—	—	28.70
4.8	154,628	1009.40	29.48	—	—	—	—	—	29.48
5.0	166,355	1009.60	30.25	—	—	—	—	—	30.25

Continues on next page...

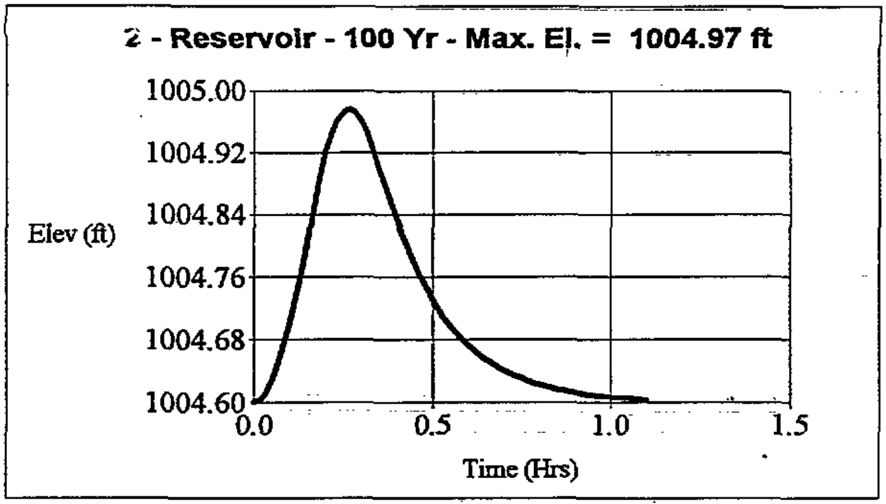
Stage / Storage / Discharge Table

Stage (ft)	Storage (cuft)	Elevation (ft)	Culv. A (cfs)	Culv. B (cfs)	Culv. C (cfs)	Weir A (cfs)	Weir B (cfs)	Weir C (cfs)	Discharge (cfs)
5.2	178,082	1009.80	31.00	—	—	—	—	—	31.00
5.4	189,809	1010.00	31.73	—	—	—	—	—	31.73
5.6	201,536	1010.20	32.44	—	—	—	—	—	32.44
5.8	213,263	1010.40	33.14	—	—	—	—	—	33.14
6.0	224,990	1010.60	33.82	—	—	—	—	—	33.82
7.0	320,030	1011.60	35.20	—	—	—	—	—	35.20
8.0	415,070	1012.60	36.10	—	—	—	—	—	36.10
9.0	510,110	1013.60	36.97	—	—	—	—	—	36.97
10.0	605,150	1014.60	37.82	—	—	—	—	—	37.82
11.0	700,190	1015.60	38.65	—	—	—	—	—	38.65
12.0	795,230	1016.60	39.47	—	—	—	—	—	39.47
13.0	890,270	1017.60	40.27	—	—	—	—	—	40.27
14.0	985,310	1018.60	41.05	—	—	—	—	—	41.05
15.0	1,080,350	1019.60	41.82	—	—	—	—	—	41.82
16.0	1,175,390	1020.60	42.58	—	—	—	—	—	42.58



Basin #3

Q vs Time



Basin #3
Elev. vs Time

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WATER RESOURCES BULLETIN
AMERICAN WATER RESOURCES ASSOCIATION
VOL. 23, NO. 3 JUNE 1987

REGIONAL RAINFALL INTENSITY-DURATION-FREQUENCY CURVES FOR PENNSYLVANIA[1]

Gert Aron, David J. Wall, Elizabeth L. White, and Christopher N. Dunn[2]

ABSTRACT: A statistical analysis of all available continuous hourly and 15-minute duration rainfall records for Pennsylvania was performed to develop an updated procedure to estimate design storms. As a result of this study, Pennsylvania was divided into five homogeneous rainfall regions and a set of rainfall intensity-duration curves developed for each region, for return periods of 1 to 100 years and durations ranging from 5 minutes to 24 hours. The PDT-IDF curves were judged to be a better representation of Pennsylvania rainfall than the nationwide TP-40 maps, particularly for storm events of 10-years and lower return periods.

The average time distribution of 24-hour storms in Pennsylvania was found to be well represented by the SCS Type II distribution. The Corps of Engineers SPS 24-hour distribution was found to differ appreciably from both the SCS Type II and the Pennsylvania 24-hour storm distribution. For storm durations between 15 and 90 minutes the standard Yarnell intensity-duration curves closely resemble Pennsylvania storm distributions.

(KEY TERMS: design rainfall; Pennsylvania; regional rainfall analysis; storm duration; storm frequency; storm intensity; meteorology.)

INTRODUCTION

The duration, quantity, and intensity of rainfall have major effects on highway drainage and inundation problems. For any hydrologic analysis or design, ranging from the simplest rational formula flow rate estimate to the most sophisticated stormwater runoff simulation, reliable rainfall estimates are necessary.

The most widely used source of design rainfall depths for various return periods and durations is the U.S. Weather Bureau Technical Paper No. 40 (Hershfield, 1961) commonly referred to as TP-40. This rainfall atlas contains 49 rainfall contour maps of the United States for durations varying from 30 minutes to 24 hours and return periods from 2 to 100 years. It is simple to use and is, in general, representative of regional rainfall. However, the maps, which comprise the atlas, lack the resolution needed to recognize local areas of characteristically low or high rainfall. For example, Kerr, *et al.* (1970, pg. 20) have shown that for some locations in Pennsylvania along a line stretching from Dauphin County northeastward to Pike County (see Figure 1), the TP 40 contours underestimate some rainfall amounts by more than 20 percent while overestimating others in different parts of the state.

In 1977, the TP-40 manual was supplemented by the NOAA (National Oceanographic and Atmospheric Administration) Technical Memorandum, HYDRO-35 (NOAA, 1977). HYDRO-35 includes maps that cover the eastern United States and indicate rainfall depth contours of 15-, 30-, and 60-minute durations for 2-, 10-, and 100-year return periods; equations for interpolation are also provided. These maps provide a means for estimating design rainfalls having short durations, but they still lack adequate resolution, because they were generated from a small network of rainfall stations widely distributed over a large area.

In 1970, a more detailed set of rainfall maps for Pennsylvania was developed by Kerr, *et al.* (1970), for the Pennsylvania Department of Environmental Resources. These maps contain more detail than does TP-40, but many users found the procedure for determining the magnitude of a design rainfall to be somewhat tedious, resulting in its limited use. In addition, more than 15 years of rainfall data have become available since the Kerr study to further justify another attempt to improve estimates of design rainfall for Pennsylvania.

In January 1985, an agreement was reached between the Pennsylvania Department of Transportation and The Pennsylvania State University to conduct a statistical analysis of all available continuous-record rainfall data in Pennsylvania. The primary objective of the study was to develop a set of regional rainfall intensity-duration-frequency curves, later referred to as the PDT—IDF curves, representative of the rainfall variations in Pennsylvania. A secondary objective was to compare the temporal distribution of Pennsylvania storms with those represented by "standard" distributions such as the Soil Conservation Service (SCS) Type II and the Corps of Engineers Standard Project Storm (SPS).

[1] Paper No. 86101 of the *Water Resources Bulletin*. Discussions are open until February 1, 1988.

[2] Respectively, Professor of Civil Engineering, Assistant Professor of Civil Engineering, Senior Research Associate, and Graduate Assistant, Department of Civil Engineering, and the Environmental Resources Research Institute, The Pennsylvania State University, University Park, Pennsylvania 1680[illegible]

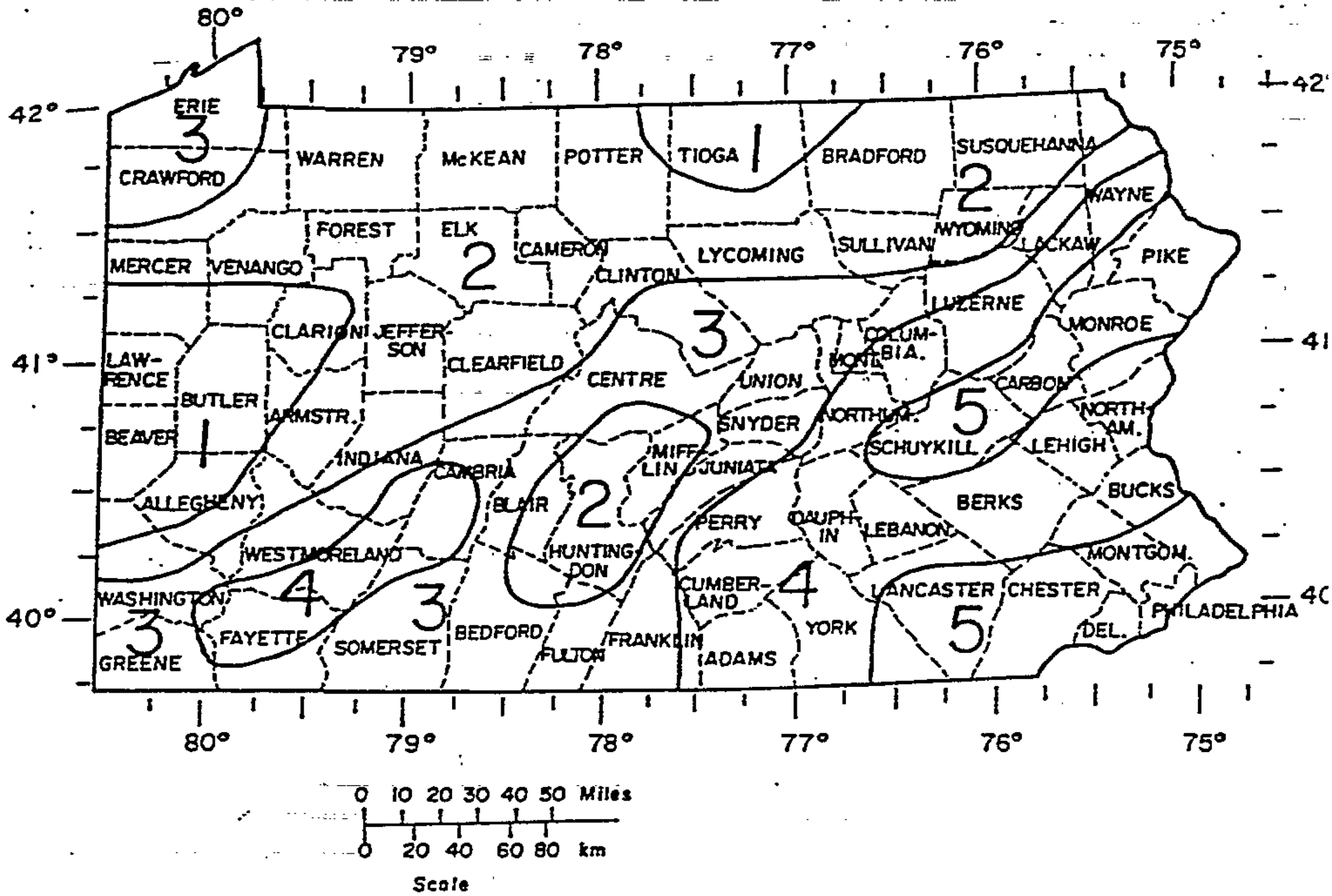

Figure 1. Delineated Regions With Uniform Rainfall.

RAINFALL DATA

The primary source of data for this study was the National Climatic Data Center (NCDC) located in Asheville, North Carolina. The data base included over three million pieces of hourly-station rainfall data, measured in 0.01-inch increments, from 252 stations uniformly distributed throughout the state with varying record lengths, collected between 1938 and 1983.

Because it is generally accepted that more than 10 years of rainfall records should be available for statistical analysis, stations with less than 10 years of record were omitted from further study. The records of some stations (approximately 35) were combined in order to increase the record length per station; combinations were considered justified when a station was moved only a short distance or replaced at the same location.

Data having a 15-minute sampling interval and 0.01-inch depth increment were also available from the National Climatic Data Center for 121 of the 252 stations. From these data, rainfall events of 15, 30, 45, 60, 90, and 120-minute durations were extracted.

RAINFALL FREQUENCY ANALYSIS

Extraction of Significant Rainfall Data

As a first step in the analysis, independent events, defined as those storms separated by at least 24 hours of zero rainfall, were identified. Each independent event was then scanned to determine the maximum 1, 2, 3, 6, 12, and 24-hour rainfall amounts and then evaluated for significance. A rainfall amount was considered significant if its value was at least as high as the threshold values of 0.6, 0.75, 0.9, 1.1, 1.3, and 1.5 inches, respectively, for the six durations shown above. These threshold values were chosen because they have a 90-percent exceedence probability in any one year, as extrapolated from the results of the study by Kerr, *et al.* (1970, pp. 20-35). The significant rainfall amounts for each of the six durations were then extracted for statistical frequency

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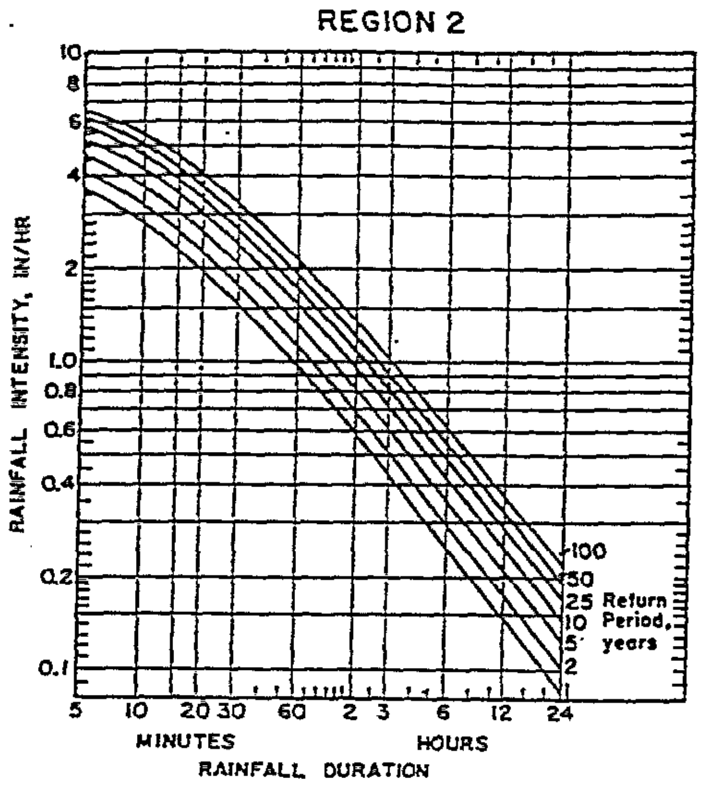

Figure 3. Region 2 Rainfall Intensity-Duration-Frequency Curves.

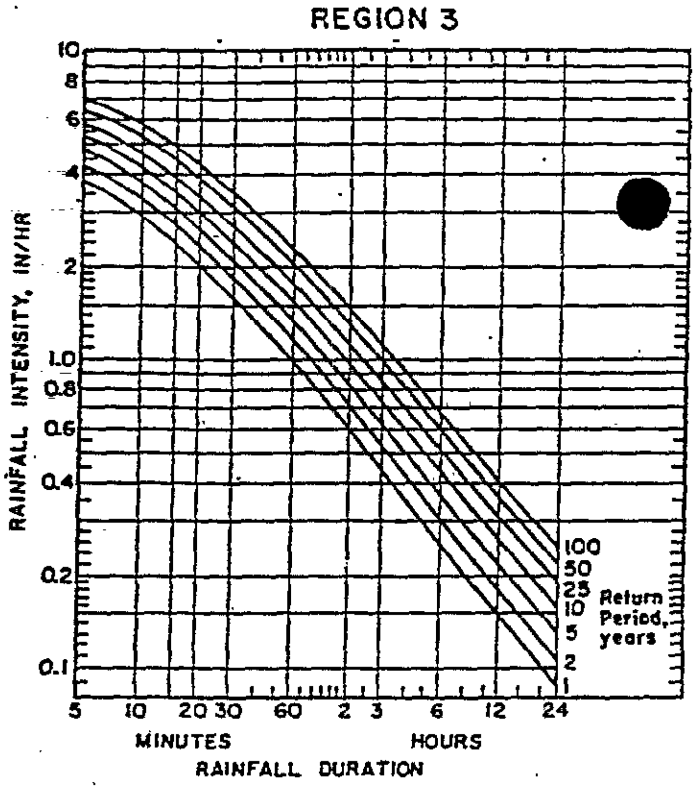

Figure 4. Region 3 Rainfall Intensity-Duration-Frequency Curves.

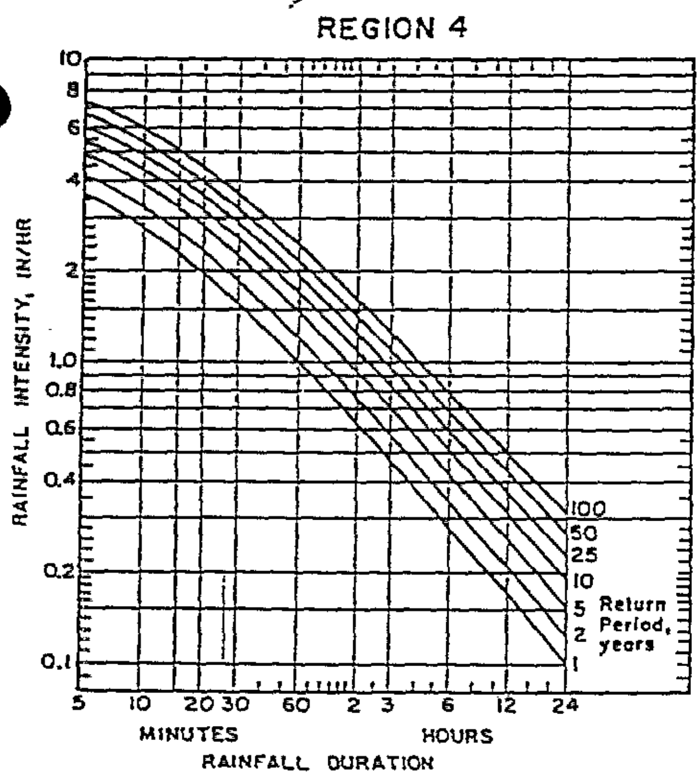

Figure 5. Region 4 Rainfall Intensity-Duration-Frequency Curves.

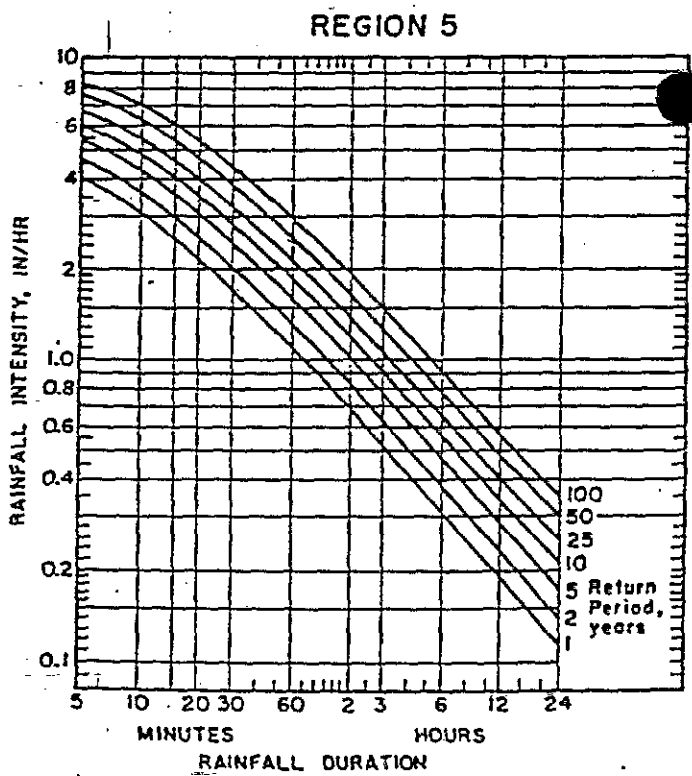

Figure 6. Region 5 Rainfall Intensity-Duration-Frequency Curves.

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3. Time of Concentration (T_C) = 15 min. (given)

4. Determine Rainfall Intensity Factor (i)
(i) = 4.9 in/hr - (from Plate 5-3)

5. Q = C(i)(A)
Q = .43(4.9)(80) = 168.56 cfs

Table 5-2
VALUES OF RUNOFF COEFFICIENT (C) FOR RATIONAL FORMULA

Land use	C	Land use	C
Business:		Lawns:	
Downtown areas	0.70-0.95	Sandy soil, flat, 2%	0.05-0.10
Neighborhood areas	0.50-0.70	Sandy soil, average, 2-7%	0.10-0.15
		Sandy soil, steep, 7%	0.15-0.20
Residential:		Heavy soil, flat, 2%	0.13-0.17
Single-family areas	0.30-0.50	Heavy soil, average, 2-7%	0.18-0.22
Multi units, detached	0.40-0.60	Heavy soil, steep, 7 %	0.25-0.35
Multi units, attached	0.60-0.75		
Suburban	0.25-0.40	Agricultural land:	
		Bare packed soil	
Industrial:		Smooth	0.30-0.60
Light areas	0.50-0.80	Rough	0.20-0.50
Heavy areas	0.60-0.90	Cultivated rows	
		Heavy soil no crop	0.30-0.60
Parks, cemeteries	0.10-0.25	Heavy soil with crop	0.20-0.50
		Sandy soil no crop	0.20-0.40
Playgrounds	0.20-0.35	Sandy soil with crop	0.10-0.25
		Pasture	
Railroad yard areas	0.20-0.40	Heavy soil	0.15-0.45
		Sandy soil	0.05-0.25
Unimproved areas	0.10-0.30	Woodlands	0.05-0.25
Streets:			
Asphaltic	0.70-0.95		
Concrete	0.80-0.95		
Brick	0.70-0.85		
Drives and walks	0.75-0.85		
Roofs	0.75-0.95		

Note: The designer must use judgement to select the appropriate C value within the range. Generally, larger areas with permeable soils, flat slopes and dense vegetation should have lowest (C) values. Smaller areas with dense soils, moderate to steep slopes, and sparce vegetation should be assigned highest (C) values.

Source: American Society of Civil Engineers

General Guidline:

C = .90 Impervious surfaces (Bituminous or concrete pavement, roofs, etc.)

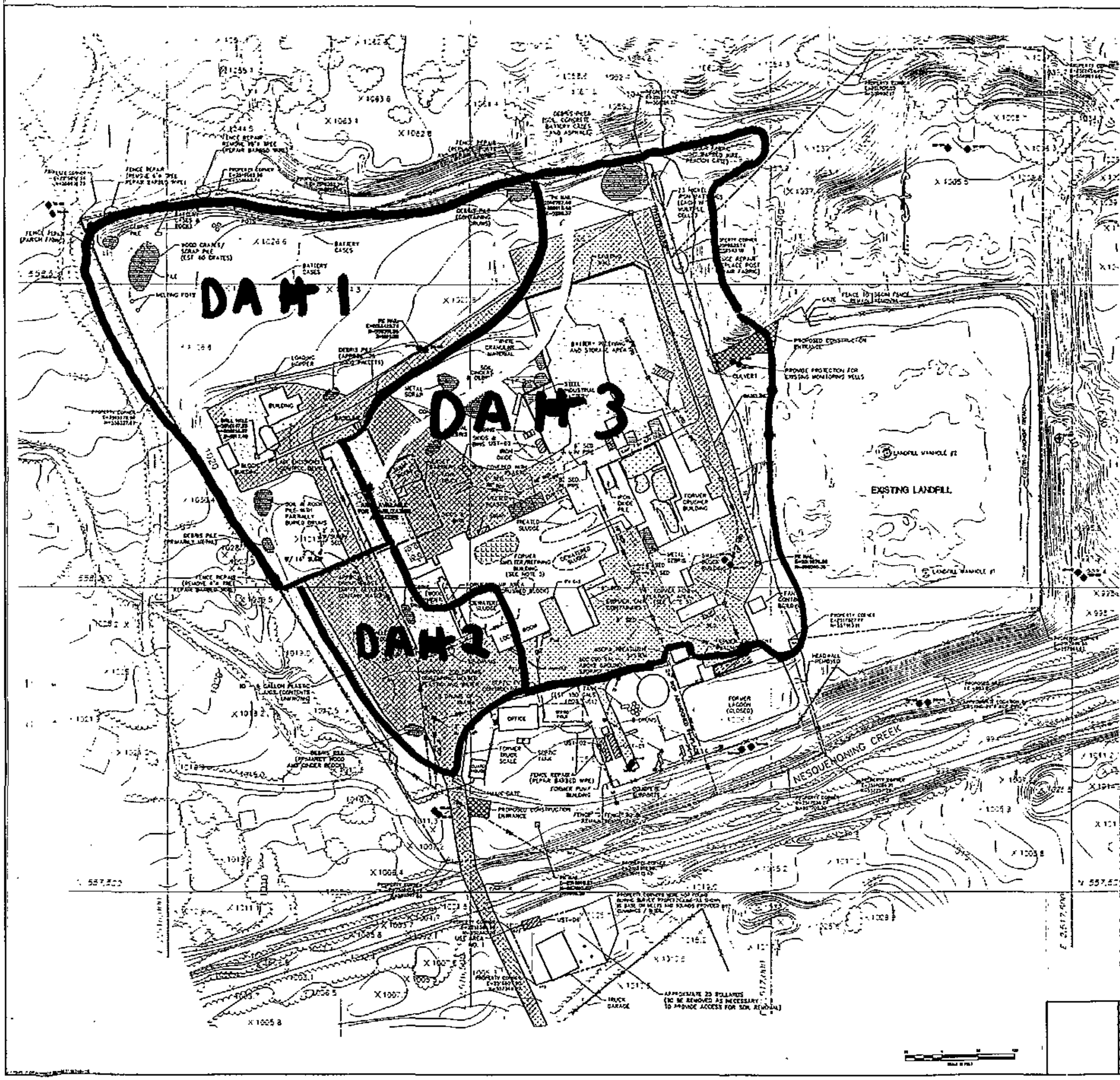
C = .50 Partially impervious surfaces (Crushed stone, loosely laid brick)

C = .30 Lawns or Grassy areas; sandy soils

TABLE 15.1.2
Summary of time of concentration formulas

Method and Date	Formula for t_c (min)	Remarks
* Kirpich (1940)	$t_c = 0.0078L^{0.77}S^{-0.385}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply t_c by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.
California Culverts Practice (1942)	$t_c = 60(11.9L^3/H)^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft	Essentially the Kirpich formula; developed from small mountainous basins in California (U. S. Bureau of Reclamation, 1973, pp. 67–71).
Izzard (1946)	$t_c = \dfrac{41.025(0.0007i + c)L^{0.33}}{S^{0.333}i^{0.667}}$ i = rainfall intensity, in/h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product i times L should be ≤ 500.
Federal Aviation Administration (1970)	$t_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from air field drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.

Source: Viessman Warren, Jr., Lewis Gary L. "Introduction to Hydrology" Harper Collins College Publishers, 1995.



WASTE PILE DISPOSAL SUMMARY

WASTE PILE TYPE	CONTENTS	NUMBER OF PILES	VOLUME C.Y.	DISPOSAL METHOD
LAGOON SOIL	SOIL W/ MISC DEBRIS	1	210	ON-SITE LANDFILL
UREA REMOVAL SLUDGES	TREATED/DEWATERED SLUDGES W/ MISC. DEBRIS	5	2,200	ON-SITE LANDFILL
CRUSHER BUILDING DUST PILES	CRUSHER BUILDING DUST (BATTERY WASTE)	5	1,100	STABILIZED AND ON-SITE LANDFILL DISPOSAL OR RESOURCE RECOVERY
IRON OXIDE	IRON OXIDE (BATTERY WASTE)	2	55	ON-SITE LANDFILL DISPOSAL OR RESOURCE RECOVERY
MISCELLANEOUS DEBRIS WASTE	METAL DEBRIS WOOD PALLETS CONCRETE ASPHALT	10	700 (NOTE 5)	ON-SITE LANDFILL DISPOSAL OR RESOURCE RECOVERY

LEGEND:

- PROPERTY BOUNDARY
- EXISTING CHAIN LINK FENCE
- EXISTING RAILROAD TRACKS
- EXISTING STORMSEWER
- SURVEYOR'S BASELINE (ESTABLISHED 1996)
- EXISTING ASPHALT PAVED SURFACE
- EXISTING CONCRETE PAVED SURFACE
- UNDERGROUND STORAGE TANK (UST)
- CRUSHER BUILDING DUST PILE
- MISCELLANEOUS DEBRIS/WASTE PILE
- BATTERY CASES
- SUMP/MANHOLE SEDIMENT
- LAGOON SOILS
- IRON OXIDE
- EXISTING DRAINAGE DIVERSION
- MONITORING WELL INSTALLED DURING RI
- MONITORING WELL INSTALLED DURING PRE-DESIGN INVESTIGATION

NOTES:

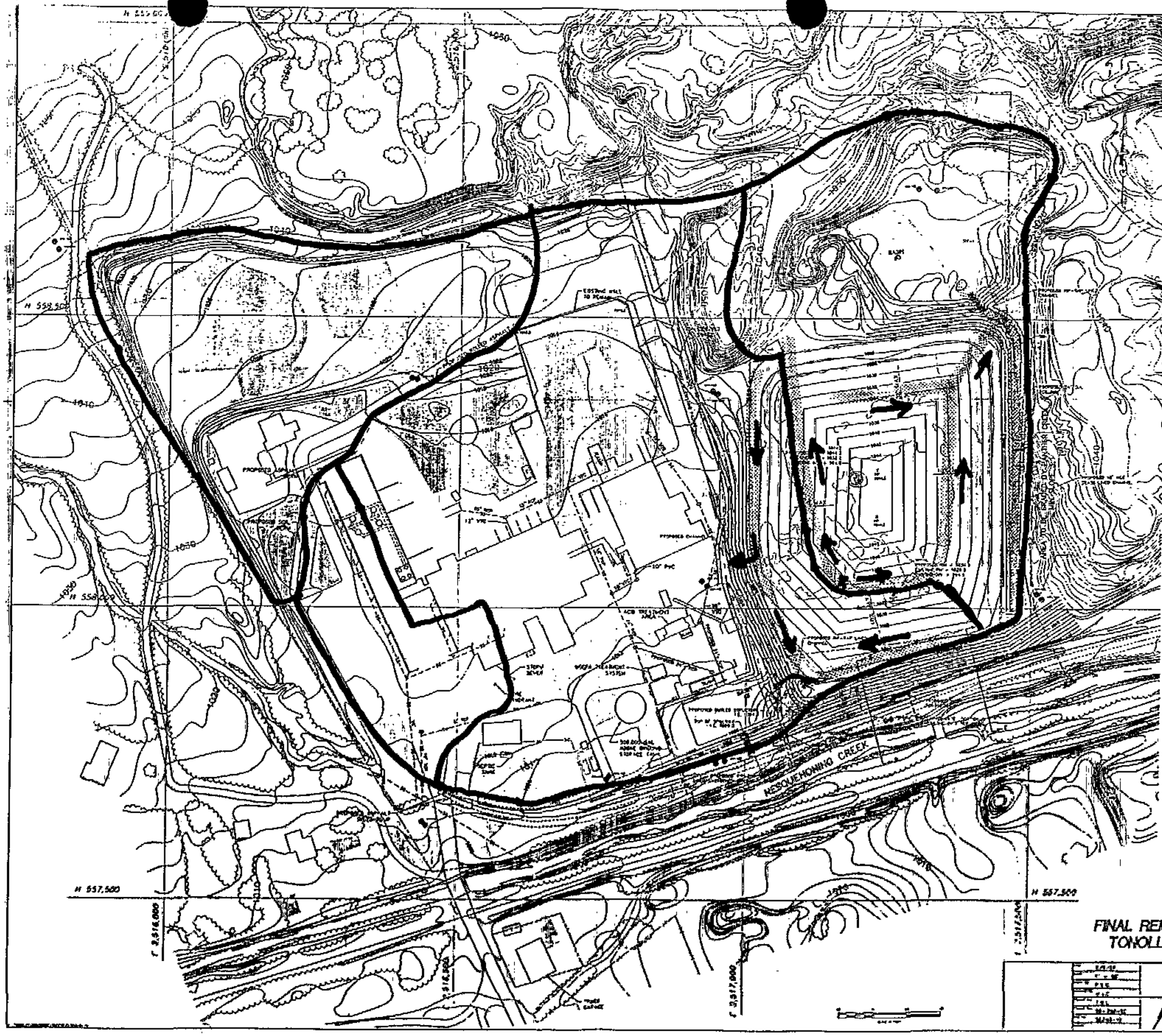
1. FENCE REPAIR LOCATIONS SHOWN ARE APPROXIMATE.
2. SITE CONTAINS DISTINCT PILES OF MATERIAL NOTED ON THIS PLAN, AS WELL AS A SIGNIFICANT AMOUNT OF WOOD, METAL, PLASTIC CHIPS AND OTHER DEBRIS SCATTERED ON THE GROUND SURFACE. CONTRACTOR WILL BE RESPONSIBLE FOR REMOVING ALL MATERIALS (UNLESS OTHERWISE NOTED) FOR PLACEMENT IN LANDFILL OR OFF-SITE DISPOSAL/RECYCLING AS REQUIRED BY PROJECT SPECIFICATIONS.
3. BASEMAP TAKEN FROM FIGURE 2-1 OF THE REMEDIAL DESIGN WORK PLAN PREPARED BY ELMWINGS / INTER CONSULTANTS, INC., DATED OCTOBER 3, 1995 (REVISED DECEMBER 21, 1995).
4. SITE PREDESIGN INVESTIGATION REPORT FOR DETAILED DESCRIPTION OF DEBRIS PILES LOCATED THROUGHOUT SITE AND WITHIN SITE BUILDINGS.
5. THE TOTAL VOLUME FOR MISCELLANEOUS DEBRIS INCLUDES APPROXIMATELY 130 CUBIC YARDS OF MISCELLANEOUS DEBRIS LOCATED IN FORMER SMELTER/REFINING BUILDING, STORAGE ANNEX AND LOCKER ROOM, AND APPROXIMATELY 20 CUBIC YARDS LOCATED IN ACID TREATMENT AREA.
6. SEDIMENT SHALL BE REMOVED FROM STORM SEWER PIPES, CATCH BASINS AND SUMPS AND DISPOSED IN ON-SITE LANDFILL OR SENT OFF-SITE FOR RESOURCE RECOVERY, AT THE BEGINNING AND END OF CONSTRUCTION ACTIVITIES.
7. NICKEL / IRON BATTERIES SHALL BE REMOVED FOR OFF-SITE DISPOSAL OR RESOURCE RECOVERY.

22 OCT 22

Drainage Boundary

PRE-FINAL REMEDIAL DESIGN SUBMISSION
TONOLLI CORP. SUPERFUND SITE
NESQUEHONING, PENNSYLVANIA

9/15/97	SITE PREPARATION PLAN	SHEET: 2
1" = 60'		
PSG		
PGS		
SML	Advanced GeoServices Corp.	
96-246-02		
96245-06		



[redacted]

LEGEND:

- PROPERTY BOUNDARY
- PROPOSED GROUNDWATER TREATMENT TRENCH
- PROPOSED TOP OF CAP CONTOUR
- FORMER BUILDING FOOT PRINT
- PROPOSED SWALE
- PROPOSED CONTOUR (OUTSIDE LIMIT OF REFUSE)
- RECONSTRUCTED FENCE
- MONITORING WELL INSTALLED DURING RI
- CHAIN LINK FENCE
- RAILROAD TRACKS
- STORMWATER PIPING
- EXISTING CONTOUR
- STORMWATER CATCH BASIN
- REQUIRED REMOVAL AREA

NOTES:

1. ON-SITE SURFACES NOT INCLUDED WITHIN EXCAVATION AREAS SHALL BE CLEANED OF ANY BATTERY CHIPS OR OTHER DEBRIS. PAVED ON-SITE SURFACES SHALL BE FREE OF ACCUMULATED SEDIMENT, SOIL OR OTHER MATERIAL WHICH MAY CONTAIN LEAD.
2. ON-SITE EXCAVATION AREAS NOT COVERED BY FINISHED LANDFILL OR BASIN SLOPES SHALL BE STABILIZED WITH STONE.
3. OFF-SITE AREAS SHALL RECEIVE 6 INCHES OF TOPSOIL AND BE REVEGETATED WITH GRASS.
3. LANDFILL CAP, EMBANKMENTS, AND BASINS SHALL RECEIVE 6 INCHES OF TOPSOIL AND REVEGETATED.

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FINAL REMEDIAL DESIGN SUBMISSION
TONOLLI CORP. SUPERFUND SITE
NESQUEHONING, PENNSYLVANIA

FINAL GRADING PLAN AND SITE RESTORATION		SHEET 17
Advanced GeoServices Corp.		

CHANNEL CALCULATIONS

CHANNEL CALCULATIONS (East Side of Landfill)

THE FOLLOWING CALCULATION DETERMINS THE REQUIRED DIMENSIONS FOR THE PROPOSED CHANNEL TO BE LOCATED ALONG THE EASTERN EDGE OF THE CAP.

THE RATIONAL METHOD WAS USED TO DETERMINE THE PEAK DISCHARGE FOR A 100 YEAR STORM EVENT. A RUNOFF COEFFICIENT OF 0.30 WAS USED, THIS VALUE REPRESENTS GRASS AREAS ON SANDY SOIL, WHICH IS THE VEGETATIVE COVER PROPOSED FOR THE CAP.

USING THE RATIONAL METHOD

$$Q_p = C I A$$

WHERE:

$C = 0.30$ (ASCE)
$A = 1.0$ acre
$I = 8.2$ in/hr (FROM ATTACHED RAINFALL INTENSITY CURVES FOR $t_c = 5$ MINUTES)

$$Q_p = (0.30)(1.0)(8.2) = 2.46 \text{ cfs}$$

FINDING DEPTH OF FLOW AND VELOCITY FOR CHANNEL USING MANINGS EQUATION

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$ (ASSUME RECTANGULAR CHANNEL)

WHERE:

$n = 0.027$ (GRASS LINED CHANNEL)
CHANNEL WIDTH = 10 ft
S = VARIES

$$2.5 = \frac{1.49}{0.027}(10d)\left(\frac{10d}{10+2d}\right)^{2/3} S^{1/2}$$

FOR $S = 0.01\ ft/ft$ (1%)

$d = 0.20\ ft$, BY TRIAL AND ERROR

$V = \frac{Q}{A} = \frac{2.5}{(10)(0.20)} = 1.3\ ft/sec$ (O.K. TO BE GRASS LINED)

FOR $S = 0.33\ ft/ft$ (33%)

$d = 0.06\ ft$, BY TRIAL AND ERROR

$V = \frac{Q}{A} = \frac{2.5}{(10)(0.6)} = 4.5\ ft/sec$ (USE RIP RAP PROTECTION $d_{50} = 3''$ or greater)

Checking capacity of at pre-vegetation conditions (Added 8/20/98)

Using the Rational Method

$Q_p = CIA$

Where:

$C = 0.50$ (ASCE)
$A = 1.0$ acres
$I = 8.2$ in/hr (100-yr rainfall intensity)

$Q_p = (0.50)(1.0)(8.2) = \underline{4.1\ cfs}$

Finding depth of flow

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$n = 0.027$ and $S = 1\%$

$$4.1 = \frac{1.49}{0.027}(10d)\left(\frac{10d}{10+2d}\right)^{2/3} S^{1/2}$$

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

By trial and error

$d = 0.21$ ft.

∴ Channel adequate for pre-vegetation conditions

SHEET 3 OF 12	PROJECT NO. 96-248	PROJECT NAME Tonolli			
BY TDT	DATE 8-20-98	DESCRIPTION Channel Calculation			
CHK. BY	DATE				

AR305444

CAP BENCH / SWALE CALCULATIONS

THE FOLLOWING CALCULATIONS DETERMINE THE REQUIRED DIMENSIONS FOR THE PROPOSED BENCH / SWALE ON THE CAP AND ALONG THE NORTHERN, WESTERN, AND SOUTHERN LANDFILL EMBANKMENT.

THE RATIONAL METHOD WAS USED TO DETERMINE THE PEAK DISCHARGE FOR A 100-YEAR STORM EVENT. A RUN OFF COEFFICIENT OF 0.30 WAS USED. THIS VALUE REPRESENTS GRASS AREAS ON SANDY SOIL WHICH IS THE VEGETATIVE COVER PROPOSED FOR THE CAP

CALCULATION:

USING THE RATIONAL METHOD.

$$Q_p = CIA$$

WHERE:

$C = 0.30$ (ASCE)
$A = 1.5$ acres (LARGEST AREA DRAINING TO ANY BENCH)
$I = 8.2$ in/hr ($t_c = 5$ MINUTES)

$$Q_p = (0.30)(1.5)(8.2)$$

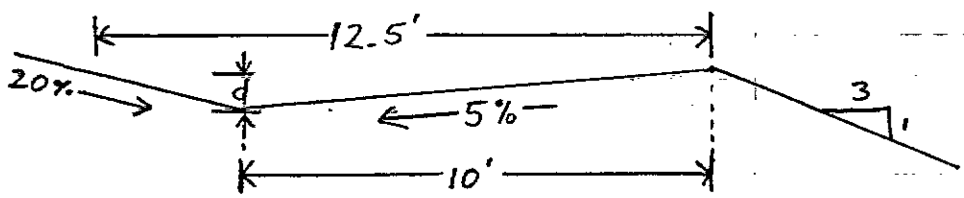
$$Q_p = 3.7 \text{ cfs}$$

FINDING THE DEPTH OF FLOW AND VELOCITY IN BENCH/SWALE USING MANING'S EQUATION

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

WHERE:

$n = 0.027$ (GRASS LINED CHANNEL)
$S = 1\%$



USING FLOW MASTER SOFTWARE

$d = 0.4$ ft
$V = 1.9$ ft/sec (GRASS-LINED IS ACCEPTABLE)

THE CHANNEL IS CAPABLE OF HANDLING A 100 YEAR STORM EVENT.

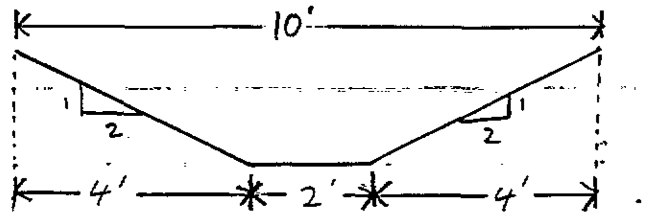
THE FOLLOWING CALCULATION DETERMINES THE DEPTH OF FLOW AND VELOCITY OF A 100 YEAR DISCHARGE FROM THE BENCHES DOWN THE SIDE OF THE CAP.

USING MANINGS EQUATION

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

WHERE:

$n = 0.027$
$S = 20\%$
$Q = 3.7$ cfs



USING "FLOW MASTER" SOFTWARE

$d = 0.2$ ft
$V = 7.52$ ft/sec (USE RIP RAP $d_{50} = 3''$)

RE-RUN CALCULATION FOR RIP RAP LINED CHANNEL

WHERE:

$n = 0.04$
$S = 20\%$

USING "FLOW MASTER" SOFTWARE

$d = 0.26$ ft
$V = 5.76$ ft/sec ($d_{50} = 3''$ ACCEPTABLE)

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"Engineering for the Environment"™

Checking Cap Bench / Swale Capacity for pre-vegetation Conditions

Using the Rational Method

$$Q_p = CIA$$

Where:

C = 0.50 (ASCE)
A = 1.5 acres (Largest Area Draining to Any Bench)
I = 8.2 in/hr (tc = 5 minutes, 100-yr. storm)

$$Q_p = (0.50)(1.5)(8.2)$$

$$\underline{Q_p = 6.2 \text{ cfs}}$$

Finding depth of flow using the Manning Equation

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Where

n = 0.027
S = 1%

Using Flow Master Software

$$\underline{d = 0.48 \text{ ft}}$$

∴ Channel adequate for pre vegetation conditions

SHEET 7 OF 12 PROJECT NO. 96-248 PROJECT NAME Tunelli
BY T.DT DATE 8/20/98 DESCRIPTION Channel Calcs
CHK. BY DATE

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

Temporary Top - of Waste Drainage Swale

The Following calculations determine required dimensions for the proposed channel to be located along northern, western, eastern, and southern landfill embankment.

The Rational Method was used to determine the peak discharge for a 100 year storm event. A run-off coefficient of 0.50 was used. This value represents rough bare packed soil.

Calculation:

Using the rational method:

$Q_p = CIA$

where $C = 0.50$ (American Society of Civil Engineers)

A = Drainage Area = 2.2 acres

$I = 8.2$ in/hr ($t_c = 5$ minutes)

So $Q_p = (0.5)(2.2)(8.2)$

$Q_p = 9.0$ cfs

SHEET 8 OF 12 PROJECT NO. 96248.79 PROJECT NAME TONOLLI

BY JWP DATE 7/22 DESCRIPTION Top of Waste Channel Calc.

CHK. BY TDt DATE 7/22

AR305449

ADVANCED GEOSERVICES CORP.

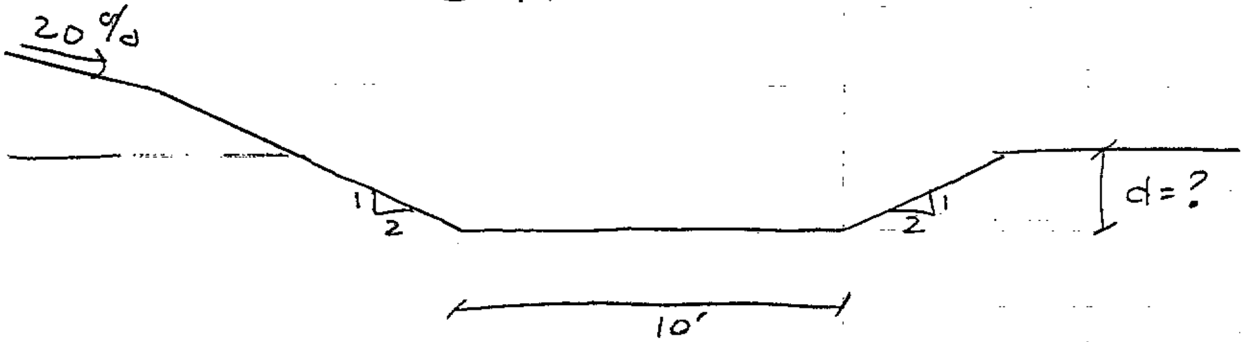
"Engineering for the Environment"™

Finding the depth of flow and velocity in channel using Mannings Equation.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Where n = 0.018 (earth channel, straight & uniform; clean, recently completed)

S = 1%



Using Flow Master Software:

d = 0.26 ft
V = 3.25 fps

SHEET 9 OF 12 PROJECT NO. 9C-24879 PROJECT NAME Tonelli
BY JWP DATE 7/22 DESCRIPTION Top of Waste Channel Calc
CHK. BY TDT DATE 7/22

AR305450

3. Time of Concentration (T_c) = 15 min. (given)

4. Determine Rainfall Intensity Factor (i)
(i) = 4.9 in/hr - (from Plate 5-3)

5. Q = C(i)(A)
Q = .43(4.9)(80) = <u>168.56 cfs</u>

Table 5-2
VALUES OF RUNOFF COEFFICIENT (C) FOR RATIONAL FORMULA

Land use	C	Land use	C
Business:		Lawns:	
Downtown areas	0.70-0.95	Sandy soil, flat, 2%	0.05-0.10
Neighborhood areas	0.50-0.70	Sandy soil, average, 2-7%	0.10-0.15
		Sandy soil, steep, 7%	0.15-0.20
Residential:		Heavy soil, flat, 2%	0.13-0.17
Single-family areas	0.30-0.50	Heavy soil, average, 2-7%	0.18-0.22
Multi units, detached	0.40-0.60	Heavy soil, steep, 7 %	0.25-0.35
Multi units, attached	0.60-0.75		
Suburban	0.25-0.40	Agricultural land:	
		Bare packed soil	
Industrial:		Smooth	0.30-0.60
Light areas	0.50-0.80	Rough	0.20-0.50
Heavy areas	0.60-0.90	Cultivated rows	
		Heavy soil no crop	0.30-0.60
Parks, cemeteries	0.10-0.25	Heavy soil with crop	0.20-0.50
		Sandy soil no crop	0.20-0.40
Playgrounds	0.20-0.35	Sandy soil with crop	0.10-0.25
		Pasture	
Railroad yard areas	0.20-0.40	Heavy soil	0.15-0.45
		Sandy soil	0.05-0.25
Unimproved areas	0.10-0.30	Woodlands	0.05-0.25
Streets:			
Asphaltic	0.70-0.95		
Concrete	0.80-0.95		
Brick	0.70-0.85		
Drives and walks	0.75-0.85		
Roofs	0.75-0.95		

Note: The designer must use judgement to select the appropriate C value within the range. Generally, larger areas with permeable soils, flat slopes and dense vegetation should have lowest (C) values. Smaller areas with dense soils, moderate to steep slopes, and sparce vegetation should be assigned highest (C) values.

Source: American Society of Civil Engineers

General Guidline:

C = .90 Impervious surfaces (Bituminous or concrete pavement, roofs, etc.)

C = .50 Partially impervious surfaces (Crushed stone, loosely laid brick)

* C = .30 Lawns or Grassy areas; sandy soils

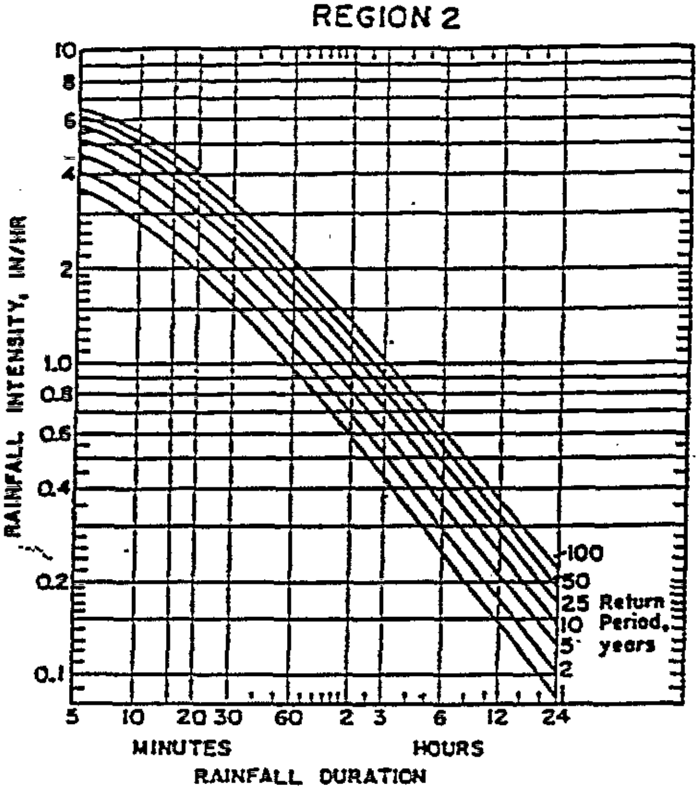


Figure 3. Region 2 Rainfall Intensity-Duration-Frequency Curves.

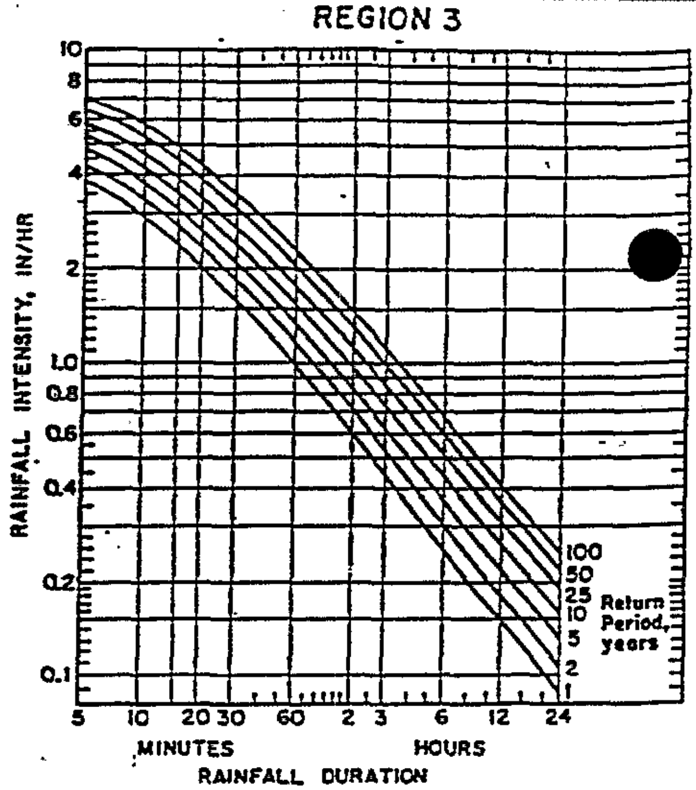


Figure 4. Region 3 Rainfall Intensity-Duration-Frequency Curves.

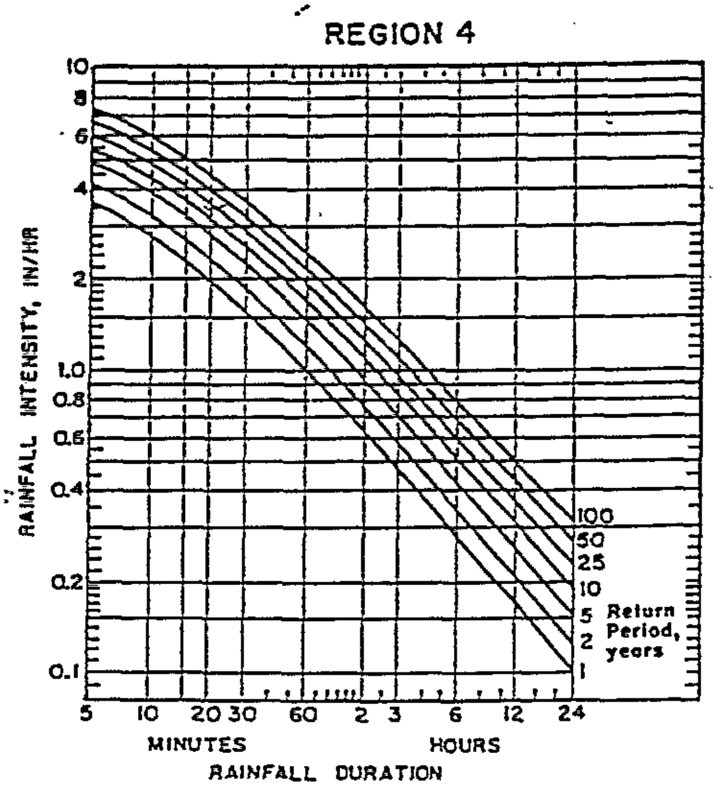


Figure 5. Region 4 Rainfall Intensity-Duration-Frequency Curves.

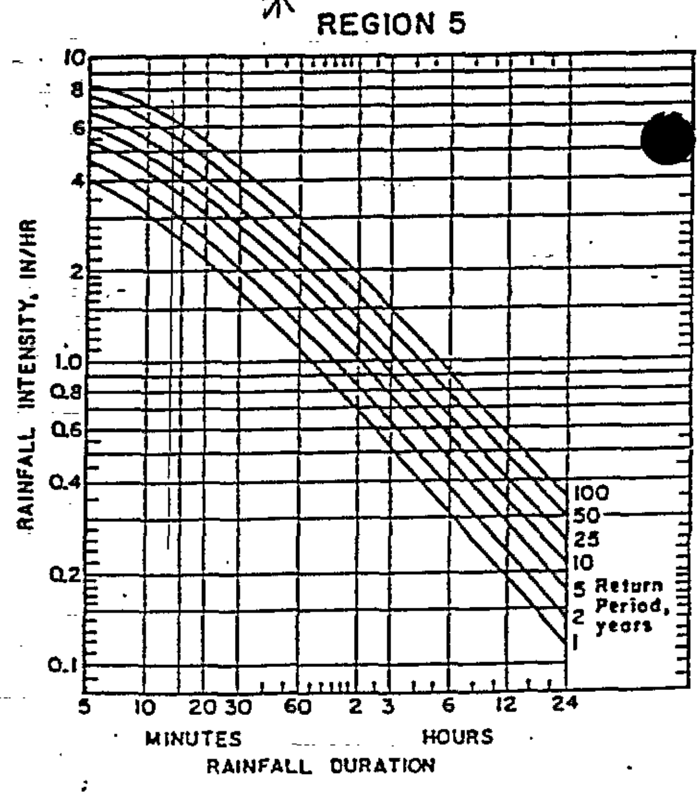


Figure 6. Region 5 Rainfall Intensity-Duration-Frequency Curves.

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AR305452

Richard H. French

OPEN-CHANNEL HYDRAULICS

McGraw-Hill, Inc.
New York St. Louis San Francisco Auckland Bogotá
Caracas Lisbon London Madrid Mexico City Milan
Montreal New Delhi San Juan Singapore
Sydney Tokyo Toronto

12 of 12

Type of channel and description	Minimum	Normal	Maximum
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
1. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
j. Vegetal lining	0.030		0.500
C. Excavated or dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033

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R305454

Type of channel and description	Minimum	Normal	Maximum
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as no. 4, more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy, reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060

TABLE 4.7c Maximum Permissible Velocities for Rock Lined Channels and Riprap

NSA NO.	Graded Rock Size (In.) Max.	D_{50}	Min.	Permissible velocity fps*
R-1	1.5	.75	NO. 8	2.5
R-2	3	1.50	1	4.5
R-3	6	3	2	6.5
R-4	12	6	3	9.0
R-5	18	9	5	11.5
R-6	24	12	7	13.0
R-7	30	15	12	14.5

* Permissible velocities based on rock at 165 lbs. per cubic foot. Adjust velocities for other rock weights used. See Figure 4.6

TABLE 4.7d Maximum Permissible Velocities for Reno Mattress and Gabions

Type	n	Thickness inches	Rock fill Gradation-in.	Permissible* Velocity-fps
Reno Mattress	.025	6	3 - 6	13.5
	.025	9	3 - 6	16.0
	.025	12	4 - 6	18.0
Gabion	.027	18 +	5 - 9	22.0

* Permissible velocities may be increased by the introduction of sand mastic grout. Refer to manufacturers recommendations/specifications for permissible velocities.

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IMAGERY COVER SHEET
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OPERABLE UNIT 00

ADMINISTRATIVE RECORDS- SECTION III VOLUME L

REPORT OR DOCUMENT TITLE LANDFILL Cap Redesign Report

DATE OF DOCUMENT 8/26/98

DESCRIPTON OF IMAGERY Landfill Preparation and Top of Waste Plan

NUMBER AND TYPE OF IMAGERY ITEM(S) 1 map

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DOC ID 141570
PAGE # 305458

IMAGERY COVER SHEET
UNSCANNABLE ITEM

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ADMINISTRATIVE RECORDS- SECTION III **VOLUME** L

REPORT OR DOCUMENT TITLE LANDFILL Cap Redesign Report

DATE OF DOCUMENT 8/26/98

DESCRIPTON OF IMAGERY Landfill Final Grading plan

NUMBER AND TYPE OF IMAGERY ITEM(S) 1 map

EPA REGION III
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PAGE # 305459

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UNSCANNABLE ITEM

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ADMINISTRATIVE RECORDS- SECTION III **VOLUME** L

REPORT OR DOCUMENT TITLE Landfill Cap Redesign Report

DATE OF DOCUMENT 8/26/98

DESCRIPTON OF IMAGERY Cap Erosion + Sediment Control Plan

NUMBER AND TYPE OF IMAGERY ITEM(S) 1 map

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PAGE # AR305460

IMAGERY COVER SHEET
UNSCANNABLE ITEM

SITE NAME Tonolli

OPERABLE UNIT 00!

ADMINISTRATIVE RECORDS- SECTION III VOLUME L

REPORT OR DOCUMENT TITLE Landfill Cap Redesign Report

DATE OF DOCUMENT 8/26/98

DESCRIPTON OF IMAGERY Landfill Capping Details

NUMBER AND TYPE OF IMAGERY ITEM(S) 1 Oversized map

EPA REGION III
SUPERFUND DOCUMENT MANAGEMENT SYSTEM

DOC ID 141570
PAGE # AR305461

IMAGERY COVER SHEET
UNSCANNABLE ITEM

SITE NAME Tonolli

OPERABLE UNIT 00

ADMINISTRATIVE RECORDS- SECTION III VOLUME L

REPORT OR DOCUMENT TITLE Landfill Cap Design Report

DATE OF DOCUMENT 8/26/98

DESCRIPTON OF IMAGERY Western + Northern Embankment Slope Capping Details

NUMBER AND TYPE OF IMAGERY ITEM(S) 1 oversized map