CTO-203-5082-0130

N00129.AR.000216
NSB NEW LONDON
5090.3a

DESIGN ANALYSIS
BASIS OF DESIGN AND CALCULATIONS

AREA A LANDFILL

FINAL
SUBMISSION

DATE 5 2 94
MONTH - DAY - YEAR

Prepared For:

Northern Division
Naval Facilities Engineering Command
10 Industrial Highway
Lester, PA 19113-2090

Navy Contract No.: N62472-88-C-1294

Prepared By:

Atlantic Environmental Services, Inc.
P.O. Box 297 • 188 Norwich Avenue
Colchester, CT 06415

Telephone: (203) 537-0751
Fax: (203) 537-6347

Atlantic Project No.: 1256-31-05

May 2, 1994

ATLANTIC

TABLE OF CONTENTS

SECTION		PAGE
1.0	INTRODUCTION	1
1.1	Purpose and Scope	1
2.0	SITE CHARACTERISTICS	3
2.1	Site Operations	3
2.2	Surface Characteristics	3
2.3	Limits of Landfill	4
2.4	Subsurface Conditions	5
2.5	Other Conditions	5
3.0	DESIGN REQUIREMENTS	7
3.1	Operations After Construction	7
3.2	Surface Requirements and Storm Drainage	8
	3.2.1 Storm Drainage System	9
	3.2.2 Existing Storm Drains	11
3.3	Subsurface Requirements	11
3.4	Other Design Requirements	13
	3.4.1 Pavement Design	13
	3.4.2 Utilities	13
	3.4.3 Proof Rolling	13
	3.4.4 Monitoring Wells	14
	3.4.5 Maintenance and Repair	14
	3.4.6 Regulatory Standards	15
	3.4.7 Structures	17
4.0	CONCLUSIONS	18

LIST OF APPENDICES

Appendix A: Supplemental Soil Sampling Results
Appendix B: Crane Loading on Existing Soils and Cap Design
Appendix C: Special Fence Post Base
Appendix D: Earthwork and Materials Balance
Appendix E: Riprap Sizing and Flat Bottom Swale
Appendix F: Storm Drainage Calculations
Appendix G: Settlement Analysis and Bearing Capacity Analysis
Appendix H: Methane Gas Survey Results
Appendix I: Environmental Permit Checklist

1.0 INTRODUCTION

1.1 Purpose and Scope

The Naval Submarine Base in New London (NSB-NLON) consists of approximately 547 acres of land and associated buildings in southeastern Connecticut in the towns of Ledyard and Groton. NSB-NLON is on the east bank of the Thames River, approximately 6.0 miles north of Long Island Sound. NSB-NLON was placed on the National Priorities List (NPL) on August 28, 1991, by the U.S. Environmental Protection Agency (U.S. EPA) pursuant to the comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) of 1980.

The purpose of this project is to provide an interim remedial design for the Area A Landfill at NSB-NLON in Groton, Connecticut. This interim remedial design includes source excavation and containment (capping) action. Excavated contaminated material (source excavation) consists of two hot spots to be disposed of in accordance with applicable regulations. This activity has been included with excavation and disposal under a separate contract, ''Spent Acid and PCB Contaminated Soil Disposal,'' and is to be completed prior to this contract.

This contract, ''Area A Landfill Cap,'' represents the actions necessary to prevent the release of contaminants into the environment and prevent human exposure to the contaminants. This objective is achieved through final capping of the Area A Landfill and diversion of stormwater and groundwater. The Navy's goal is to begin interim remedial actions at NSB-NLON as quickly as possible to protect human health and the environment and to comply with applicable or relevant and appropriate requirements (ARARs).

The basis for this design with respect to the limits of excavation, extent of the landfill, and levels of contaminants is derived from prior studies conducted for Northern Division by

Atlantic Environmental Services, Inc. (Atlantic). The reports generated from these studies are listed as follows.

- Design Work Plan, Area A Landfill, January 24, 1994.
- Design Work Plan, Interim Remedial Design, Spent Acid Storage and Disposal Area and Area A Landfill-Concrete Pad, October 19, 1993.
- Focused Feasibility Study, Area A Landfill, March 28, 1994.

2.0 SITE CHARACTERISTICS

2.1 Site Operations

The Area A Landfill is used by the U.S. Government for several purposes. There is a fenced area approximately 140 feet by 530 feet for deployed parking, where vehicles are stored. There are several sheds, used for storage, and a quonset hut on concrete blocks from which base personnel monitor operations of Area A. There is also an electrical shop, Building 496, which consists of a metal structure on a concrete foundation. Between the quonset hut and Building 496 there is a bituminous concrete pad, approximately 160 feet by 100 feet, which is used for storage and for testing cranes. A portion of what is normally stored on this pad includes steel and concrete weights used for crane testing. There is a salt storage shed adjacent to the electrical shop and sand stockpiles near the north edge of the landfill. The salt and sand are mixed on site, when required, to treat snow and ice throughout the Subase. The remaining surface area of the landfill is used to store various items and materials, some of which require cranes to load, unload, or move. The Subase personnel responsible for operating cranes at this site have indicated that existing soils provide good support with settlements under outriggers of one to two inches.

2.2 Surface Characteristics

Aside from the bituminous concrete pad, the site surface is almost exclusively sandy gravel. The surface generally slopes in a northeasterly direction at one to four percent and drainage follows various paths (least resistance), depending on surficial disturbance and stored objects, which vary. The constant traffic and poor drainage have caused potholes and ruts to form throughout the Area A Landfill. In addition to rain falling on this area, portions of the sideslope south of the landfill drain directly onto the landfill. There are two pipes crossing under the landfill which can carry some of the flow; however, flow is often blocked and

stormwater overflows onto the landfill surface.

The area south of the landfill is generally wooded, sloping at approximately 20 to 30 percent down onto the landfill, with portions of exposed bedrock and boulders. The slope at the north side of the landfill is northward into wetlands, at approximately 30 to 40 percent. This north slope is covered with mixed vegetation, mostly brush and thorns with some trees. This vegetation grew after placement of the landfill and this sideslope, considered part of the landfill, is potentially contaminated.

2.3 Limits of Landfill

The determination of the limit of landfill material was one of the most important items required for design of this remedial measure. The new protective cap is intended to seal off this landfill material, but the history of the site makes landfill limits difficult to determine. The prior feasibility studies set approximate limits and, after final topographic plans were available, all available information was used to portray this limit as accurately as possible. In addition, supplemental soil samples were collected at the eastern limit of the landfill to determine whether fill in this area is contaminated. The test results, which are included in the appendices, indicate that fill in this area is not contaminated. The history of the site, based on aerial photographs and other information, is as follows.

The area was wooded, sloping down into a stream which flowed westerly, eventually to the Thames River. After 1952, the slope was cut and flattened to provide a mobilization area for work involving hydraulic fill. A portion of this material was also used to construct a dike, to contain the dredged fill. This dredged material was placed in variable depths, resulting in a surface elevation of approximately 70 (mean sea level datum). During the period from before 1957 to 1973, the site was used as a landfill. This use resulted in a layer approximately 10 to 20 feet in thickness. The landfill was then covered with the sandy gravel which is the current

surface.

By compiling aerial photographs, historic and current topographic maps, boring logs and test results, and using judgement based on site visits to visually confirm the present conditions, this limit was set as shown on the plans. The limit of landfill material, as estimated, is believed to be conservative, but if excavation work during construction reveals additional material which appears to be landfill, it can be excavated and placed within the area to be capped.

2.4 Subsurface Conditions

The groundwater flow is generally toward the north, with the westerly end flowing northwest. The slope down to the landfill is generally shallow bedrock and thus carries one component of groundwater at a fairly shallow depth into the landfill. The wetlands, on the north side of the landfill, are a result of the dredge fill placed there with the water table approximately at the surface. This water table extends under the landfill, which is supported by the dredged material, with the water table varying in height between the toe of the shallow ledge (elevation approximately 90) and the wetlands (approximately elevation 70). Therefore, groundwater flow is passing through the landfill material, potentially leaching contaminants toward the wetlands.

2.5 Other Conditions

The government intends to remove stored materials and vehicles from this site prior to construction of the landfill cap. The one exception pertains to two large metal and wood structures, which will be cut up and disposed of as part of this contract. These two structures are estimated to weigh 25 tons each and are commonly referred to as "camels." They are (floating) fenders originally used to hold ships in position during maintenance and construction work.

The only known subsurface utility located on site is a grounding well located approximately 50 feet east of Building 496. The cover of this well will be raised to match new

grades. The grounding wire connecting to this manhole will be lowered to match new grades, where necessary. Also, there is an overhead electric service on poles, some of which include luminaires, primarily to illuminate the deployed parking. There are several monitoring wells which have been installed during the studies of this site. They will need to remain to allow for continued monitoring after construction activities to determine the elevation of groundwater and extent of contamination. Groundwater sampling results could then be compared to pre-installation results to quantify improved conditions.

The government has indicated the need to continue some operations during this contract performance period. The contractor will need to take this requirement into account in managing or sequencing work activities. Depending on the season, salt and sand stockpiles may be required for the government to remain operational. When a significant area of landfill cap is completed, it may be made available to the government for storage or other purposes.

3.0 DESIGN REQUIREMENTS

3.1 Operations After Construction

The government has indicated that Building 496 will remain. At the conclusion of the work, grading will provide access to both doors and electrical service will be provided. The salt storage shed is to be removed, and a similar structure will be provided as part of this contract. This salt shed and the relocated sand stockpiles will be placed on a new bituminous concrete pad. This pad will armor the crushed stone cap and prevent mixing of the cap materials with sand and salt or damage to the cap caused by loading equipment.

The contract will also provide a second new bituminous concrete pad for testing cranes. The new location will provide more open area so the boom can be turned a full 360° circle. Cranes will also operate, for loading and unloading of storage items, throughout the new cap. The calculations for this cap are included in the appendices. Signs will be posted, similar to signs provided at the DRMO cap, requiring the use of outrigger supports during crane operations and warning against excavation. The deployed parking will be in approximately the same size, shape, and location as it currently exists but will have a bituminous concrete pavement. A minimum 20 feet wide travelway will be provided north and south of the new fencing for security requirements. This new 7-foot chain-link fence will have a three-strand, barbed wire top. Construction of fencing on the cap requires specially constructed post foundations. Gate and other important posts will be standard concrete foundations, cut through the geosynthetic cap materials, and sealed water tight. The line posts (10-foot spacing) between these main posts will be set in shallow concrete foundations to avoid a series of cap perforations. The penetrations would tend to weaken the cap and result in greater potential of surface-water leakage through the cap. The calculations for these special foundations are included in the appendices.

3.2 Surface Requirements and Storm Drainage

The grading of this site received considerable effort due to the special nature of this site. The existing sand-gravel surface becomes subgrade to the new cap. It is desirable to avoid concentrating surface flow, which would require drainage structures through the cap and piping within the landfill, and which could also cause erosion damage. The grading has therefore been designed to smooth the uneven site into a uniformly sloped surface that rain water will cross as sheet flow. This is only possible because all existing flow that runs onto the site will be intercepted and diverted around the landfill, with the only surface flow being what falls directly onto the cap.

This issue is further complicated by the potentially contaminated nature of the existing surface material, which limits where excavated material can be placed. If earthwork balance were to leave excess material, the material may require special disposal. This disposal requirement would be expensive and therefore should be avoided. The eastern end is fairly low in elevation, closer to wetland grades; the grading was designed to raise this landfill elevation, while still providing approximate earthwork balance. Some borrow (clean) material from off site will be required to construct this project but it has been kept to a reasonable quantity and is much less expensive than disposal of contaminated material. Earthwork calculations are included in the appendices. The government requested a "filter strip" along the north edge of the new Area A cap similar to the one to be provided at the Defense Reutilization and Marketing Office (DRMO) site. This filter strip will consist of uniform crushed stone sized to trap any particles from materials stored on site, to prevent them from washing into the wetlands. The bottom half of the slope down to the wetlands will be seeded, the result of which will tend to further filter the flow prior to entering the wetlands. The stone filter strip will also reduce chances of erosion of the new slope by uniformly distributing the flow onto and into the surface

soils.

During construction, erosion and sedimentation controls will be maintained to protect the wetlands. Storm drainage from the upper (south) slope will be intercepted and diverted through erosion and sedimentation controls until the new storm drain system is in place.

The existing slope above the wetlands includes landfill material and has been considered as contaminated for this cap design. The upper portion of this slope is to be capped to prevent rain-water infiltration which could leach contaminants. The lower limit of this slope will not be "capped," so any groundwater coming through the landfill will not be trapped. Trapped groundwater, if allowed to build up, could result in slope failure. This noncapped area will also allow water to reach plant roots to keep new ground cover alive. Existing vegetation on this slope will be cleared and grubbed and disposed of. The new ground cover will be from a seed mix that does not require mowing.

3.2.1 Storm Drainage System

The new storm drainage system has been carefully designed to intercept all surface flow from south of the new landfill cap, and convey it, in pipes, east and then north around the end of the cap. The runoff will be carried into the wetlands in a flat riprap swale to keep velocities low and prevent erosion. This system will also be sized for a 25-year storm event using reinforced concrete pipe (RCP). Other piping materials may be used, provided they have flow (based on Manning's n value of 0.013) equal to or better than RCP. Normal design practice would require using a 10-year storm event but, due to the environmental concerns and regulations, a 25-year storm event is required. The design provides for pipes flowing almost full but as "open-channel" type flow (i.e., not under pressure). This design provides a safety factor for larger storm events which could put the system under pressure. Additional factor of safety may also result from the storage capacity in the surface swales when the system is under

pressure (i.e., it could provide detention volume). This capacity is not a requirement and has not been calculated. The calculations of the 25-year flows, pipe capacities, and related drainage calculations are included in the appendices.

The components for this system have been chosen to minimize potential clogging. The existing storm drains at the site are ineffective due to clogging. All storm drain systems should be maintained but typically are not. The new storm drain system here will be more critical because it will be protecting a capped landfill. Type C-G catch basin tops, a Connecticut Department of Transportation (CTDOT) standard, provide more opening area for flow into the piped system than types C or C-L. Other openings are direct pipe connections with concrete headwalls to prevent flow from bypassing.

This storm drain system receives surface flow from wooded areas and, therefore, no oil separators have been provided. Protection from oil in runoff from parking lots south of the wooded area, if provided, should be at the outlet from these lots. No work at these parking lots has been included in this contract.

The new storm drains, if installed in potentially contaminated materials, could result in transport of contaminants to the pipe outlet and into the wetlands. Much of the system has been placed south of the landfill limit previously described; however, this placement was not always feasible. In the areas along the eastern portion of the site, where the landfill material was filled against existing rock surfaces, the new pipes are to be buried adjacent to the rock to minimize blasting. In this case, the potentially contaminated materials will be removed and used as fill within the area to be capped. The pipe trench will then be sealed off from the landfill with a geomembrane and then backfilled with clean material. Clean backfill will come from excess on-site material from outside the landfill limit or borrow from off-site sources. The route of this pipe, in order to minimize pipe within landfill material, is longer than would otherwise be

required. The final pipe in the new system passes through the southeastern corner of the landfill. This pipe will be sealed on both sides.

This separation of clean versus potentially contaminated materials was carefully considered in calculating earthwork balance, as noted in subsection 3.2.

3.2.2 Existing Storm Drains

Current landfill regulations recommended practice is that storm drains not pass through the site. As such, the backfilled trench can provide a migration pathway for groundwater and leachate. The two existing pipes shall be removed under this contract. Removal of the pipes alone does not undo this situation but, by sealing the ends of the trench with concrete cutoff walls, this flow tendency can be virtually eliminated. Additional reduction of potential flow will result from compaction of the trench after pipe removal.

3.3 Subsurface Requirements

As noted previously, the storm-drain piping is to be placed along the southern edge of the landfill, against the toe of the shallow ledge which extends from the slope down under the edge of the landfill. Several borings and probes were used as a basis to set the pipe locations to avoid large rock excavations. Some rock removal will be required, including sideslope placement of a groundwater interceptor pipe system. Blasting of rock is permitted but special Subase instructions will apply, due to the proximity of the weapons center. These requirements will be noted in the specifications.

Due to the nature of the lower level soils, a settlement analysis was conducted. The calculations are included in the appendices. The general conclusion is that some minor settlement may occur due to the weight of the new cap, but the settlement is not expected to result in any special design or construction methods. This minor settlement, estimated at 1.5 inches (maximum) in 15 years, will probably not be noticed and no damage is expected from

its occurrence.

Tests, the results of which are included in the appendices, were conducted to determine whether site-generated gasses were present. These tests found methane in sufficient quantity to warrant the design of vents. These vents are to be placed at suitable locations to ensure escape of any methane from under the cap and thereby prevent buildup of potentially explosive volumes. The materials within the cap generally are not expected to generate large volumes of methane and it will likely continually decrease in potential as the materials generating the gas are depleted. Vents will be located to avoid close proximity to structures that could trap such released gases.

The groundwater interceptor pipe, as previously noted, will be placed at or slightly into the shallow rock surface on the south slope. The purpose of this system is to reduce the groundwater flow into the landfill, which will decrease potential leaching of the contaminants being capped. This system will intercept the upper flow rainfall which infiltrates the shallow surface soils and then flows northward along the rock. Deeper groundwater flow within bedrock is not affected by this system and will continue to flow under, and possibly through, the landfill. There is potential that the water table under the new cap may be somewhat lowered, but this potential is difficult to estimate. Monitoring wells will be left in place, including a pumping well and observation wells, to monitor the levels and quality of groundwater after the cap is completed. It is likely that the interceptor pipe will reduce the volume and velocity of groundwater flow through the site, particularly during the wet seasons, thus reducing potential leaching of contaminants northward into the wetlands. Further reduction in leaching is expected to result from the elimination of rainwater infiltration because the cap seals the landfill.

3.4 Other Design Requirements

3.4.1 Pavement Design

The deployed parking area, currently unpaved, will receive a 2-inch bituminous concrete surface course. Although two inches is the minimum pavement thickness allowable by government standards (Design Manual 5.4), it will be sufficient for use as a parking lot. The crushed stone cap is equivalent to a base course more than one foot thick. The new paved area for sand and salt handling has been provided to prevent these materials from mixing with the cap and to armor the cap against excavation equipment digging into the cap. Government requirements for tracked vehicles is 4 inches; however, the equipment to be used here is expected to be rubber-tired; therefore 3 inches is being provided.

The area to be used for crane testing is subject to the greatest loading. Based on discussions with the government, this area was to be set to match the existing thickness. Borings indicate this area will require approximately 2 inches in thickness, which may be insufficient for cranes. The minimum allowable thickness of 4 inches for tracked vehicles appears to be reasonable. This pavement will be placed in two 2-inch (compacted thickness) lifts.

3.4.2 Utilities

The overhead electric service shall remain to supply the site lighting and Building 496. The poles will be relocated just outside (south of) the new landfill cap and shortened due to elimination of the quonset hut, presently the western terminus.

3.4.3 Proof Rolling

The existing site varies in surface compaction due to original placement, erosion, and site activities. To ensure that a strong and uniform subgrade is provided for cap support, the contractor will be required to proof-roll the site. In areas to be filled, proof rolling will precede placement of fill. Where grades are to be lowered, proof rolling will be required on the

subgrade surface after cutting to the new grades. The contractor will be required to provide this compacted subgrade, at proper moisture content, just prior to placement of the geosynthetic clay liner (GCL) and other cap materials.

3.4.4 Monitoring Wells

All existing monitoring wells on site, approximately 16, plus a pumping well and four observation wells will be maintained for future testing, sampling, and other monitoring of groundwater quantity and quality. This well maintenance will require sealing around each well as it penetrates the new cap. The top of these wells will be modified under this contract to provide tops flush with new finished grade. This modification will prevent damage by collision during future site operations.

3.4.5 Maintenance and Repair

There should be very little maintenance required for this landfill cap, which will be considered permanent. The operation of various equipment to store or move unknown items could result in disturbance or possibly minor settlement of the upper few inches of compacted crushed stone. Maintenance would include regrading and recompacting the upper few inches. The deflection of the geosynthetics will tend to mobilize their tensile strengths, making the regraded area stronger.

If unforeseen events ever damage the cap liner, repair is fairly simple. The damaged area would be uncovered and the damaged geosynthetics removed, if necessary. New geosynthetics would be placed, overlapping existing materials at a minimum of 12 inches. The GCL overlap will be supplemented with granular bentonite for a more positive seal. The crushed stone would be relaid and recompacted. The thickness of this cap is expected to be sufficient to prevent such an occurrence. Specifications provide for the contractor to provide spare materials, for use if a failure occurs, and an operations and maintenance manual for

government use.

3.4.6 Regulatory Standards

Presented below is a discussion of the final design regarding its compliance with regulatory standards. The landfill accepted all waste generated at the Subase which consisted primarily of solid waste; however, it also accepted hazardous waste. The wastes accepted are very similar to those that were accepted by municipal solid waste disposal areas operated during the same time period (1957-1973). For these reasons, the most appropriate standards regarding closure are those governing solid waste disposal areas. Regardless, the design also meets standards applicable to a hazardous waste disposal area. Presented below is a listing of major solid and hazardous waste closure standards followed by a brief discussion regarding criteria used in this design.

Final Cover. CTDEP solid waste regulations specify 2 feet of cover material with vegetative cover. U.S. EPA solid waste requirements specify an 18-inch thick infiltration layer (permeability less than 1×10^{-5} cm/sec) overlain by 6-inch thick vegetated erosion layer. Hazardous waste regulations require that a cover have a permeability less than any bottom liner or natural soils. Hazardous waste guidance (minimum technology standards) specifies a 2-foot thick infiltration layer with a permeability less than 1×10^{-7} cm/sec, overlain by an impervious flexible membrane liner,overlain by a 1-foot thick drainage layer, overlain by a 2-foot thick soil layer capable of supporting vegetation.

The final design to meet the performance criteria in the above standards includes an impervious geocomposite bentonite liner with a permeability less than 1×10^{-7} cm/sec overlain by a drainage fabric. As the design for the landfill had to provide for future use of this area, in lieu of layer of soil to support vegetation the upper cover layer consists of 18 inches of compacted crushed stone with bituminous concrete pavement provided in some areas. The

thicker layers provided in the referenced regulations is necessary to provide a media to support vegetative growth. As the final surface in this design is either compacted crushed stone or bituminous concrete pavement and is not to be vegetated, there is no need to provide 24 inches of cover.

Grading. All of the referenced regulations require grading to promote run-off and prevent run-on, and to accommodate settling. Only CTDEP has numerical criteria to address these concerns. The CTDEP regulations specify a minimum slope of 4 percent and a maximum slope of 3 horizontal to 1 vertical. The design does prevent all run-on including shallow overburden groundwater from impacting the site and promotes the run-off of all precipitation. A settlement analysis was performed (see appendices) and slopes steeper than those proposed (1 to 4 percent) are not necessary to prevent ponding or to promote run-off; therefore, the 3 percent minimum in CTDEP regulations was not utilized. In addition, to minimize the disturbance of landfill materials, the grades were kept as close as possible to existing grades. Steeper slopes would also cause erosion of the compacted crushed stone cover. Connecticut standards also contain a maximum of 3:1 slopes to minimize erosions. The only areas with slopes exceeding this number are at the toe of the slope of the landfill adjacent to the wetland. Slopes in this area are as steep as 2:1. Due to the short run of these slopes, and combination of stone and vegetative cover, erosion will not be a problem on these slopes. In addition, providing the steeper slope minimizes disturbance of landfill materials.

Decomposition Gases. Both state and federal solid waste regulations require decomposition gases to be controlled to limit gases at the property line to the Lower Explosive Limit (LEL) of the landfill gases and in structures to 25 percent of the LEL. The design incorporates gas vents along the southern perimeter of the landfill to prevent gas migration. Gas migration is not a concern at the northern limit due to the wetland. In addition, gas vents

around the one structure with a foundation are provided in the design. As stated in the previous section on subsurface conditions, landfill materials are not expected to generate large volumes of gas.

3.4.7 Structures

The storm drain system requires two minor headwall structures to provide for minor water courses where existing storm drains have been eliminated. Standard Navy or Connecticut Department of Transportation details are not quite adequate to meet the special needs of this project. These structures have been carefully detailed, and some reinforcing steel has been added, mostly for temperature stresses, but they are similar enough to standard details in size and use so that no structural analysis was necessary.

The new salt shed is to be a replacement of the existing structure, basically an open wooden shed. Materials for this structure, with appropriate sizes and dimensions, have been included in the bid documents for contractors' use in estimating costs in their bid. A shop drawing will be required for this item as part of the construction process.

The storm drain pipes, which are also structures, were chosen to provide for standard highway loads (H20). Most pipes will not be subject to any live loads and none are to be installed at significant depths but the strength of the pipes will provide flexibility in any future changes at the site which could require such strength. It is not desirable to excavate and replace these pipes in the future due to the special construction of cap and membrane materials adjacent to the pipe.

4.0 CONCLUSIONS

The placement of the cap on the Area A Landfill is expected to provide a much better working surface than what currently exists. The proof rolling prior to cap placement, the provision of bituminous concrete pavement in areas of high usage, and the elimination of stormwater running onto the surface all combine to ensure that this cap will be stable. The impervious liner (GCL) will protect the landfill from rain-water infiltration while the entire cap seals potentially contaminated landfill materials from contact by personnel, visitors, or animals. While some groundwater flow through this landfill may still occur, the volume and rate have been reduced to the extent possible using a shallow cutoff trench and interceptor pipe. This project provides the desired improvements to this landfill, while maintaining the site for current government uses.

APPENDIX A

SUPPLEMENTAL SOIL SAMPLING RESULTS

NAVAL SUBMARINE BASE – NEW LONDON
AREA A LANDFILL
SUMMARY OF SOIL ANALYTICAL DATA (VOLATILE ORGANICS)

PARAMETER	2LSS20 (0.5–1.0)	2LSS21 (0.5–1.0)	2LSS22 (0.5–1.0)	2LSS23 (0.5–1.0)	2LSS24 (0.5–1.0)	DUPLICATE (0.5–1.0)	EQUIPMENT RINSATE	TRIP BLANK
VOLATILE ORGANIC COMPOUNDS (ppb)								
Chloromethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Bromomethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Vinyl Chloride	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Chloroethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Methylene Chloride	5 BJ	6 BJ	5 BJ	6 BJ	5 BJ	6 BJ	2 BJ	4 BJ
Acetone	9 J	14	12	15	11 U	13	55	10 U
Carbon Disulfide	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,1–Dichloroethene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,1–Dichloroethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,2–Dichloroethene (total)	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Chloroform	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,2–Dichloroethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
2–Butanone	11 U	12 U	11 U	11 U	11 U	12 U	11	10 U
1,1,1–Trichloroethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Carbon Tetrachloride	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Bromodichloromethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,2–Dichloropropane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
cis–1,3–Dichloropropene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Trichloroethene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Dibromochloromethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,1,2–Trichloroethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Benzene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
trans–1,3–Dichloropropene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Bromoform	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
4–Methyl–2–pentanone	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
2–Hexanone	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Tetrachloroethene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
1,1,2,2–Tetrachloroethane	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Toluene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Chlorobenzene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Ethylbenzene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Styrene	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U
Xylene (total)	11 U	12 U	11 U	11 U	11 U	12 U	10 U	10 U

Notes:

B (Organics) – Indicates the anlayte was found in the associated blank as well as the sample.

J – Indicates estimated value.

U – Indicates not detected, value shown is the detection limit.

Shading indicates detected compound.

NAVAL SUBMARINE BASE – NEW LONDON
AREA A LANDFILL
SUMMARY OF SOIL ANALYTICAL DATA (SEMI-VOLATILES)

PARAMETER	2LSS20 (0–1)	2LSS21 (0–1)	2LSS22 (0–1)	2LSS23 (0–1)	2LSS24 (0–1)	DUPLICATE (0–1)	EQUIPMENT RINSATE
SEMI-VOLATILE ORGANIC COMPOUNDS (ppb)							
Phenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
bis(2-Chloroethyl)ether	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2-Chlorophenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
1,3-Dichlorobenzene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
1,4-Dichlorobenzene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
1,2-Dichlorobenzene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2-Methylphenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,2'-oxybis(1-Chloropropane)	380 U	400 U	380 U	380 U	360 U	380 U	11 U
4-Methylphenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
N-Nitroso-di-n-propylamine	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Hexachloroethane	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Nitrobenzene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Isophorone	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2-Nitrophenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,4-Dimethylphenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
bis(2-Chloroethoxy)methane	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,4-Dichlorophenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
1,2,4-Trichlorobenzene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Naphthalene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
4-Chloroaniline	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Hexachlorobutadiene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
4-Chloro-3-methylphenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2-Methylnaphthalene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Hexachlorocyclopentadiene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,4,6-Trichlorophenol	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,4,5-Trichlorophenol	930 U	970 U	920 U	930 U	880 U	930 U	26 U
2-Chloronaphthalene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2-Nitroaniline	930 U	970 U	920 U	930 U	880 U	930 U	26 U
Dimethylphthalate	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Acenaphthylene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,6-Dinitrotoluene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
3-Nitroaniline	930 U	970 U	920 U	930 U	880 U	930 U	26 U
Acenaphthene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,4-Dinitrophenol	930 U	970 U	920 U	930 U	880 U	930 U	26 U
4-Nitrophenol	930 U	970 U	920 U	930 U	880 U	930 U	26 U
Dibenzofuran	380 U	400 U	380 U	380 U	360 U	380 U	11 U
2,4-Dinitrotoluene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Diethylphthalate	380 U	400 U	380 U	380 U	360 U	380 U	11 U
4-Chlorophenyl-phenylether	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Fluorene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
4-Nitroaniline	930 U	970 U	920 U	930 U	880 U	930 U	26 U
4,6-Dinitro-2-methylphenol	930 U	970 U	920 U	930 U	880 U	930 U	26 U
N-Nitrosodiphenylamine (1)	380 U	400 U	380 U	380 U	360 U	380 U	11 U
4-Bromophenyl-phenylether	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Hexachlorobenzene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Pentachlorophenol	930 U	970 U	920 U	930 U	880 U	930 U	26 U
Phenanthrene	380 U	400 U	380 U	150 J	360 U	380 U	11 U
Anthracene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Carbazole	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Di-n-butylphthalate	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Fluoranthene	380 U	400 U	140 J	230 J	360 U	380 U	11 U
Pyrene	380 U	400 U	160 J	270 J	360 U	380 U	11 U
Butylbenzylphthalate	380 U	400 U	380 U	380 U	360 U	380 U	11 U
3,3'-Dichlorobenzidine	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Benzo(a)anthracene	380 U	400 U	380 U	120 J	360 U	380 U	11 U
Chrysene	380 U	400 U	380 U	120 J	360 U	380 U	11 U
bis(2-Ethylhexyl)phthalate	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Di-n-octylphthalate	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Benzo(b)fluoranthene	380 U	400 U	380 U	170 J	360 U	380 U	11 U
Benzo(k)fluoranthene	380 U	400 U	380 U	89 J	360 U	380 U	11 U
Benzo(a)pyrene	380 U	400 U	380 U	140 J	360 U	380 U	11 U
Indeno(1,2,3-cd)pyrene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Dibenz(a,h)anthracene	380 U	400 U	380 U	380 U	360 U	380 U	11 U
Benzo(g,h,i)perylene	380 U	400 U	380 U	380 U	360 U	380 U	11 U

Notes:
J – Indicates estimated value.
U – Indicates not detected, value shown is the detection limit.
Shading indicates detected compound.

NAVAL SUBMARINE BASE – NEW LONDON
AREA A LANDFILL
SUMMARY OF SOIL ANALYTICAL DATA (PESTICIDES/PCBs)

PARAMETER	2LSS20 (0–1)	2LSS21 (0–1)	2LSS22 (0–1)	2LSS23 (0–1)	2LSS24 (0–1)	DUPLICATE (0–1)	EQUIPMENT RINSATE
PESTICIDES/PCBs (ppb)							
alpha–BHC	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
beta–BHC	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
delta–BHC	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
gamma–BHC (Lindane)	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
Heptachlor	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
Aldrin	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
Heptachlor epoxide	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
Endosulfan I	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
Dieldrin	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
4,4'–DDE	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
Endrin	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
Endosulfan II	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
4,4'–DDD	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
Endosulfan sulfate	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
4,4'–DDT	3.9 U	4 U	3.8 U	6.8	3.7 U	3.9 U	0.1 U
Methoxychlor	20 U	21 U	20 U	20 U	19 U	20 U	0.52 U
Endrin ketone	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
Endrin aldehyde	3.9 U	4 U	3.8 U	3.9 U	3.7 U	3.9 U	0.1 U
alpha–Chlordane	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
gamma–Chlordane	2 U	2.1 U	2 U	2 U	1.9 U	2 U	0.052 U
Toxaphene	200 U	210 U	200 U	200 U	190 U	200 U	5.2 U
Aroclor–1016	39 U	40 U	38 U	39 U	37 U	39 U	1 U
Aroclor–1221	78 U	81 U	78 U	79 U	74 U	79 U	2.1 U
Aroclor–1232	39 U	40 U	38 U	39 U	37 U	39 U	1 U
Aroclor–1242	39 U	40 U	38 U	39 U	37 U	39 U	1 U
Aroclor–1248	39 U	40 U	38 U	39 U	37 U	39 U	1 U
Aroclor–1254	39 U	40 U	15 JP	20 JP	37 U	39 U	1 U
Aroclor–1260	39 U	40 U	38 U	39 U	37 U	39 U	1 U

Notes:

J – Indicates estimated value.

P – Indicates there is greater than a 25% concentration difference between quantitation on the primary and confirmatory GC columns. The lower value was reported.

U – Indicates not detected, value shown is the detection limit.

Shading indicates detected compound.

NAVAL SUBMARINE BASE – NEW LONDON
AREA A LANDFILL
SUMMARY OF SOIL ANALYTICAL DATA (INORGANICS)

PARAMETER	2LSS20 (0–1)	2LSS21 (0–1)	2LSS22 (0–1)	2LSS23 (0–1)	2LSS24 (0–1)	DUPLICATE (0–1)	EQUIPMENT RINSATE
INORGANICS (ppm)							
Aluminum	7150	12200	6180	8220	8580	5680	0.0179 U
Antimony	2.6 UN	3.2 UN	2.9 UN	3 UN	2.4 UN	2.3 UN	0.0156 U
Arsenic	0.5 B	2.2	1.2 B	1.9 B	1.1 B	0.59 B	0.0029 U
Barium	54.5	36.6 B	38.8	48.1	42.5	49.1	0.0024 U
Beryllium	0.31 B	0.5 B	0.36 B	0.34 B	0.37 B	0.24 B	0.0004 U
Cadmium	0.35 U	0.43 U	0.39 U	0.41 U	0.33 U	0.31 U	0.0021 U
Calcium	1300	794 B	1500	6090	989	1100	0.0618 B
Chromium	8.2	12.7	9.2	10.7	9.7	6.5	0.0023 U
Cobalt	5.7 B	5.2 B	4.5 B	5.4 B	5.3 B	4.6 B	0.0032 U
Copper	4.3	7.3	15.5	9.4	6.2	3.1 B	0.0026 U
Iron	10100	11700	7660	10100	10200	7960	0.0184 B
Lead	5	7.4	13.6	15.9	5	4.9	0.0007 U
Magnesium	3960 *	2610 *	2320 *	2990 *	3120 *	3070 *	0.022 U
Manganese	247 N	145 N	177 N	179 N	254 N	213 N	0.001 U
Mercury	0.02 U	0.04 U	0.04 U	0.03 B	0.03 U	0.02 U	0.0001 U
Nickel	5.6 B	7.6 B	7.4 B	7.2 B	5.9 B	4.9 B	0.0104 U
Potassium	3110	1270	1470	1830	1930	2370	0.649 U
Selenium	0.28 U	0.35 U	0.32 U	0.33 U	0.26 U	0.25 U	0.0017 U
Silver	0.59 UN	0.74 UN	0.68 UN	0.7 UN	0.56 UN	0.53 UN	0.0036 U
Sodium	37.4 B	34.3 B	42.5 B	113 B	38.6 B	33.6 B	0.0224 U
Thallium	0.46 U	0.57 U	0.53 U	0.55 U	0.44 U	0.41 U	0.0028 U
Vanadium	17.9	24.5	15.6	19.7	18	15	0.0033 U
Zinc	34	24.5	74.4	46.6	28.7	29.6	0.0024 B
Boron	2.9 U	3.6 U	3.3 U	3.4 U	2.7 U	2.6 U	0.0175 U
Cyanide	0.11 U	0.11 U	0.09 B	0.11 U	0.09 U	0.1 U	0.001 U

Notes:

B (Inorganics) – Indicates the result was less than the Contract Required Detection Limit (CRDL), but greater than or equal to the Instrument Detection Limit (IDL).

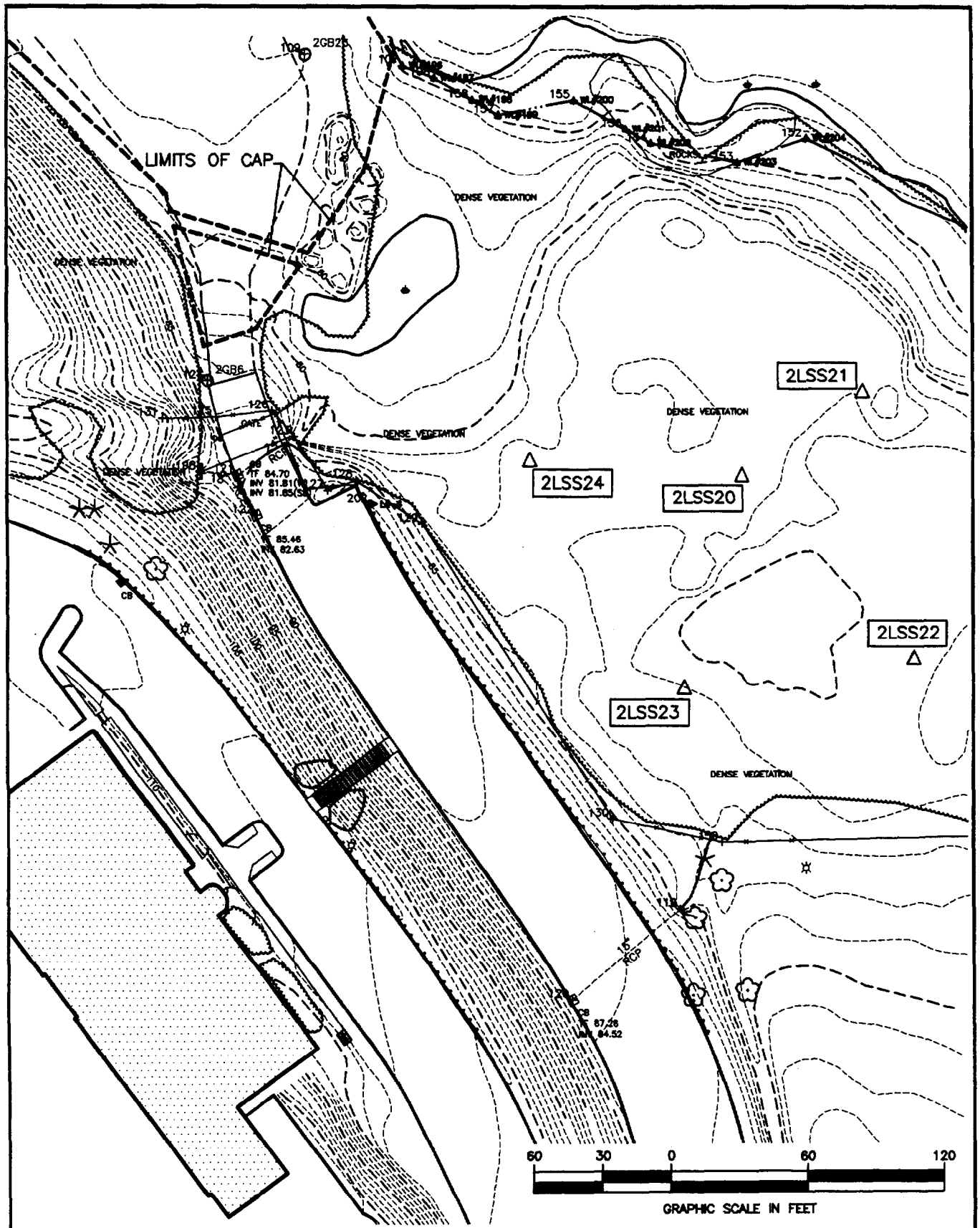
J – Indicates estimated value.

N – Indicates spiked sample recovery not within control limits.

U – Indicates not detected, value shown is the detection limit.

* – Indicates duplicate analysis is not within control limits.

Shading indicates detected compound.



INSTALLATION RESTORATION STUDY
NAVAL SUBMARINE BASE–NEW LONDON
GROTON, CONN.

LEGEND

⊕ 6MW1	MONITORING WELL	—10—	EXISTING CONTOUR
○ 6TB1	TEST BORING	123	BUILDING NUMBER
□ 2DSD1	SEDIMENT SAMPLE		WATERCOURSE
△ 6SS1	SURFACE SOIL SAMPLE	—STM—	STORM SEWER
◇ 2DSW1	SURFACE WATER SAMPLE	□	CATCH BASIN

AREA A LANDFILL
LIMITS OF CAP
SUPPLEMENTAL
SOIL SAMPLE LOCATIONS

ATLANTIC ENVIRONMENTAL SERVICES, INC.

APPENDIX B

CRANE LOADING ON EXISTING SOILS AND CAP DESIGN

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY MG DATE 3-23-94 SUBJECT AREA "A" LANDFILL – CAP DESIGN SHEET NO. 1 OF 28
CHKD. BY G DATE 4-20-94 SUMMARY OF SELECTED BORINGS JOB NO.

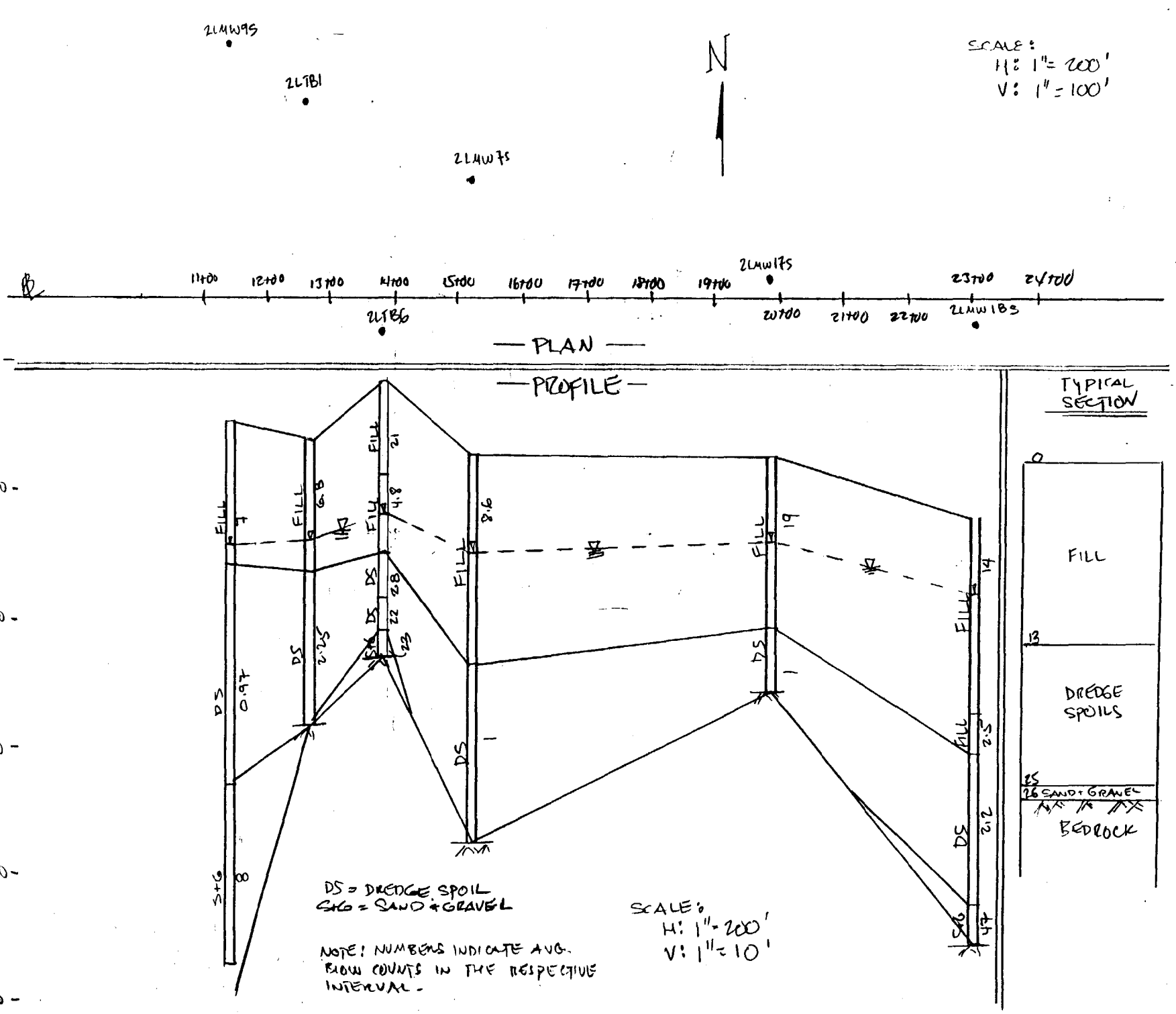
SUMMARY OF
AVERAGE BLOW COUNTS AT
SELECTED BORINGS

BORING NUMBER	INTERVAL	DESCRIPTION	AVG. BLOW COUNT * PER 6" PENETRATION	GROUND ELEV	ELEV OF GW
2LTB 1	0-10	FILL	6.75	84.0	~76
	12-22	DREDGE SPOIL	2.25		
	22	AUGER REFUSAL	—		
2LTB 6	0-7	FILL	21	88.4	~78
	7-13	FILL	4.75		
	13-16.5	DREDGE SPOIL	2.57		
	16.5-19	DREDGE SPOIL	21.8		
	19-21	SAND & GRAVEL	23		
	21	AUGER REFUSAL	—		
2LMW7S	0-16	FILL	8.6	82.8	~75
	16-30	DREDGE SPOIL	1		
	30	BEDROCK	—		
2LMW9S	0-11	FILL	7	85.3	~76
	11-28	DREDGE SPOIL	0.97		
	28-42	SAND & GRAVEL	8		
2LMW17S	0-13	FILL	19	82.48	~76
	13-18	DREDGE SPOILS	1.3		
	18	AUGER REFUSAL	—		
2LMW18S	0-15	FILL	14	77.94	~72
	15-18	FILL	25		
	18-30	DREDGE SPOILS	2.2		
	30-33	SAND & GRAVEL	47		
	33	AUGER REFUSAL	—		

* 140 LB HAMMER, 30 INCH DROP.

AVERAGE CONDITIONS THROUGH LANDFILL

LAYER	DEPTH INTERVAL	BLOW COUNT	INTERVAL RANGE TOP	BOTTOM
FILL	0-13	9.6	0	10-18
DREDGE SPOIL	13-25	1.7	10-18	18-30
SAND & GRAVEL	25-26	23		
BEDROCK	26	—	18	42
GROUNDWATER	8	—	6	10.4



ATLANTIC

ENVIRONMENTAL SERVICES, INC.

COLCHESTER, CT

BY DLG DATE 3-23-94

CHKD. BY G DATE 4-20-94

SUBJECT AREA A LANDFILL CAP DESIGN
PLAN + PROFILE OF SELECTED BORINGS

SHEET NO. 2 OF 28

JOB NO.

FILL - AVG. CONDITIONS

AVERAGE BLOW COUNTS IN FILL LAYER RANGE FROM 2.5 TO 21 PER 6 INCHES OR 5 TO 42 PER FOOT. WITH AN AVERAGE OF ~10 PER 6" OR 20 PER FOOT.

FROM FOUNDATION ENGINEERING HANDBOOK (FEH) FIG 1.21

R_s = SAMPLER HAMMER RATIO = $\frac{D_o^3 - D_i^3}{144\,WH}$ (FT^2/LB)

$= \frac{3^3 - 2^3}{144(140)(30)} = 3.1 * 10^{-5}$

B = BLOWS PER FOOT ⇒ 5-42 (RANGE), 20 (AVG)

∴ LOOSE TO MEDIUM DENSE WITH AVG $D_R \approx 35\%$ (RELATIVE DENSITY)

FROM TABLE 2.43 w/ N = 20/FT, LOOSE TO MEDIUM, MAINTAINING CONSISTENCY BETWEEN TABLE 2.43 + FIG 1.21 WITH $D_R = 0.35$ ∴ $\phi = 30°$.

FROM TABLE 7-4, ESTIMATE ALLOWABLE BEARING VALUE ON SOILS = 3 TON/SF
= 42 psi

FOR CONDITIONS AT DREDGE SPOILS LAYER, REFER TO APPENDIX ENTITLED "SETTLEMENT ANALYSIS"

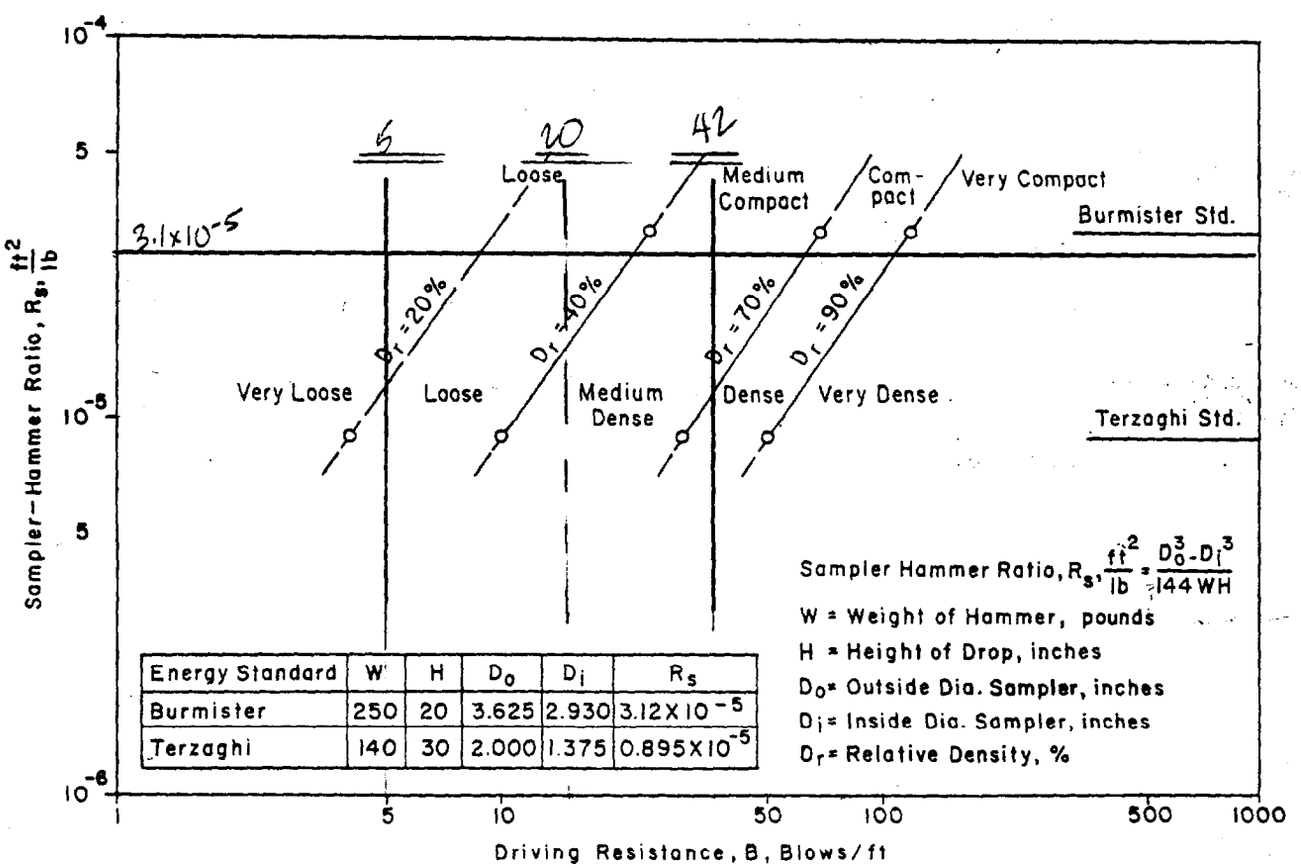
SHEET 4 OF 28

driving but upon withdrawal of the sampler from the hole the flap closes and retains the sample. An alternative type of retainer is the basket retainer which consists of a series of fingers of spring steel mounted vertically on the periphery of a metal ring. The tips of the fingers deflect toward the center of the sampler to form a basker shape. As soil enters the sampler it easily pushes the steel fingers back against the walls of the sampler. If upon withdrawal of the sampler the sample tends to slide out, the points of the spring move in and grip the sample. Other types of retainers for use under various conditions also are available. One, referred to as the L.A.D. retainer, consists of spring steel fingers and a plastic sleeve. The latter drops over the fingers if the sample tends to fall out and effectively closes the opening at the base of the sampler.

In general, the split spoon is available with inside diameters ranging from 1.5 to 4.5 inches in 0.5-inch increments. Barrels are available in standard lengths of 18 and 24 inches and have a wall thickness of 0.25 inch. The 1.5-inch-diameter sampler is popular since correlations have been developed between the number of blows required for penetration of this sampler and the relative density of cohesionless soils and the shear strength of cohesive soils. The larger diameter samplers are used when gravel particles are present. An advantage of the larger size barrels is that they provide more material for classification tests.

Sampling with the split barrel sampler is accomplished by driving the sampler into the ground with a drive hammer, whose weight will depend on the size of the sampler. Generally the 1.5-inch sampler is driven with a 140-lb hammer dropped 30 inches; the larger samplers are driven with a 300-lb hammer dropped 18 inches. It is common practice to record the number of blows for each 6 inches of the total sampler penetration. The number of blows required to drive the 1.5-inch sampler a distance of 12 inches using the stipulated hammer weight and drop is the Standard Penetration Resistance (SPT values) developed by Terzaghi and Peck. The graphs of Figs. 1.21 and 1.22 permit the blows of different energy used on the larger samples to be converted into SPT values. If a sampler is driven 18 inches the blows for the last 12 inches of penetration are used as the SPT value in order to eliminate the effect of any disturbance or sediment at the bottom of the hole. Similarly, when a drive of 24 inches is made, the blows for the middle 12 inches are used. In order to assure the uniformity and accuracy of SPT values it is essential that the hammer have a free fall. This can be accomplished by using a rope instead of a cable to hoist the hammer and by not restraining the rope during the drop. Many correlations have been made between SPT values and soils properties. These correlations and their validity are discussed in detail by de Mello (1971) and Fletcher (1965).

Split barrel sampling is used primarily in those instances where it is necessary to determine the stratification, identification, consistency, and density of the soils present at a site. Practically all subsurface explorations are initiated with this type of sampling and frequently design can be made on the basis of the split barrel sample data. This is particularly the case where a minor type structure is involved and where foundation conditions are good. In other instances the split barrel borings are supplemented with sampling by one or more of the undisturbed type samplers. The primary advantages of split barrel sampling are that it is simple, quick, and economical.



Energy Standard	W	H	D_o	D_i	R_s
Burmister	250	20	3.625	2.930	3.12×10^{-5}
Terzaghi	140	30	2.000	1.375	0.895×10^{-5}

Fig. 1.21 Sampler driving resistance vs compactness-cohesionless sands and silts.

TABLE 2.43 RELATIONSHIP BETWEEN RELATIVE DENSITY, PENETRATION RESISTANCE, AND ANGLE OF INTERNAL FRICTION OF COHESIONLESS SOILS.

Type of Soil	Penetration Resistance N	Relative Density D_r	Angle of Internal Friction ϕ	
			Peck (1974)	Meyerhof (1956)
Very loose sand	<4	<0.2	<29	<30
Loose sand	4–10	0.2–0.4	29–30	30–35
Medium sand	10–30	0.4–0.6	30–36	35–40
Dense sand	30–50	0.6–0.8	36–41	40–45
Very dense sand	>50	>0.8	>41	>45

Table 7-4. Allowable Bearing Values on Soils, Tons per Sq Ft*

Massive crystalline bedrock: granite, gneiss, traprock—in sound condition	100
Foliated rock: schist and slate—in sound condition	40
Sedimentary rock: hard shales, siltstones, sandstones—in sound condition	15
Exceptionally compacted gravels or sands	10
Compact gravel or sand-gravel mixtures	6
Loose gravel; compact coarse sand	4
Loose coarse sand or sand-gravel mixtures; compact fine sand, or wet, confined coarse sand	3 ←
Loose fine sand or wet, confined fine sand	2
Stiff clay	4
Medium stiff clay	2
Soft clay	1

*Code Manual, New York State Building Construction Code. See also Table 7-3.

FROM "BASIC SOILS ENGINEERING", B.K. HOUGH

FILL - AT CRANE PAD (BASED ON BORING 2LTB1)

BLOW COUNTS RANGE FROM 4/FT TO 30/FT W/ AN AVG OF 12.2/FT.

FROM FEH FIG 1.21

$k_s = 3.1 * 10^{-5}$ B = 4-30 (RANGE), 12 (AVG)

∴ LOOSE TO MEDIUM DENSE WITH AVG $D_R \approx 25\%$

FROM TABLE 2.43 W/ N = 12/FT, LOOSE TO MEDIUM, MAINTAINING CONSISTENCY BETWEEN TABLE 2.43 & FIG 1.21

CONSERVATIVE ESTIMATE ⇒ $\phi = 30°$

FROM TABLE 7-4, ESTIMATE ALLOWABLE BEARING VALUE ON SOILS = 3 TONS/SF

= 42 PSF

CONCLUSION:

AVERAGE CONDITIONS & CONSERVATIVE CONDITIONS (@ CRANE PAD) YIELD SIMILAR BEARING CAPACITIES

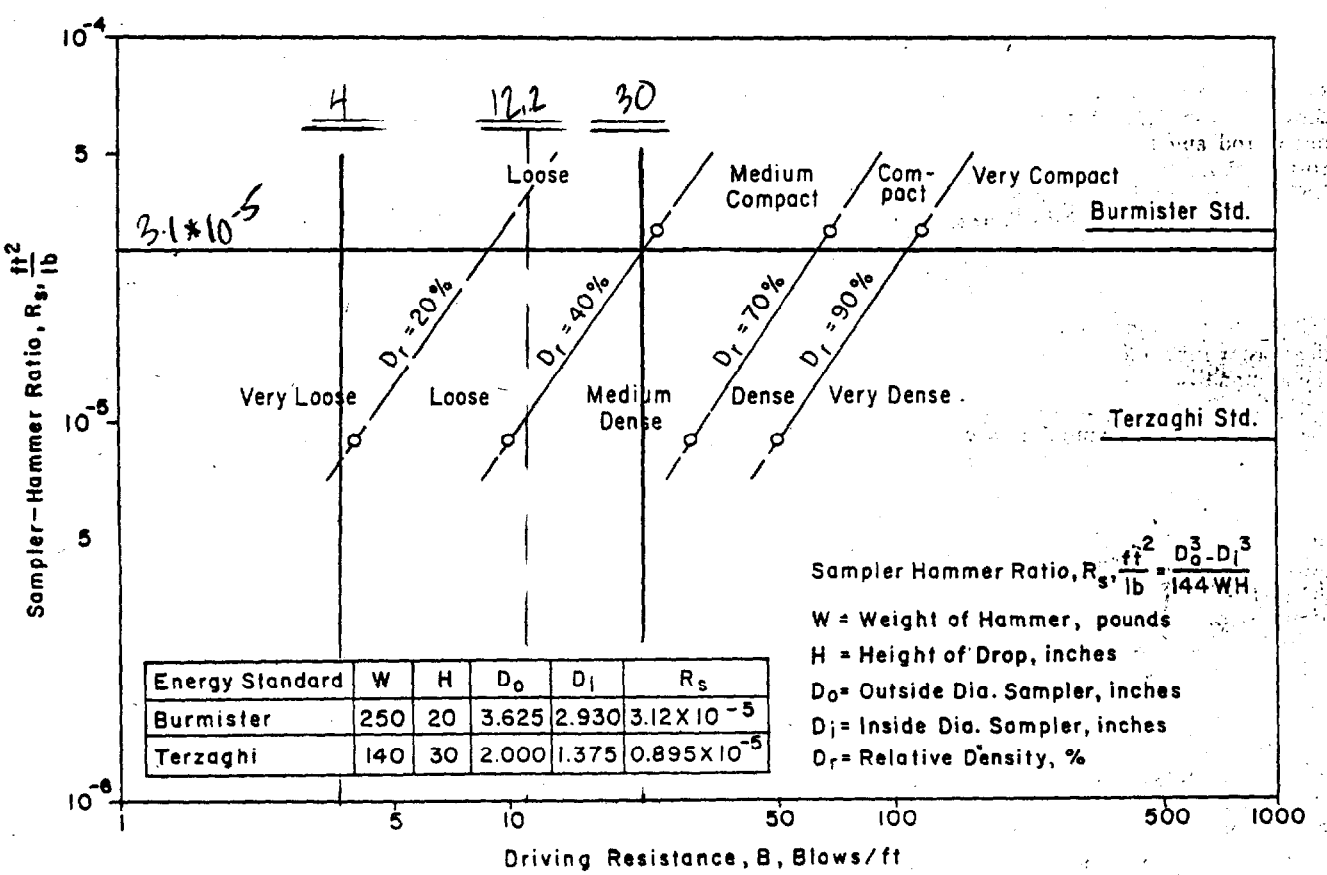
SHEET 7 OF 28

driving but upon withdrawal of the sampler from the hole the flap closes and retains the sample. An alternative type of retainer is the basket retainer which consists of a series of fingers of spring steel mounted vertically on the periphery of a metal ring. The tips of the fingers deflect toward the center of the sampler to form a basker shape. As soil enters the sampler it easily pushes the steel fingers back against the walls of the sampler. If upon withdrawal of the sampler the sample tends to slide out, the points of the spring move in and grip the sample. Other types of retainers for use under various conditions also are available. One, referred to as the L.A.D. retainer, consists of spring steel fingers and a plastic sleeve. The latter drops over the fingers if the sample tends to fall out and effectively closes the opening at the base of the sampler.

In general, the split spoon is available with inside diameters ranging from 1.5 to 4.5 inches in 0.5-inch increments. Barrels are available in standard lengths of 18 and 24 inches and have a wall thickness of 0.25 inch. The 1.5-inch-diameter sampler is popular since correlations have been developed between the number of blows required for penetration of this sampler and the relative density of cohesionless soils and the shear strength of cohesive soils. The larger diameter samplers are used when gravel particles are present. An advantage of the larger size barrels is that they provide more material for classification tests.

Sampling with the split barrel sampler is accomplished by driving the sampler into the ground with a drive hammer, whose weight will depend on the size of the sampler. Generally the 1.5-inch sampler is driven with a 140-lb hammer dropped 30 inches; the larger samplers are driven with a 300-lb hammer dropped 18 inches. It is common practice to record the number of blows for each 6 inches of the total sampler penetration. The number of blows required to drive the 1.5-inch sampler a distance of 12 inches using the stipulated hammer weight and drop is the Standard Penetration Resistance (SPT values) developed by Terzaghi and Peck. The graphs of Figs. 1.21 and 1.22 permit the blows of different energy used on the larger samples to be converted into SPT values. If a sampler is driven 18 inches the blows for the last 12 inches of penetration are used as the SPT value in order to eliminate the effect of any disturbance or sediment at the bottom of the hole. Similarly, when a drive of 24 inches is made, the blows for the middle 12 inches are used. In order to assure the uniformity and accuracy of SPT values it is essential that the hammer have a free fall. This can be accomplished by using a rope instead of a cable to hoist the hammer and by not restraining the rope during the drop. Many correlations have been made between SPT values and soils properties. These correlations and their validity are discussed in detail by de Mello (1971) and Fletcher (1965).

Split barrel sampling is used primarily in those instances where it is necessary to determine the stratification, identification, consistency, and density of the soils present at a site. Practically all subsurface explorations are initiated with this type of sampling and frequently design can be made on the basis of the split barrel sample data. This is particularly the case where a minor type structure is involved and where foundation conditions are good. In other instances the split barrel borings are supplemented with sampling by one or more of the undisturbed type samplers. The primary advantages of split barrel sampling are that it is simple, quick, and economical.



Energy Standard	W	H	D_o	D_i	R_s
Burmister	250	20	3.625	2.930	3.12×10^{-5}
Terzaghi	140	30	2.000	1.375	0.895×10^{-5}

Fig. 1.21 Sampler driving resistance vs compactness-cohesionless sands and silts.

CRANE PAD

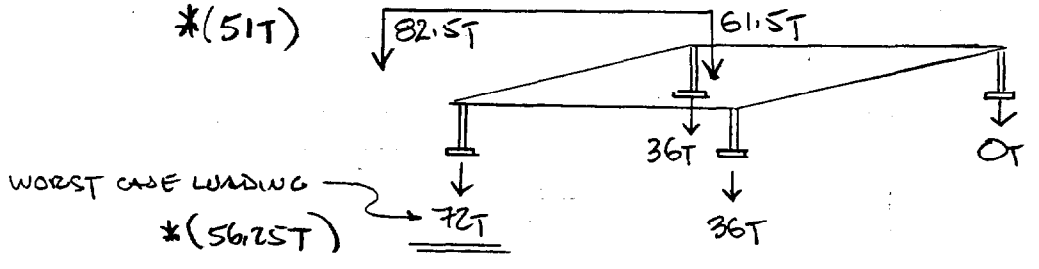
WORST CASE CONDITION (PER CRANE & RIGGING DEPT @ U.S. SUBMARINE BASE, NLON

75 TON CRANE, TESTING @ 110% CAPACITY

VEHICLE WEIGHT:	123,000 LBS	=	61.5 TONS
LOAD CAPACITY:	150,000 LBS	=	75 TONS
MAX TEST LOAD:	165,000 LBS	=	82.5 TONS
OUTRIGGER PADS:	4 @ 31" ϕ	=	4 @ 755 IN^2

TOTAL MAXIMUM WEIGHT = 61.5 + 82.5 = 144 TONS

ASSUME NON-UNIFORM DISTRIBUTION ON 3 OF 4 OUTRIGGERS



STRESS @ OPERATING SURFACE = (72 TONS * 2000 LBS/TON) / 755 IN^2 = 191 psi

OPERATING SURFACE

MAXIMUM OPERATING CONDITION

75 TON CRANE @ OPERATING CAPACITY OF 51 TONS *

VEHICLE WT:	123,000 LBS	=	61.5 TONS
OPERATING CAPACITY:	102,000 LBS	=	51.0 TONS
OUTRIGGER PADS:	4 @ 31" ϕ	=	4 @ 755 IN^2

TOTAL MAX WT = 61.5 + 51 = 112.5 TONS

ASSUME SIMILAR DISTRIBUTION AS ABOVE

∴ WORST CASE LOADING = 56.25 TONS* ON (1) OUTRIGGER

ie) (56.25 TONS)(2000 LBS/TON) / 755 IN^2 = 149 psi

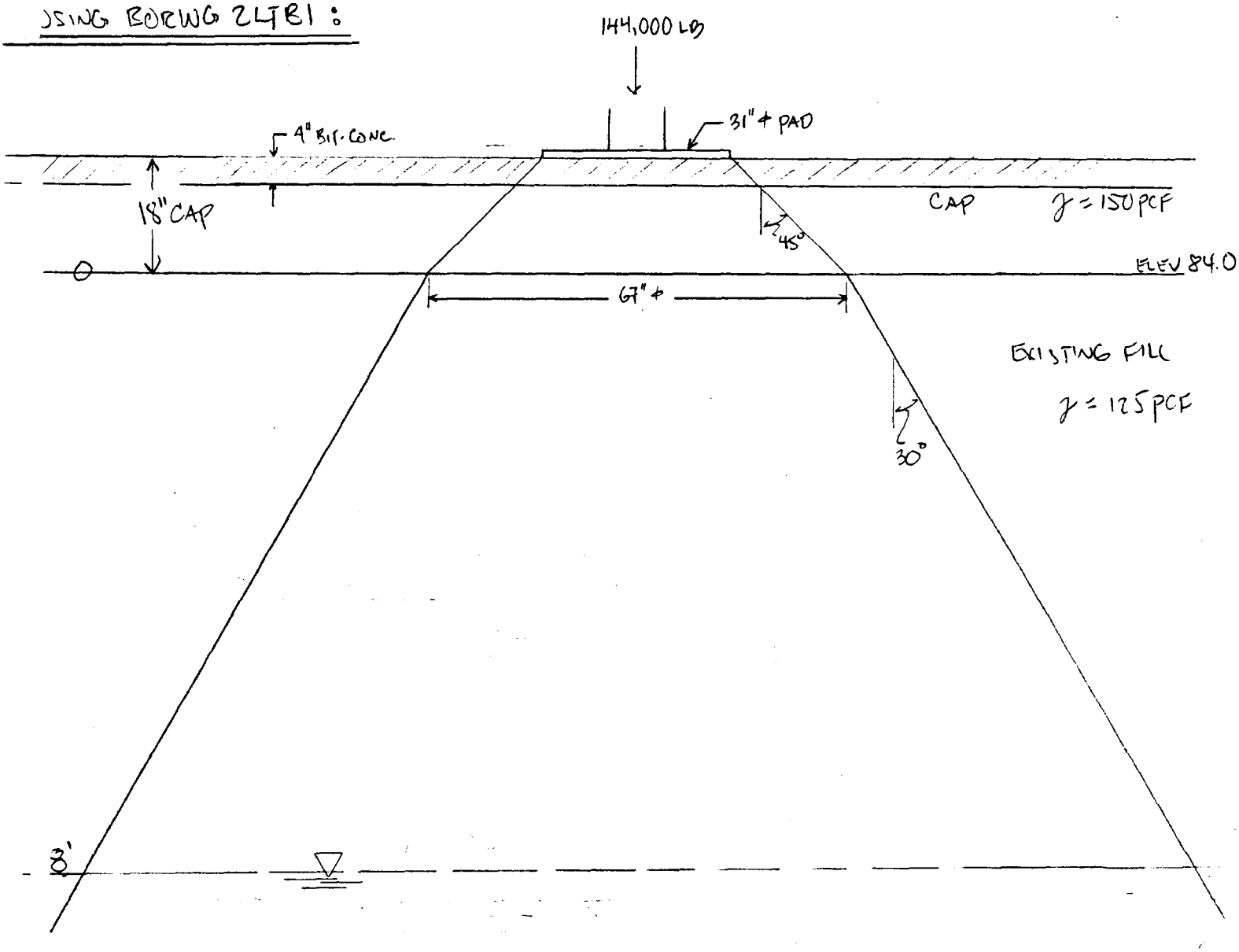
ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY [illegible] DATE 3-23-94 SUBJECT AREA A LANDFILL – CAP DESIGN SHEET NO. 9 OF 28

CHKD. BY [illegible] DATE 4-20-94 DESIGN CONDITIONS ANALYSIS JOB NO.

CRANE PAD

USING BORING 2LTB1:



FILL

11'

$W_{11} = (\tan 30 * 11)(2) + (67''/12) = 18.29$ FT

DREDGE SPOILS

CALCULATE STRESS AT $d=0$
BEFORE LOADING

$\sigma_i = 1.5' * 150\ PCF = 225\ PSF = 1.56\ psi < 42\ psi \therefore OK$

AFTER LOADING

$\Delta\sigma = 144000\ LBS / 67''\phi = 144000\ LBS / 3526\ in^2 = 40.8\ psi$

$\sigma_{FINAL} = \sigma_i + \Delta\sigma = 1.56 + 40.8 = 42.4\ psi \approx 42\ psi \therefore OK$

FACTORS OF SAFETY WILL BE REALIZED THROUGH THE USE OF ADDITIONAL LOAD SPREADING PADS AND COMPACTION OF THE SUBGRADE.

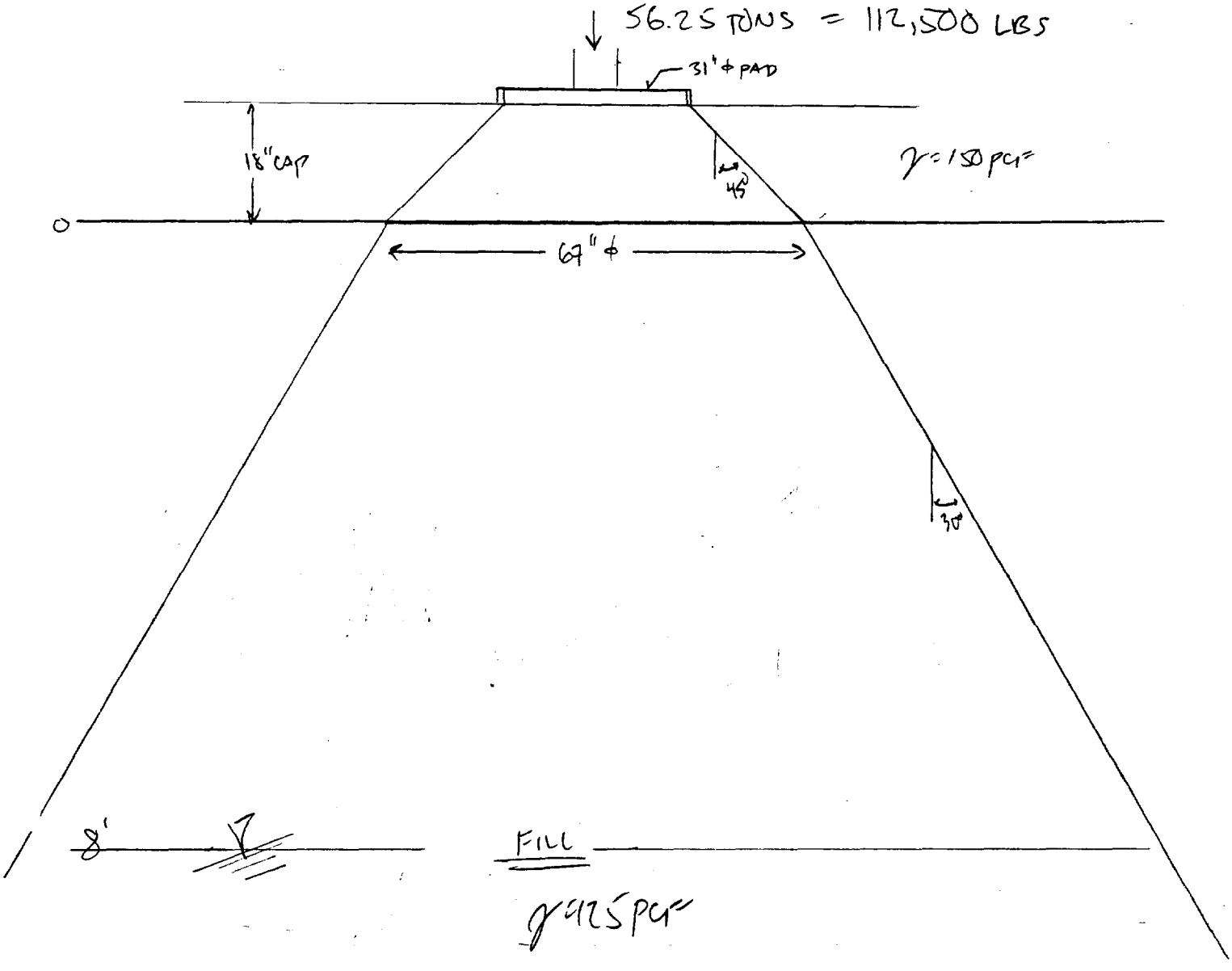
NOTE: SUBASE CRANE & RIGGING DEPT STAFF REPORT MINIMAL SETTLEMENT (1-2") BENEATH OUTRIGGERS.

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY MG DATE 4-11-94
CHKD. BY G DATE 4-20-94

SUBJECT AREA 'A' LANDFILL - CAP DESIGN
DESIGN CONDITIONS ANALYSIS
OPERATING SURFACE

SHEET NO. 11 OF 28
JOB NO.



11

$W_{11} = (\tan 30 * 11)(2) + (67''/12) = 18.3$ FT

DREDGE SPOILS

CALCULATE STRESS @ d=0

BEFORE LOADING

$\sigma_i = 1.5' * 150\ pcf = 225\ pcf = 1.56\ psi < 42\ psi \therefore$ O.K.

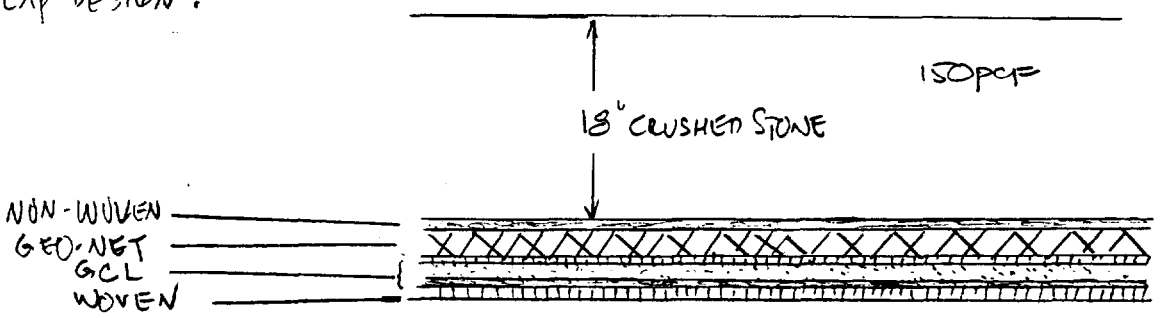
AFTER LOADING

$\Delta\sigma = 112{,}500\ LBS / 67'\phi = 112500 / 3526\ in^2 = 32\ psi$

$\sigma_{FINAL} = \sigma_i + \Delta\sigma = 1.56 + 32 = 33.56 < 42\ psi \therefore$ O.K

NOTE: ADDITIONAL FACTOR OF SAFETY WILL BE REALIZED THROUGH USE OF SUPPORT PADS UNDER OUTRIGGERS AND COMPACTION OF LANDFILL PRIOR TO CAP INSTALLATION.

PROPOSED CAP DESIGN:



GEONET IS PROVIDED TO ALLOW INFILTRATING RAINWATER TO DRAIN.

CALCULATE FLOW CAPACITY OF GEONET:

FROM MANUFACTURERS TRANSMISSIVITY CHART WITH THE FOLLOWING INPUT VALUES:

CROSS SECTION: SOIL / GEOTEXTILE / GEONET / GEOTEXTILE / SOIL

PRESSURE & ASSUME STATIC LOADS ONLY → 150 PCF * 18" = 225 PSF

∴ USE MINIMUM PRESSURE USED ON CHARTS (2000 PSF).

→ HYDRAULIC TRANSMISSIVITY = 1×10^{-3} m^2/SEC (646 FT^2/MIN / M^2/SEC)

= 0.646 FT^2/MIN * 1 FT WIDTH = 0.646 CF/MIN - FT WIDTH

USING DARCY'S (ASSUME SATURATED SYSTEM UNDER LAMINAR FLOW) (KOERNER p. 353-355)

$$q = kiA = ki(t \times w) = (kt)(i \times w) \Rightarrow kt = q/iw$$

$$kt = \theta \Rightarrow \theta = q/iw \Rightarrow q = \theta iw$$

WHERE

q = VOLUMETRIC FLOW RATE

k = COEFFICIENT OF PERMEABILITY

i = HYDRAULIC GRADIENT

A = FLOW CROSS-SECTIONAL AREA

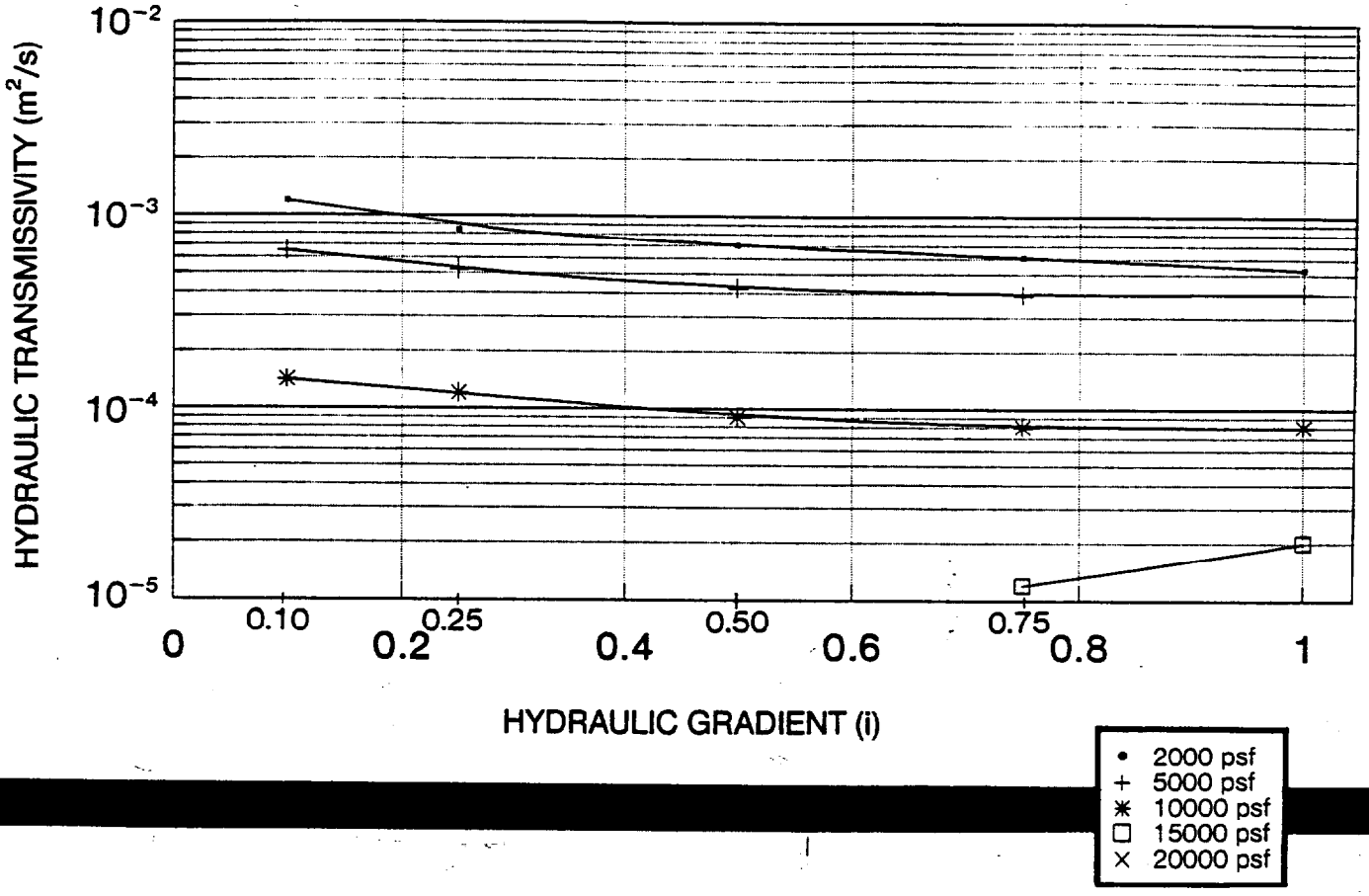
θ = TRANSMISSIVITY

w = WIDTH

t = THICKNESS

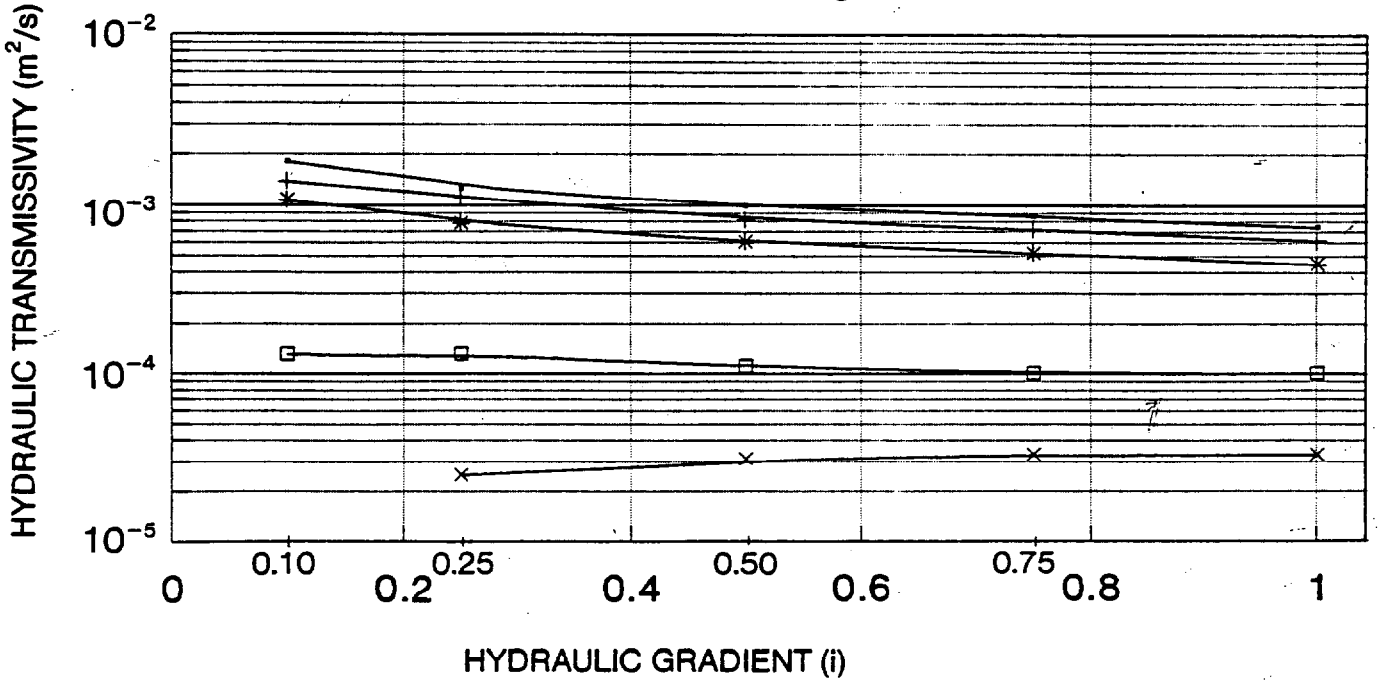
POLY-NET PN3000CN

soil/geotextile/PN3000CN/geotextile/soil



POLY-NET PN4000

soil/geotextile/PN4000/geotextile/soil



SOLVE FOR q:

$q = \theta i w$

$\theta = 0.646$ CF/MIN-FT WIDTH (FROM PREV. CALC)

$i = 0.03$ (3% SLOPE ACROSS CAP)

$w = 1$ FT (UNIT WIDTH)

$$q = 0.646 \frac{CF}{MIN\text{-}FT} (0.03)(1FT) = \underline{\underline{0.01938 \frac{CF}{MIN - FT\ WIDTH}}} = q_{ULT}$$

CALCULATE q_{ALLOW} BASED ON FACTORS OF SAFETY:

FROM KOERNER P. 557-559, $q_{ALLOW} = q_{ULT}\left(\frac{1}{FS_{IN} * FS_{CR} * FS_{CC} * FS_{BC}}\right)$

WHERE:

FS_{IN} = ELASTIC DEFORMATION

FS_{CR} = CREEP

FS_{CC} = CHEMICAL CLOGGING

FS_{BC} = BIOLOGICAL CLOGGING

FROM TABLE 6.10 (KOERNER P558), FACTORS OF SAFETY VALUES FOR SURFACE WATER DRAINS FOR LANDFILL CAPS.

$FS_{IN} = 1.4$, $FS_{CR} = 1.3$, $FS_{CC} = 1.1$, $FS_{BC} = 1.3$

$$q_{ALLOW} = 0.01938 / (1.4 * 1.3 * 1.1 * 1.3) = \underline{\underline{0.00745 \frac{CF}{MIN\text{-}FT}}}$$

CALCULATE VOLUME OF WATER TO DRAIN AT SATURATED CONDITIONS:

ASSUME CRUSHED STONE HAS 10% VOID SPACE

MAXIMUM LENGTH OF CAP SUBJECT TO RAINWATER INFILTRATION: 160 FT

∴ VOLUME = 160 FT * 1 FT WIDTH * 1.5 FT * 0.10 = 24 CF WATER

CALCULATE TIME REQUIRED TO DRAIN SATURATED WATER IN CAP.

24 CF WATER / 0.00745 CF/MW-FT = 3223 MIN = 54 HOURS

ESTIMATE TIME FOR SATURATION TO OCCUR

1.5'/2'/DAY = 0.75 DAYS = 18 HOURS

CONCLUSION:

THIS ANALYSIS REPRESENTS A WORST CASE SITUATION IN WHICH THE CAP BECOMES COMPLETELY SATURATED. THE CRUSHED STONE CAP SHALL BE VERY DENSE AND TIGHTLY COMPACTED, AND GRADED TO DRAIN MOST RAINWATER VIA SHEET FLOW. A SATURATED CONDITION IS NOT LIKELY TO OCCUR, EXCEPT AFTER LENGTHY & HEAVY RAINFALL. THE GEONET PROVIDES POSITIVE DRAINAGE OF THE CAP, TO ALLOW FOR BETTER WORKING CONDITIONS AND TO PREVENT POTENTIAL INFILTRATION THROUGH THE GCL RESULTING FROM HEAD BUILDUP.

THE NON-WOVEN GEOTEXTILE SHALL BE DESIGNED FOR SEPARATION AND SOIL RETENTION PER THE DESIGN CONDITIONS.

CRANE PAD

Problem Parameters/Assumptions:

Pressure @ surface of geosynthetics =	42 psi	Calculated Previously
Assumed gravel size	2.0 inch	Conservative Estimate

GEOTEXTILE DESIGN

Selected geotextile: Trevira 1120

	Trevira 1120 (minimum properties)
Mullen Burst Strength	275 psi
Ultimate Fabric Strength (Grab Tensile)	160 lbs
Puncture Resistance	80 lbs
Tear Resistance	60 lbs
Permittivity	1.74 sec−1
Water Flow Rate	130 gpm/sf
AOS (Apparent Opening Size)	0.21 mm
AOS (Apparent Opening Size)	0.00827 inch
Unit Weight [320lbs/(15'x360')]	6 oz/sy

DESIGNING FOR SEPARATION

Calculate Burst Resistance

T(reqd) = p'*dv — Koerner p.127

T(reqd) = required fabric strength		
p' = stress at the fabrics surface	42.0 psi	Calculated previously
da = average stone diameter	2.0 inch	Assumed material size
dv = maximum void diameter (~ =0.4da)	0.8 inch	Calculated
T(reqd) = p'*dv	33.6 psi	Calculated

Mullen Burst Strength (ASTM D3786)

P(test) = (FS*da*p')/3.6 — Koerner p.128
FS = Factor of safety = (P(test)*3.6)/(da*p')

P(test) = Mullen Burst Strength =	275 psi	Material Specification
FS = Factor of safety = (P(test)*3.6)/(da*p') =	11.8	Calculated

Calculate Tensile Strength Requirement

T = p'*e^2		Koerner p.130
T = the mobilized tensile force		
p' = applied pressure =	42 psi	Calculated previously
e = strain of geotextile between contact points =	0.375 (ratio)	Assumes 50% slippage*
T = p'*e^2	5.9 lbs	Calculated

Calculate the factor of safety based on ultimate fabric strength

T(ult) = Ultimate fabric strength of selected fabric =	160 lbs	Material Specification
FS = T(ult)/T =	27.1	Calculated

Calculate the factor of safety based on allowable fabric strength

FS = T(allow)/T

T(allow) = (T(ult))/(FSid*FScr*FScd*FSbd)		Koerner p. 115
FSid = factor of safety for installation damage	2	Koerner p. 115
FScr = factor of safety for creep	1.2	Koerner p. 115
FScd = factor of safety for chemical degradation	1.5	Koerner p. 115
FSbd = factor of safety for biological degration	1.2	Koerner p. 115
T(allow) = (T(ult))/(FSid*FScr*FScd*FSbd) =	37.04 lbs	Calculated
FS = T(allow)/T	6.3	Calculated

* Maximum strain with no slippage = 0.75, (Koerner p.130)

Calculate Puncture (Tear) Resistance

F(vert) = ((@pi*dh)*(hh)*p')*S' — Koerner p.131

F(vert) = total vertical force imposed on fibers adjacent to the puncture.

dh = average diameter of the hole (= da)	2.0 inch	Assumed material size
hh = propagation height (~ =da)	2.0 inch	Assumed = da
p' = pressure exerted on the geotextile	42 psi	Calculated previously
S = sphericity	0.4	Crushed rock (Koerner p.132)
S' = shape factor (= 1−S)	0.60	Calculated

F(vert)/T(reqd) = (da/di) — Koerner p.132

T(reqd) = required tensile force in fibers

di = initial average void dia. of geotextile = AOS = **0.00827 in** — Material Specification

T(reqd) = (@pi*di*da)*p'*S' — 1.31 lbs. — Calculated

Calculate the factor of safety based on allowable fabric strength

T(allow) = 37.04 lbs — Previously Calculated

Factor of Safety = T(allow)/T(req'd) = 28.29 — Calculated

7

CHECK THE SOIL RETENTION — Koerner p.217

d50 = soil particle size corresponding to 50% finer	13 mm	Estimated*
d60 = soil particle size corresponding to 60% finer	20.00 mm	Estimated*
d10 = soil particle size corresponding to 10% finer	0.20 mm	Estimated*

* Estimated from Processed Aggregate Base and Pavement Surface Treatment, Section M.05.0–1, p. 549, modified so that 5%+/– passes the #200 sieve.

CU = coeficient of uniformity = d60/d10	100.00	Calculated
O95<18*d50/CU		Koerner (Table 2.14 p.122)
O95 = apparent opening size of geotextile (~ =AOS sieve value in mm)		
AOS(req'd) = O95 < 18*d50/CU =	2.34 mm	Calculated
AOS(selected) =	**0.21 mm**	Material Specification
FS = AOS(req'd)/AOS(selected) =	11	Calculated

CONCLUSION: It has been shown that this geotextile can be used, with a reasonable factor of safety, for separation and soil retention functions.

THE NON-WOVEN GEOTEXTILE SHALL BE DESIGNED FOR SEPARATION AND SOIL RETENTION FUNCTIONS.

OPERATING SURFACE

Problem Parameters/Assumptions:

Pressure @ surface of geosynthetics =	34 psi	Calculated Previously
Assumed gravel size	2.0 inch	Conservative Estimate

GEOTEXTILE DESIGN

Selected geotextile: Trevira 1120

	Trevira 1120 (minimum properties)
Mullen Burst Strength	275 psi
Ultimate Fabric Strength (Grab Tensile)	160 lbs
Puncture Resistance	80 lbs
Tear Resistance	60 lbs
Permittivity	1.74 sec−1
Water Flow Rate	130 gpm/sf
AOS (Apparent Opening Size)	0.21 mm
AOS (Apparent Opening Size)	0.00827 inch
Unit Weight [320lbs/(15'x360')]	6 oz/sy

DESIGNING FOR SEPARATION

Calculate Burst Resistance

T(reqd) = p'*dv — Koerner p.127

T(reqd) = required fabric strength

p' = stress at the fabrics surface	34.0 psi	Calculated previously
da = average stone diameter	2.0 inch	Assumed material size
dv = maximum void diameter (~ =0.4da)	0.8 inch	Calculated
T(reqd) = p'*dv	27.2 psi	Calculated

Mullen Burst Strength (ASTM D3786)

P(test) = (FS*da*p')/3.6 — Koerner p.128

FS = Factor of safety = (P(test)*3.6)/(da*p')

P(test) = Mullen Burst Strength =	275 psi	Material Specification
FS = Factor of safety = (P(test)*3.6)/(da*p') =	14.6	Calculated

Calculate Tensile Strength Requirement

T = p'*e ^ 2		Koerner p.130
T = the mobilized tensile force		
p' = applied pressure =	34 psi	Calculated previously
e = strain of geotextile between contact points =	0.375 (ratio)	Assumes 50% slippage*
T = p'*e ^ 2	4.8 lbs	Calculated

Calculate the factor of safety based on ultimate fabric strength

T(ult) = Ultimate fabric strength of selected fabric =	160 lbs	Material Specification
FS = T(ult)/T =	33.5	Calculated

Calculate the factor of safety based on allowable fabric strength

FS = T(allow)/T

T(allow) = (T(ult))/(FSid*FScr*FScd*FSbd)		Koerner p. 115
FSid = factor of safety for installation damage	2	Koerner p. 115
FScr = factor of safety for creep	1.2	Koerner p. 115
FScd = factor of safety for chemical degradation	1.5	Koerner p. 115
FSbd = factor of safety for biological degration	1.2	Koerner p. 115
T(allow) = (T(ult))/(FSid*FScr*FScd*FSbd) =	37.04 lbs	Calculated
FS = T(allow)/T	7.7	Calculated

* Maximum strain with no slippage = 0.75, (Koerner p.130)

Calculate Puncture (Tear) Resistance

F(vert) = ((@pi*dh)*(hh)*p')*S' — Koerner p.131

F(vert) = total vertical force imposed on fibers adjacent to the puncture.

dh = average diameter of the hole (= da)	2.0 inch	Assumed material size
hh = propagation height (~ =da)	2.0 inch	Assumed = da
p' = pressure exerted on the geotextile	34 psi	Calculated previously
S = sphericity	0.4	Crushed rock (Koerner p.132)
S' = shape factor (= 1−S)	0.60	Calculated

F(vert)/T(reqd) = (da/di) — Koerner p.132

T(reqd) = required tensile force in fibers

di = initial average void dia. of geotextile = AOS =	0.00827 in	Material Specification
T(reqd) = (@pi*di*da)*p'*S'	1.06 lbs.	Calculated

Calculate the factor of safety based on allowable fabric strength

T(allow) =	37.04 lbs	Previously Calculated
Factor of Safety = T(allow)/T(req'd) =	34.95	Calculated

CHECK THE SOIL RETENTION — Koerner p.217

d50 = soil particle size corresponding to 50% finer	13 mm	Estimated*
d60 = soil particle size corresponding to 60% finer	20.00 mm	Estimated*
d10 = soil particle size corresponding to 10% finer	0.20 mm	Estimated*

* Estimated from Processed Aggregate Base and Pavement Surface Treatment, Section M.05.0–1, p. 549, modified so that 5%+/– passes the #200 sieve.

CU = coeficient of uniformity = d60/d10	100.00	Calculated
O95<18*d50/CU		Koerner (Table 2.14 p.122)

O95 = apparent opening size of geotextile (~ =AOS sieve value in mm)

AOS(req'd) = O95 < 18*d50/CU =	2.34 mm	Calculated
AOS(selected) =	0.21 mm	Material Specification
FS = AOS(req'd)/AOS(selected) =	11	Calculated

CONCLUSION: It has been shown that this geotextile can be used, with a reasonable factor of safety, for separation and soil retention functions.

CONCLUSION:

THE SELECTION OF A 6 OZ NON-WOVEN NEEDLEPUNCHED GEOTEXTILE YIELDED SIMILAR FACTOR'S OF SAFETY FOR BOTH APPLICATIONS (CRANE PAD & OPERATING SURFACE).

THE WOVEN GEOTEXTILE SHALL BE DESIGNED FOR PROTECTION OF THE GCL AND FOR SUPPLEMENTAL REINFORCEMENT WITH MINIMAL DEFLECTION (ELONGATION)

WORST CASE CONDITION WOULD OCCUR AFTER INSTALLATION, WHEN 42psi IS THE MAXIMUM STRESS AT THE GEOSYNTHETIC SURFACE. HOWEVER, THIS ANALYSIS EVALUATES INSTALLATION SURVIVABILITY, BUT USES INPUT PARAMETERS WHICH CREATE AN EQUIVALENT PRESSURE AT THE GEOSYNTHETIC SURFACE AS THE WORST CASE.

THE FOLLOWING ANALYSIS WAS PERFORMED USING AMOCO'S SOFTWARE AMOSPEC™ VERSION 1.3. THE ANALYSIS WAS PERFORMED USING 3 DIFFERENT METHODS, THE MOST CONSERVATIVE IS SHOWN BELOW. OTHER RESULTS RECOMMENDED THE SAME FABRIC, BUT WITH LESSER AGGREGATE THICKNESS (8 INCH & 4 INCH).

The aggregate thickness design parameters for the complex (Giroud and Noiray) method are:

1. The subgrade has a CBR of 1.5.
2. The axle load is 18000 lbs.
3. The traffic will be composed of on-highway trucks.
4. The tire-track width is 5 ft.
5. The design traffic is for 1000 axle load passes.
6. The tire pressure is 100 psi.
7. The allowable rut depth is 3 inches.

The geotextile design parameters are:

1. The max. contact pressure is 35 - 100 psi.
2. The minimum lift thickness is 6 inches.
3. The size of the fill aggregate is 2 inches.
4. The angularity of the base aggregate is angular.
5. Beneath the geotextile is loose to moderately compacted fill.
6. The shear strength of the subgrade is 500 - 1000 psf.
7. The inspection of construction is moderate monitoring.

The generated unpaved road design is:

Aggregate thickness with the geotextile = 12.5 inches.
The recommended Amoco geotextile for the design and survivability criteria is Amoco CEF 2006.

The geotextile shall be overlapped with a minimum overlap distance of 36 inches.

BASED ON AMOCO CEF 2006

Table 1.

PHYSICAL REQUIREMENTS - SEPARATION GEOTEXTILES

Property	Units	Value	Test Method
Machine Grab Tensile Strength	lbs.	300	ASTM-D-4632
Cross-Machine Grab Tensile Strength	lbs.	300	ASTM-D-4632
Machine Grab Tensile Elongation	%	15	ASTM-D-4632
Cross-Machine Grab Tensile Elongation	%	15	ASTM-D-4632
Mullen Burst	psi	600	ASTM-D-3786
Puncture	lbs.	120	ASTM-D-4833
Trapezoid Tear	lbs.	120	ASTM-D-4533
UV Resistance	%	70	ASTM-D-4355
UV Resistance	%	70	ASTM-D-4355
Apparent Opening Size	US Sieve	40	ASTM-D-4757
Permittivity	1/sec	.02	ASTM-D-4491
Flow Rate	Gal./min./sf	2	ASTM-D-4491

A WOVEN GEOTEXTILE WAS CHOSEN FOR THIS APPLICATION AS IT REALIZES ITS FULL TENSILE STRENGTH AT ONLY 15% ELONGATION, AS OPPOSED TO 50% FOR A COMPARABLE NON-WOVEN GEOTEXTILE.

THIS IS IMPORTANT FOR THIS APPLICATION, AS THE SUBGRADE HAS A LOW TO MODERATE BEARING CAPACITY, THUS MAY DEFORM SIGNIFICANTLY BEFORE THE STRENGTH OF A NON-WOVEN IS MOBILIZED. ACCORDING TO KOERNER (P. 141), GEOTEXTILES USED WITH SOILS WITH A CBR HIGHER THAN 6 WILL HAVE A NEGLIGIBLE AMOUNT OF REINFORCEMENT, WHILE SOILS WITH A CBR LOWER THAN 3.0 WILL HAVE A HIGHER DEGREE OF REINFORCEMENT. THIS SITE HAS AN ESTIMATED CBR OF 1.5, BEFORE PROOFROLLING, THUS WILL BENEFIT FROM THE WOVEN GEOTEXTILE.

THE ATTACHED APPENDIX A-1 OF THE PHILLIPS "STABILIZATION DESIGN GUIDE" POINTS OUT & DESCRIBES THE BENEFIT MECHANISMS OF GEOTEXTILES IN A TYPICAL ROAD SITUATION.

CONCLUSION:

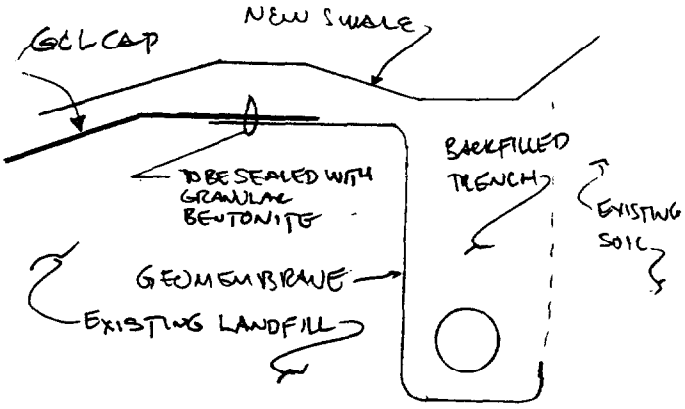
THE AMOSPEC™ PROGRAM HAS SHOWN THAT THE SPECIFIED WOVEN GEOTEXTILE, WITH A TENSILE STRENGTH OF 300 LBS AND 15% ELONGATION, WILL PROVIDE MORE THAN ADEQUATE CAPACITY TO HANDLE EXPECTED CONDITIONS.

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY MG DATE 4-13-94 SUBJECT AREA 'A' LANDFILL CAP DESIGN GEOMEMBRANE SHEET NO. 28 OF 28

CHKD. BY G DATE 4-20-94 JOB NO.

GEOMEMBRANE LINED TRENCH



THIS GEOMEMBRANE IS NOT EXPECTED TO BE SUBJECTED TO ANY SIGNIFICANT LOADS, HENCE THE DESIGN PARAMETERS WILL BE DICTATED BY INSTALLATION SURVIVABILITY REQUIREMENTS. AS SUCH, THE MINIMUM PROPERTIES AS INDICATED ON TABLE 5.11 (KOERNER 2ND ED, p. 404) FOR A HIGH DEGREE OF SURVIVABILITY, SHALL BE USED.

HDPE IS RECOMMENDED FOR ITS EXCELLENT CHEMICAL RESISTANCE, GOOD SEAMS AND LOW COST (KOERNER pp: 389, 391, 463 & 511)

IT IS ALSO RECOMMENDED THAT SEAMS RUN ALONG THE EDGE OF GEOMEMBRANE AS SHOWN ABOVE, AND THAT THE SEAM OVERLAP BE "SHINGLED" IN THE DIRECTION OF FLOW.

TABLE 5.11 RECOMMENDED MINIMUM PROPERTIES FOR GENERAL GEOMEMBRANE INSTALLATION SURVIVABILITY

Property and test method	Required degree of survivability			
	Low[a]	Medium[b]	High[c]	Very high[d]
Thickness (D1593) mils (mm)	20 (0.50)	25 (0.63)	30 (0.75)	40 (1.00)
Tensile D882 (1.0 in. [25 mm] strip) lb./in. (kN/m)	30 (5.2)	40 (7.0)	50 (8.7)	60 (10.5)
Tear (D1004 Die C) lb. (N)	5 (22)	7.5 (33)	10 (45)	15 (67)
Puncture (D3787 mod.) lb. (N)	20 (90)	25 (110)	30 (130)	35 (160)
Impact (D3998 mod.) ft.-lb. (J)	10 (7)	12 (9)	15 (11)	20 (15)

(a) *Low* refers to careful hand placement on very uniform well-graded subgrade with light loads of a static nature—typical of vapor barriers beneath building floor slabs.

(b) *Medium* refers to hand or machine placement on machine-graded subgrade with medium loads—typical of canal liners.

(c) *High* refers to hand or machine placement on machine-graded subgrade of poor texture with high loads—typical of landfill liners and covers.

(d) *Very high* refers to hand or machine placement on machine-graded subgrade of very poor texture with high loads—typical of reservoir covers and liners for heap leach pads.

APPENDIX C

SPECIAL FENCE POST BASE

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY Mle DATE 4-14-94
CHKD. BY G DATE 4-15-94

SUBJECT AREA 'A' LANDFILL
FENCE DESIGN
LINE POST FOOTING

SHEET NO. 1 OF 10
JOB NO.

DETERMINE SIZE FOOTING REQUIRED FOR LINE POSTS.

GIVEN: MAX DEPTH IS 12" DUE TO CAP THICKNESS.
USE 7 FT HIGH CHAIN LINK W/ 3 STRAND BARBED WIRE
FOOTING SPACING IS EVERY 10 FT

CALCULATE DEAD LOAD ON FOOTING

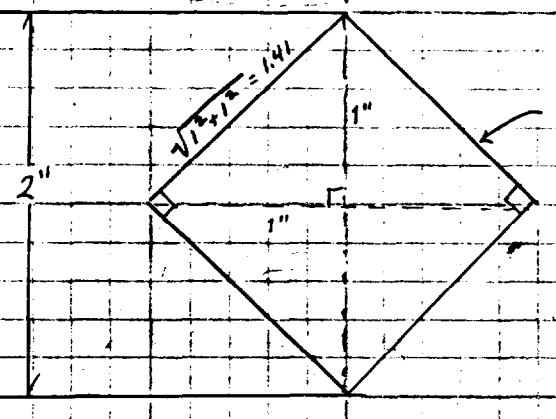
CALCULATE DEAD LOAD DUE TO FENCE FABRIC:

DIAMOND COUNT

$490 \frac{LB}{CF} \cdot \frac{CF}{12 \times 12 \times 12 \, CIN} = 0.283$

HEIGHT = 7'
POLE PLACEMENT = 10' O.C.

EACH DIAMOND MEASURES:



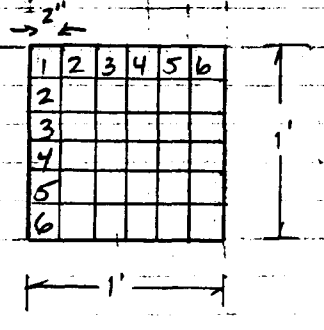
9 gage = 0.148"

$AREA = \frac{\pi (.148^2)}{4} = 0.017 \, si$

Length = 1.41 x 4 = 5.64 in.

Volume = 0.096 ci

density of steel ≈ 490 LBS/CF ≈ 0.283 LB/IN³
(FROM STEEL MANUAL p. 7-9)



DIAMOND COUNT = 36 diamonds / sf

WEIGHT OF EACH # OF FENCE FABRIC COATED WITH THE REQUIRED MINIMUM ZINC COATING OF 1.2 oz/sf (0.075 #/sf) IS

WEIGHT OF STEEL PER DIAMOND = (Density × gravity) × Volume

$= 0.283 \frac{LB}{IN^3} (0.096 IN^3)$

$= 0.027$ #/diamond

WEIGHT OF STEEL PER SF => $\frac{0.027\#}{diamond} \times \frac{36 \text{ diamonds}}{sf} = \frac{0.98\#}{sf}$

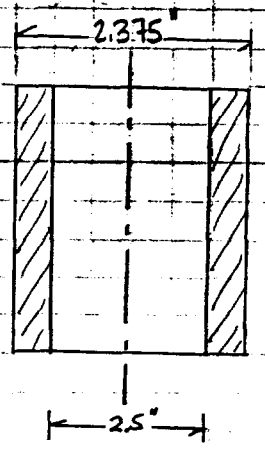
TOTAL WGT OF COATED FENCE FABRIC = Steel wgt + Zinc Coat wgt.

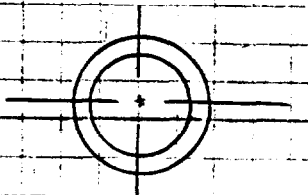
$= \frac{0.98\#}{sf} + \frac{0.075\#}{sf}$

$= \frac{1.06\#}{sf}$

CALCULATE DEAD LOAD DUE TO POST:

Assume Hollow Sch. 40 3" Fence post





USING A conservative (low) unit weight
wt/ft = 3.65 #/ft
(ref. = Boundary fencing Inc. - Sweets Cat. C2830/BOU pg. 10)

weight = $\frac{3.65\#}{ft} \times 7' = 25.55\#$

ASSUME extension arm = 2.5#

TOTAL = 28#

CALCULATE DEAD LOAD DUE TO BARBED WIRE

BARBED WIRE

Assume 9 gage wire = 0.148"

Area = 0.017 si (as previously determined)
Length = 10' = 120 IN

Volume = 0.017 si × 120 in = 2.04 ci

$\delta = 0.283 \frac{lb}{in^3}(2.04\ in^3) = 0.58$ LBS

TOTAL DEAD LOAD ON FOOTING:

= WEIGHT OF FENCE FABIC + WEIGHT OF POST + WEIGHT OF BARBED WIRE + WEIGHT OF FOOTING.
(SEE FIGURE 1.)

= (1.06 LBS/SF * 10' * 7') + 28 LBS + 3(0.58 LBS) + WT OF FOOTING

= 74.2 LBS + 28 + 1.74 + WT OF FOOTING

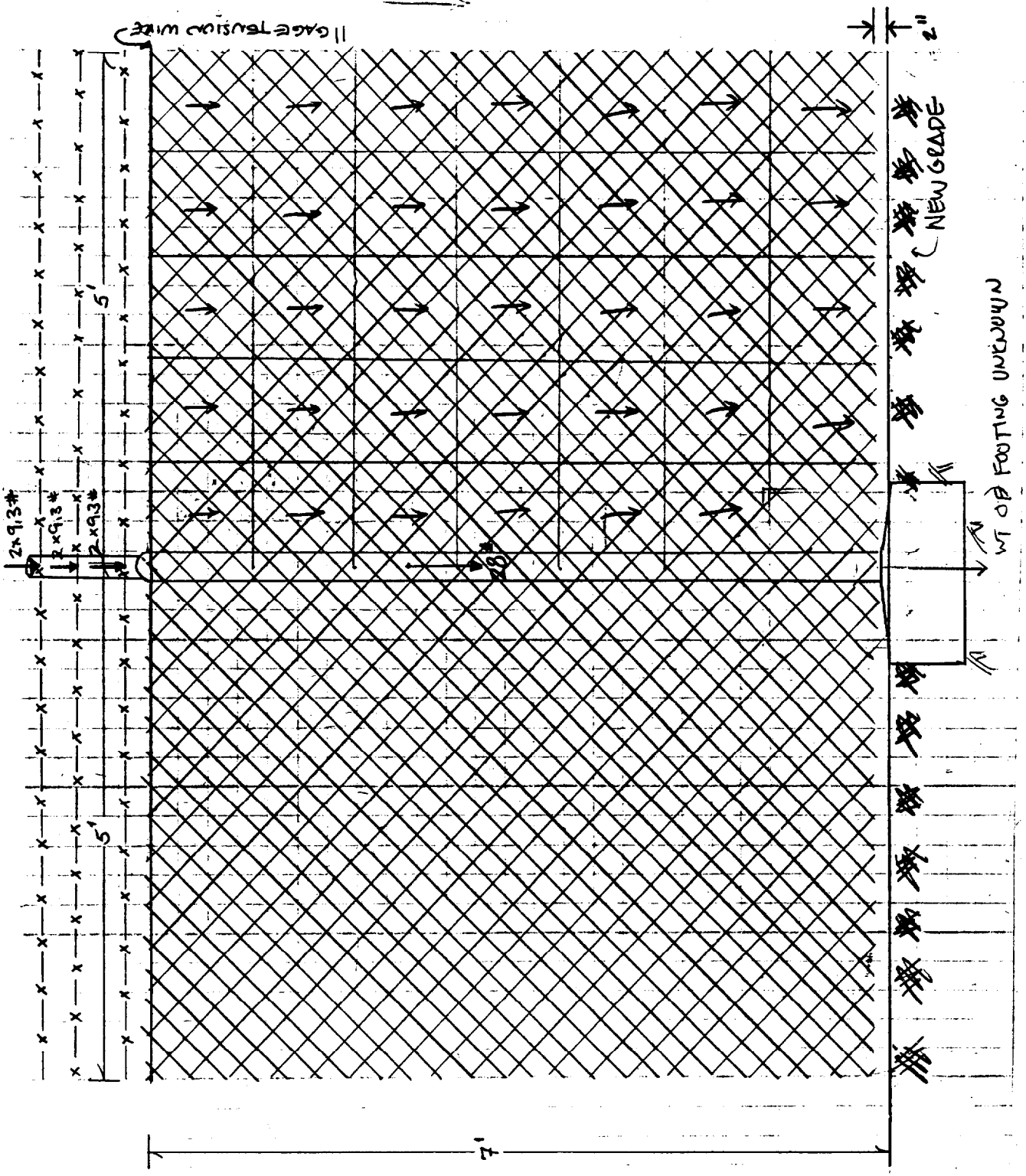
= 103.9 LBS + WT OF FOOTING.

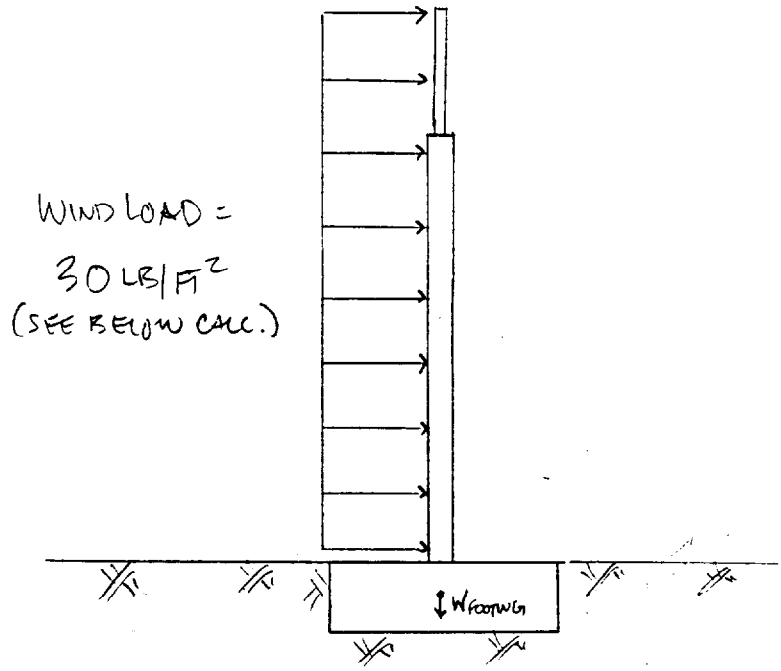
ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY MCG DATE 4-14-94 SUBJECT AREA 'A' LANDFILL FENCE DESIGN SHEET NO. 4 OF 10

CHKD. BY B DATE 4-15-94 JOB NO.

FIGURE 1





CALCULATE Pd (WIND LOAD)

$P_d = P_e \times I^2 \times C_p$ (BOCA NATIONAL BUILDING CODE/1994 p249-258)

$P_d = P_z + P_h$ = DESIGN PRESSURE FOR WIND LOADS

P_e = EFFECTIVE VELOCITY PRESSURE

I = IMPORTANCE FACTOR

C_p = EXTERNAL PRESSURE COEFFICIENT

P_z = WINDWARD DESIGN PRESSURE

P_h = LEEWARD OR SIDEWALL PRESSURE

BASIC WIND SPEED = 85 MPH (FROM FIG 1112.3.2)

FROM FIG 1112.3.3b w/ 85 MPH & EXPOSURE CLASSIFICATION C (OPEN TERRAIN) YIELDS $P_e = 22.5$ LB/FT².

FROM TABLE 1112.2a(1), WITH CATEGORY = IV (AS PER TABLE 1112.2b, LOW HAZARD TO HUMAN LIFE IN THE EVENT OF FAILURE), AND AT HURRICANE OCEANLINE YIELDS IMPORTANCE FACTOR (I) OF 1.00.

DETERMINE C_p.

FROM PREVIOUS DRAWING

EACH DIAMOND IS COMPRISED OF

$(4 * 1.41'')(0.148'') = 0.84 \text{ IN}^2$

AND 36 DIAMONDS / SF YIELDS $36 * 0.84 \text{ IN}^2 = 30 \text{ IN}^2/\text{SF}$

AND ϵ = SOLID AREA / GROSS AREA

$\therefore \epsilon = 30 \text{ IN}^2 / 144 \text{ IN}^2 = 0.21$

FROM TABLE 1112.2h w/ $\epsilon = 0.21$ &

$D\sqrt{P_e} = (0.148'')(\text{FT}/12'')\sqrt{22.5} = 0.06 \leq 2.5$

$\therefore$ $C_p = 1.3$

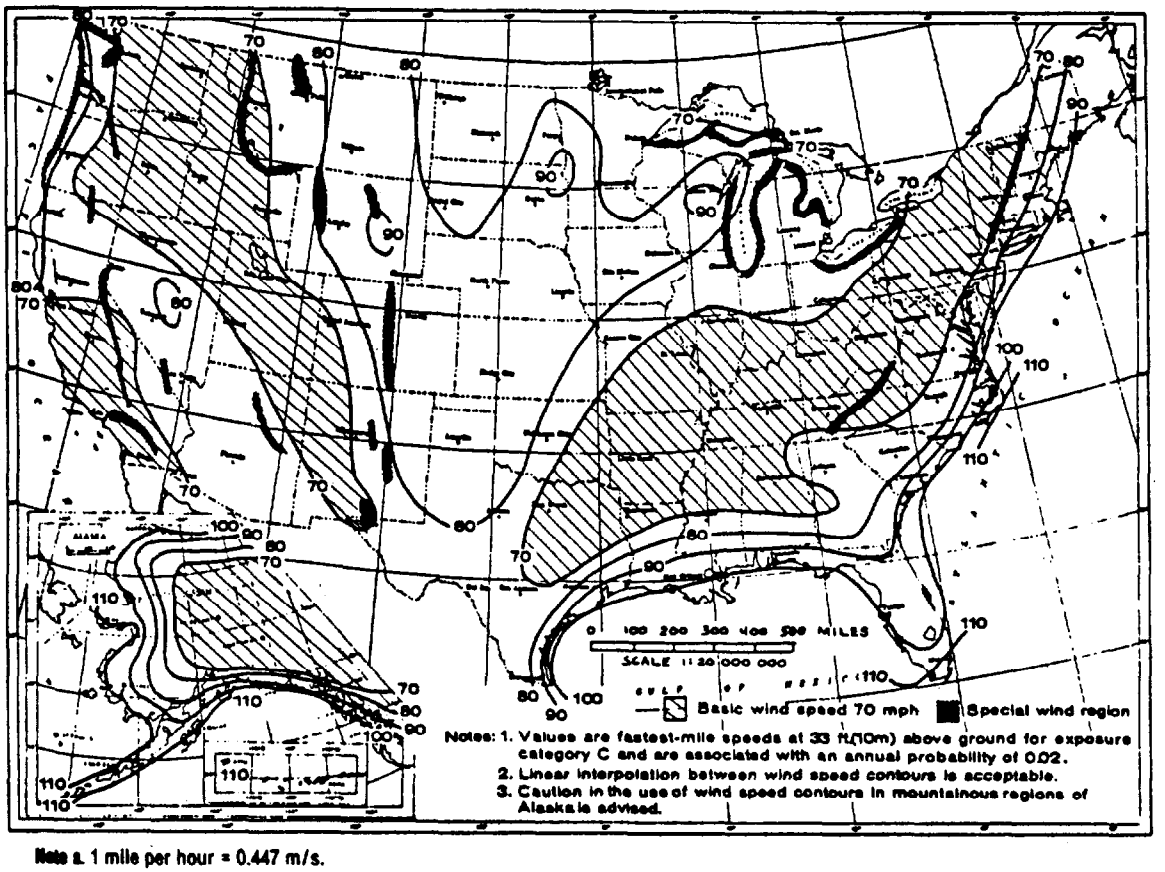
$P_d = P_e \times I^2 \times C_p = 22.5 \frac{\text{LB}}{\text{FT}^2} \times 1.00^2 \times 1.3 = 29.25 \text{ LB/FT}^2$

DESIGN WINDLOAD $= P_d * \epsilon = 29.25 * 0.21 = 6.29 \text{ LB/FT}^2 \text{ FENCE}$

TOTAL WIND LOAD ACTING ON A SINGLE POST

$= 6.29 \text{ LB/FT}^2 \text{ FENCE} * 10 \text{ FT FENCE} = 62.9 \text{ LBS/FT OF HEIGHT}$

Figure 1112.3.2
BASIC WIND SPEED (miles per hour)[a]



Note a. 1 mile per hour = 0.447 m/s.

Table 1112.3.3b
EFFECTIVE VELOCITY PRESSURES P_e (lb/ft²) FOR BUILDINGS AND STRUCTURES (EXPOSURE C)

Height above grade (ft)	Basic wind speed (mph)				
	70	80	90	100	110
0-20	15	20	25	31	37
20-40	16	21	27	33	40
40-60	19	25	31	38	47
60-100	20	26	33	41	50
100-150	23	30	37	46	56
150-200	24	31	39	49	59
200-300	25	33	42	51	62
300-400	28	36	46	56	68
> 400	Per ANSI A58.1 listed in Appendix A.				

Note a. 1 pound per square foot = 47.88 P; 1 mile per hour = 0.447 m/s; 1 foot= 304.8 mm.

Table 1112.2b
CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES FOR WIND LOADS

Nature of occupancy	Category
All buildings except those listed below	I
Buildings and structures where the primary occupancy is one in which more than 300 people congregate in one area	II
Buildings and structures designated as essential facilities, including, but not limited to: 1. Hospital and other medical facilities having surgery or emergency treatment areas 2. Fire or rescue and police stations 3. Primary communication facilities and disaster operation centers 4. Power stations and other utilities required in an emergency 5. Structures having critical national defense capabilities	III
Buildings and structures that represent a low hazard to human life in the event of failure, such as agricultural buildings, certain temporary facilities, and minor storage facilities	IV

Table 1112.2a(1)
IMPORTANCE FACTOR, I (WIND LOADS)

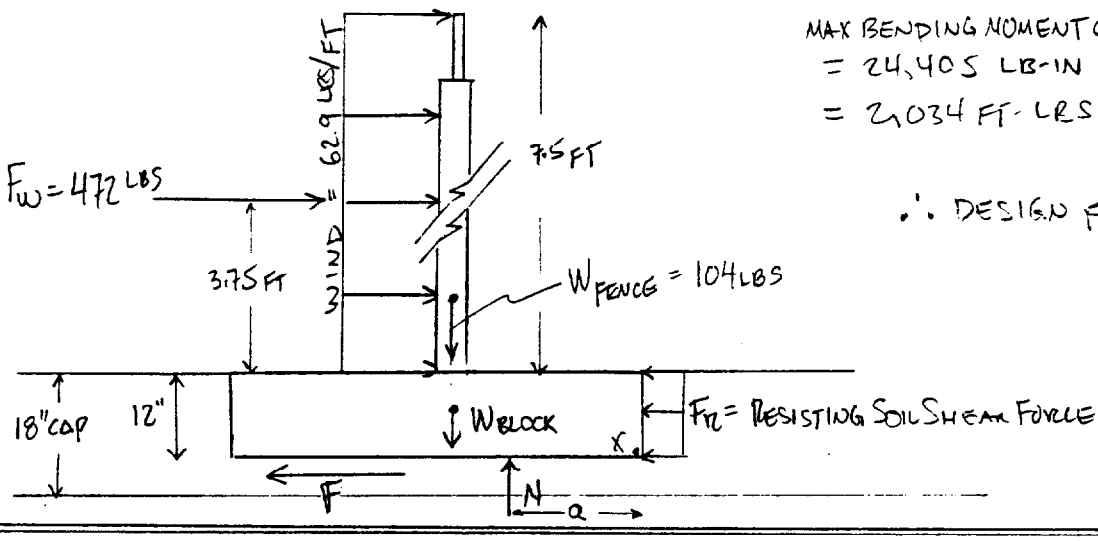
Category[a]	Importance factor, I[b]	
	100 miles (161 km) from hurricane oceanline, and in other areas	At hurricane oceanline[c]
I	1.00	1.05
II	1.07	1.11
III	1.07	1.11
IV	0.95	1.00

Note a. For building and structure classification categories, see Table 1112.2b.
Note b. For regions between the hurricane oceanline and 100 miles (161 km) inland the importance factor I shall be determined by linear interpolation.
Note c. Hurricane oceanlines are the Atlantic and Gulf of Mexico coastal areas.

Table 1112.2h
FORCE COEFFICIENTS FOR OPEN SIGNS[a] AND LATTICE FRAMEWORKS, C_p[b,c]

ϵ	C_p		
	Flat-sided members	Rounded members	
		$D\sqrt{P_e} \leq 2.5$	$D\sqrt{P_e} > 2.5$
< 0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

Note a. Signs with openings comprising 30 percent or more of the gross area are classified as open signs.
Note b. The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind direction.
Note c. Notation:
ϵ : Ratio of solid area to gross area
D: Diameter of a typical round member, in feet



MAX BENDING MOMENT CAPACITY OF 2½" POST
= 24,405 LB-IN (PER SWEETS CATALOG)
= 2,034 FT-LBS > MAX WIND INDUCED MOMENT
(472 × 3.75 = 1770)

∴ DESIGN FOR MAX WIND LOADING

TRY 6 FT × 3 FT × 1 FT CONCRETE BLOCK FOUNDATION (w/ 6 FT PERPENDICULAR TO FENCE LINE).

CHECK AGAINST SLIPPING:

$\Sigma F_y = W_{FENCE} + W_{BLOCK} + N = 104 + (6 \times 3 \times 1 \times 150\#/CF) + N = 0$

SOLVE FOR N: $N = 2804$ LBS

$\Sigma F_x = F_{WIND} - F - F_R$ ASSUME $F_R = 0$ (ADD'L FACTOR OF SAFETY)

$= 472 - \mu_s N = 472 - (0.3)(2804) = -370$ ∴ NO MOVEMENT

FACTOR OF SAFETY AGAINST SLIPPING = 841 / 472 = 1.78 (+ F_R)

CHECK AGAINST OVERTURNING:

$\Sigma M_x = (F_{WIND} * 4.75) + (N * a) - (W_{BLOCK} + W_{FENCE})(6/2)$

$= (472 \times 4.75) + (2804 * a) - (2700 + 104)(3)$

SOLVE FOR a:

$a = (8412 - 2242) / 2804 = 2.2$ FT

∴ O.K., FALLS WITHIN MIDDLE THIRD OF BLOCK.

CHECK USING MAXIMUM MOMENT CAPACITY OF POST

MAX MOMENT = 2034 FT-LBS

OR AN EQUIVALENT WIND FORCE OF 542.4 LBS = F_{EW}

CHECK AGAINST SLIPPING:

$\Sigma F_x = F_{EW} - \mu N = 542.4 - (0.3 * 2804) = -299$

∴ NO SLIPPAGE

FACTOR OF SAFETY AGAINST SLIPPING = 841.2/542.4 = 1.55

CHECK AGAINST OVERTURNING:

$\Sigma M_x = (F_{EW} * 4.75) + (2804 * a) - (2804 * 3)$.

a = 2.08 FT ∴ OK. WITHIN MIDDLE THIRD OF BLOCK.

CONCLUSION:

FENCE WILL HOLD UP AGAINST DESIGN WIND LOADS, AND, UNDER ANY LOADING CONDITION, THE FENCE POST WILL FAIL IN BENDING BEFORE THE FOOTING SLIPS OR OVERTURNS.

APPENDIX D

EARTHWORK AND MATERIALS BALANCE

SHEET 1 OF 2

AAC&F.WK3

Print Date: 04/13/94

G 4-20-94

PRELIMINARY
CUT AND FILL VOLUMES
AREA "A" LANDFILL
CONTAMINATED MATERIAL*

Station	Clear & Grub* Sq-Ft	Volume Cu-Ft	Fill @ Left Sq-Ft	Volume Cu-Ft	Other Fill Sq-Ft	Volume Cu-Ft	Cut from Pipes Sq-Ft	Volume Cu-Ft	Other Cut Sq-Ft	Volume Cu-Ft	Compact by 0.1 ft* Sq-Ft	Volume Cu-Ft
800	0		0		3		0		42		4.7	
		0		0		150		0		11100		960
900	0		0		0		0		180		14.5	
		1225		0		7950		0		14700		1575
1000	24.5		0		159		0		114		17	
		1225		0		15250		0		10750		2300
1100	0		0		146		0		101		29	
		3400		2550		8800		0		8250		2900
1200	68		51		30		0		64		29	
		6700		6300		2450		0		9700		2850
1300	66		75		19		0		130		28	
		6450		6000		1500		0		12000		2550
1400	63		45		11		0		110		23	
		6150		3950		1600		0		8750		2175
1500	60		34		21		0		65		20.5	
		6550		6200		1650		0		4200		2075
1600	71		90		12		0		19		21	
		6250		7000		1300		0		4050		2200
1700	54		50		14		0		62		23	
		4600		4450		1350		0		10100		2600
1800	38		39		13		0		140		29	
		1900		1775		825		2100		5650		1387.5
1850	38		32		20		84		86		26.5	
		1775		1925		900		4975		6450		1262.5
1900	33		45		16		115		172		24	
		4150		4150		800		10850		13700		2200
2000	50		38		0		102		102		20	
		5300		3400		600		8950		7350		2000
2100	56		30		12		77		45		20	
		5600		3450		4050		7350		4350		1850
2200	56		39		69		70		42		17	
		3400		2400		7200		7700		2600		1625
2300	12		9		75		84		10		15.5	
		1900		1700		10900		9400		700		1475
2400	26		25		143		104		4		14	
		2225		1250		4325		3475		2375		750
2450	63		25		30		35		91		16	
TOTAL (CF)		68800		56500		71600		54800		136775		34735
TOTAL (CY)		2548.15		2092.59		2651.85		2029.63		5065.74		1286.48

SEE NOTES ON SHEET 2.

NOTES

Add'l cut:	At new discharge swale: Volume = (38.5 * 110) / 27	157	CY
Add'l cut:	At new large headwall: Volume = ((30*15*4)+(40*2*7))/27	87	CY
Add'l fill:	2 Pipelines to be removed: Vol. = (11sf * 250lf) / 27	102	CY
A.	Cut = Cut from pipes + Other cut + Add'l cuts =	7,339.6	CY
B.	Fill = Fill @ left + Other fill + Add'l fill =	4,846.3	CY
C.	Clear and grub volume is compacted by 50% thus adding fill =	1,274.1	CY
D.	Compact by 0.1 foot effectively adds fill =	1,286.5	CY
E.	Total cut is reduced by 10% due to shrinkage (after refilled and compacted) =	734.0	CY
	Total Fill (Material Needed) = B + C + D =	7,406.9	CY
	Total Cut (Material Removed) = A − E =	6,605.7	CY
	Cut − Fill =	(801.2)	CY
	Area of Landfill Cap (from ACAD) =	343,381	SF
	Excess Fill: Therefore material would need to be imported (to raise cap on average througout)	−0.063	FT

NOTE: "C" assumes that the topsoil from clearing and grubbing will be used as the lower part (clean part) of the fill on the side slope at the wetland.

* Compact by 0.1 ft is an estimate of additional fill volume created by proof rolling (within the area of the cap).
* Includes only clearing and grubbing along the wetland. Additional clearing and grubbing soil along the east edge will only add to the fill volume. This soil will be considered to be suitable for clean backfill.
* Additional clean soils, not included in this analysis, will come from cuts made outside the limits of fill, some may be available to make up the 801.2 cy

Clean soil from storm pipe trench	1122.26	CY
Clean soil from interceptor pipe trench	747.41	CY

NOTE: This analysis was performed prior to final plans and sections completion to assure reasonable balance.

FINAL EARTHWORK VOLUME COMPUTATIONS, TO BE USED FOR COST ESTIMATION, FOLLOW THIS REPORT.
SEE SPREADSHEET IN POCKET AT BACK OF THE REPORT.

APPENDIX E

RIPRAP SIZING AND FLAT BOTTOM SWALE

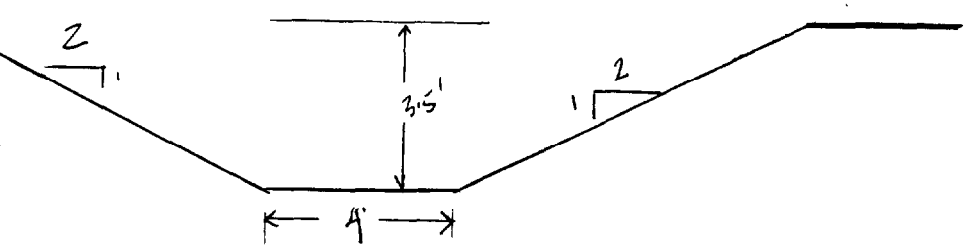
ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY [illegible] DATE 4-25-94 SUBJECT AREA 'A' LANDFILL DISCHARGE SWALE FINAL ANALYSIS SHEET NO. 1 OF 2

CHKD. BY _____ DATE _____ JOB NO. _____

DISCHARGE SWALE DESIGN;

SLOPE : 0.25%
AREA : 38.50 SF
WP : 19.65 FT
R_{HYD} : 1.96 FT
n : 0.03
LENGTH: 105 FT



FIND FREEBOARD AND VELOCITY AT MAX EXPECTED FLOW

MAX FLOW EXPECTED :
FROM DRAINAGE ANALYSIS SHEET 24 OF 30
I = 6.0 in/hr
FROM DRAINAGE ANALYSIS SHEET 23 OF 30
TOTAL AREA OF A THROUGH D = 30.1 Ac
AVG. C = 0.43.

$Q = CiA = (0.43)(6.0\ in/hr)(30.1\ Ac) = 77.7\ CFS$

TRY 0.9 FT FREEBOARD :
AREA = 23.92, WETTED PERIM = 15.6, R_{HYD} = 1.53
$V_{FULL} = 1.486/0.03\ (0.0025)^{1/2}(1.53)^{2/3} = 3.29\ FPS$
$Q = AV = 23.92 * 3.29 = 78.66 \approx 77.66$
∴ FULL FLOW OCCURS AT 0.9 FT FREEBOARD (OR 2.6 FT DEPTH)

CONCLUSION :
THE DISCHARGE SWALE PROVIDED IS ADEQUATE TO HANDLE EXPECTED FLOWS (WITH 0.9 FT OF ADDITIONAL CAPACITY) WHILE HAVING A MAXIMUM VELOCITY (3.3 FPS) LOW ENOUGH TO PREVENT EROSION OR DISTURBANCE TO THE WETLAND.

THE VELOCITY AT THE WETLANDS IS FURTHER REDUCED AS THE RIPRAP SHAPE TRANSITIONS TO 27 FEET WIDE, AND FLAT TO MATCH WETLANDS. THIS IS EXPECTED TO REDUCE FLOW VELOCITY TO 2.8 FEET PER SECOND OR LOWER. THIS IS ABOVE ERODABLE VELOCITY FOR BARE SILT/LOAM IN THE WETLANDS BUT VEGETATION IN THE WETLANDS PROVIDES ADDITIONAL PROTECTION FROM EROSION; ALLOWABLE VELOCITY 3.5 FT/SEC OR HIGHER.

THE ABOVE CONDITIONS ARE FOR A 25 YEAR STORM; THE NORMAL VELOCITIES WILL BE LOWER. FOR COMPARISON, A ONE YEAR STORM (APPROX. 40 CFS) WOULD BE EXPECTED TO ARRIVE AT THE WETLANDS INTERFACE AT A DEPTH OF 9 INCHES AND A VELOCITY OF 2.0 FT/SEC. THIS IS NOT EXPECTED TO ERODE WETLAND SOILS, EVEN IF BARE.

APPENDIX F

STORM DRAINAGE CALCULATIONS

DRAINAGE CALCULATIONS

FOR THE

AREA "A" LANDFILL

AT THE

U.S. SUBMARINE BASE, NEW LONDON, GROTON CONNECTICUT

MARCH 24, 1994

CHECKED BY/APPROVED

4-18-94

CHARLES C. CORSON P.E.

The following is an analysis of run-off from upstream of the Area "A" Landfill at the U.S. Submarine Base, New London, CT. for purposes of surficial cap design.

The drainage areas have been determined based on the best available information, which includes the following:

"Existing Storm Sewer System" Mapping, By Louriero Engineering Associates, December 31, 1980. Which shows 2 foot contours, existing buildings, pavement, roads, storm sewers etc.

Photogrammetric Survey of the Area A Landfill, which includes 1 foot contours, performed in 1993 for use in this project.

Aerial Photographs, taken in April 1993.

As-Built drawings of newly developed areas (ie. after 1980 mapping) within the drainage basin. These maps show new design contours and drainage structures, but are not necessarily "as-built" surveys.

Information gathered by Atlantic Environmental Services, Inc. staff on site visits during design.

These were all used to create a composite map (of existing conditions) of the entire drainage basin. This composite map was used to delineate six (6) different watersheds (A through F) which affect the Area A Landfill. A simplified version of this map (showing 10 foot contours) is included as Plate 1. Run-off calculations were then performed on each watershed.

As the entire drainage area was less than 200 acres, the Rational Method (Q=CiA) was used to determine run-off. However, rainfall duration (used to determine i) was calculated using the Tc Method (Time of Concentration) found in SCS-TR No. 55.

Runoff coefficients were selected from Figure 9-2, Values of Runoff Coefficients (C) for Rational Formula (CGSESC p. 9-4). Mannings Coefficients (n) were selected from Table XIV, Values of Effective Absolute Roughness and Friction Formula Coefficients (WPCF p. 84).

"Time of Concentration of Small Drainage Basins", from the Drainage Manual, Connecticut Department of Transportation (CONDOT) was used for overland flow calculations.

Figure 9-4, Rainfall Frequency-Intensity-Duration Chart for New Haven CT (CGSESC p. 9-6) was used for this project.

The design storm used for this project is a 25 year return. Normal government practice would require a 10 year storm but, due to the environmental requirements (Connecticut Department of Environmental Protection (DEP) standard), the larger storm event has been used. It has been assumed that all upstream catch basins will pick-up 100% of the flow during storm events, however, in actuality, some of the flow may bypass the catch basins and continue out of the

respective watershed. This assumption is conservative and allows for future improvements to upstream systems.

REFERENCES:

Connecticut Guidelines for Soil Erosion and Sediment Control (CGSESC).

Hydrology, SCS National Engineering Handbook, Section 4, Soil Conservation Service, U.S. Department of Agriculture.

Design and Construction of Sanitary and Storm Sewers, WPCF Manual of Practice No. 9, ASCE Manual on Engineering Practice No. 37, 1982, Water Pollution Control Federation (WPCF).

Drainage Manual, Connecticut Department of Transportation.

F:\AADRAIN

NOTE: FOR LOCATION OF WATERSHEDS, SEE MAP IN POCKET AT BACK OF REPORT.

AREA "A" LANDFILL – WATERSHED "A"

using the rational method: $Q = CiA$ where Q = peak rate of runoff (cfs)
C = runoff coefficient (dimensionless)
i = rainfall intensity (in. per hr.)
A = drainage Area (Acres)

Weighted Runoff Coefficient (C_w)

Total Area = 213,667 sf = 4.9 Ac
ROOF Area = 11,780 sf
PVMT. Area = 76,493 sf
FOREST Area = 75,236 sf
GRASS Area = 50,158 sf

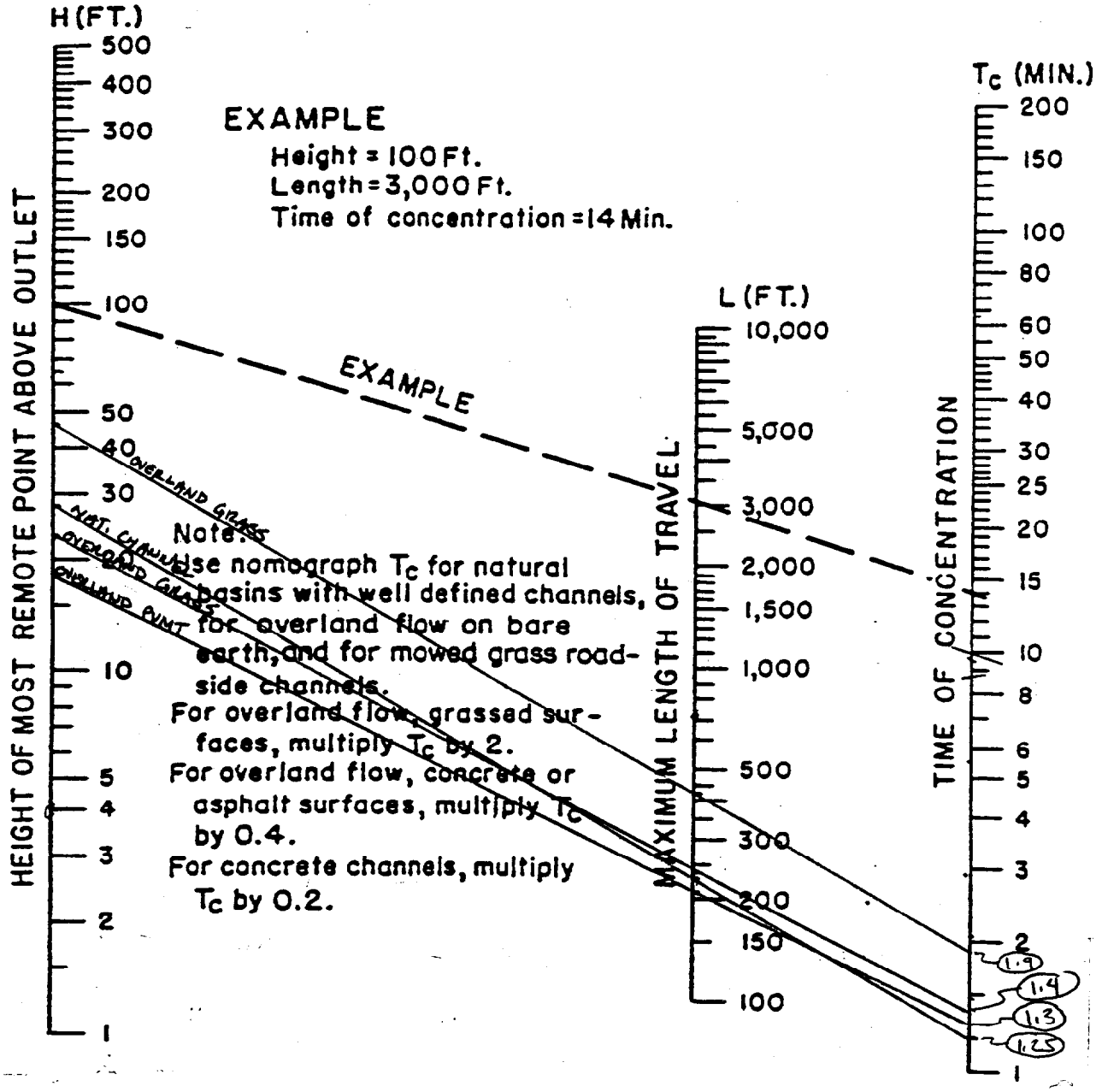
areas obtained by averaging 3 passes of a planimeter

AREA TYPE	C	Selected C	% OF TOTAL AREA	C_w
ROOF	0.75-0.95	0.95 ✓	5.51	0.0523
PVMT	0.70-0.95	0.90 ✓	35.80	0.3222
FOREST	0.10-0.30	0.30 ✓	35.21	0.1056
GRASS	0.18-0.22	0.20 ✓	23.47	0.0469
			99.99 ok	C_w = 0.53

Rainfall intensity (i)

FLOW TYPE	DISTANCE	HEIGHT (H)
overland grass	240'	24'
overland pvmt.	205'	18'
overland grass	420'	46'
overland nat. channel	230'	28'

TO OBTAIN ΣT_c refer to Kirpich nomograph

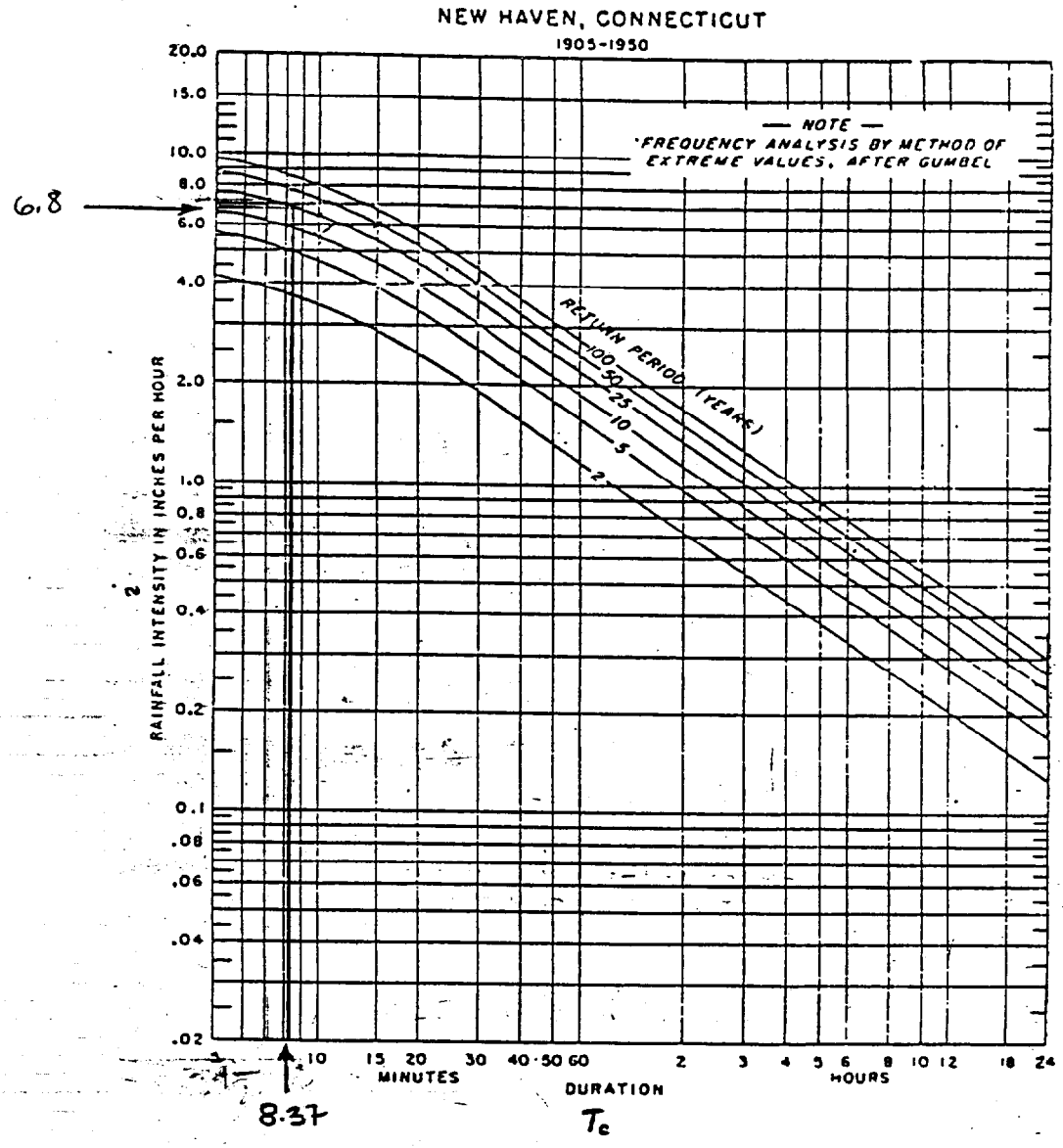


Based on study by P.Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

T_c

overland grass : $T_c = [1.9 \times 2] + [1.4 \times 2] = 6.60$ min.
overland PVMT : $T_c = [1.3 \times 0.4] = 0.52$ min
NAT. CHANNEL : $T_c = 1.25$ min

$\Sigma T_c = 6.60 + 0.52 + 1.25 = 8.37$ min



For T_c = 8.37 min, i = 6.8 in./hr

Peak rate of Runoff (Q) – WATERSHED A

$$Q = C i A$$

$$Q = (0.53)(6.8 \tfrac{in}{hr})(4.9\ Ac)$$

$$\boxed{Q = 17.66\ cfs}$$

AREA "A" LANDFILL - WATERSHED "B"

using the rational method: $Q = CiA$ where Q = peak rate of runoff (cfs)
C = runoff coefficient (dimensionless)
i = rainfall intensity (in per hr)
A = drainage Area (Acres)

Weighted Runoff Coefficient (C_w)

Total Area = 318,887 sf = 7.3 Ac
ROOF Area = 2,712 sf
PVMT AREA = 76,775 sf
FOREST AREA = 81,312 sf
GRASS AREA = 158,088 sf

areas obtained by averaging 3 passes of a planimeter

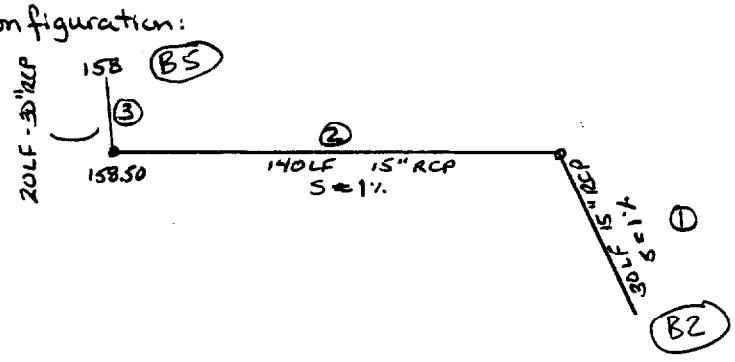
Area TYPE	C	Selected C	RATIO OF TOTAL AREA	C_w
ROOF	0.75-0.95	0.95	0.0085	0.008
PVMT	0.70-0.95	0.90	0.2408	0.217
FOREST	0.10-0.30	0.30	0.2550	0.077
GRASS	0.18-0.22	0.20	0.4957	0.099
			1.0000 OK	C_w = 0.40

RAINFALL INTENSITY (i)

	scenario #1	FLOW PATH	SCENARIO #2	FLOW PATH
OVERLAND GRASS -	0		780'; H = 172'	B9-B8
OVERLAND PVMT -	625'; H = 48'	B1-B2		
CHANNEL FLOW -	330'; H = 78'	B5-B6, B7-B8		
plus pipe and culvert flows		B2 → B5, B6-B7		

SCENARIO #1

pipe configuration:



USING MANNING'S EQUATION FOR PIPEFLOW VELOCITY:

$$V = \frac{1.49}{n}(R_H)^{2/3}(S_c)^{1/2}$$

where $n = 0.015$ for RCP

and $R_H = \frac{\frac{\pi D^2}{4}}{\pi D} = \frac{D}{4}$ for a circular pipe

Pipe #	S_c	R_H ($= \frac{D}{4}$)	n	$V = \frac{1.49}{n}(R_H)^{2/3}(S_c)^{1/2}$	$T_c = \frac{L}{V}$ (s)
①	0.01	0.3125	0.015	4.57	6.56
②	0.01	0.3125	0.015	4.57	30.63
③	0.025	0.6250	0.015	11.48	1.74

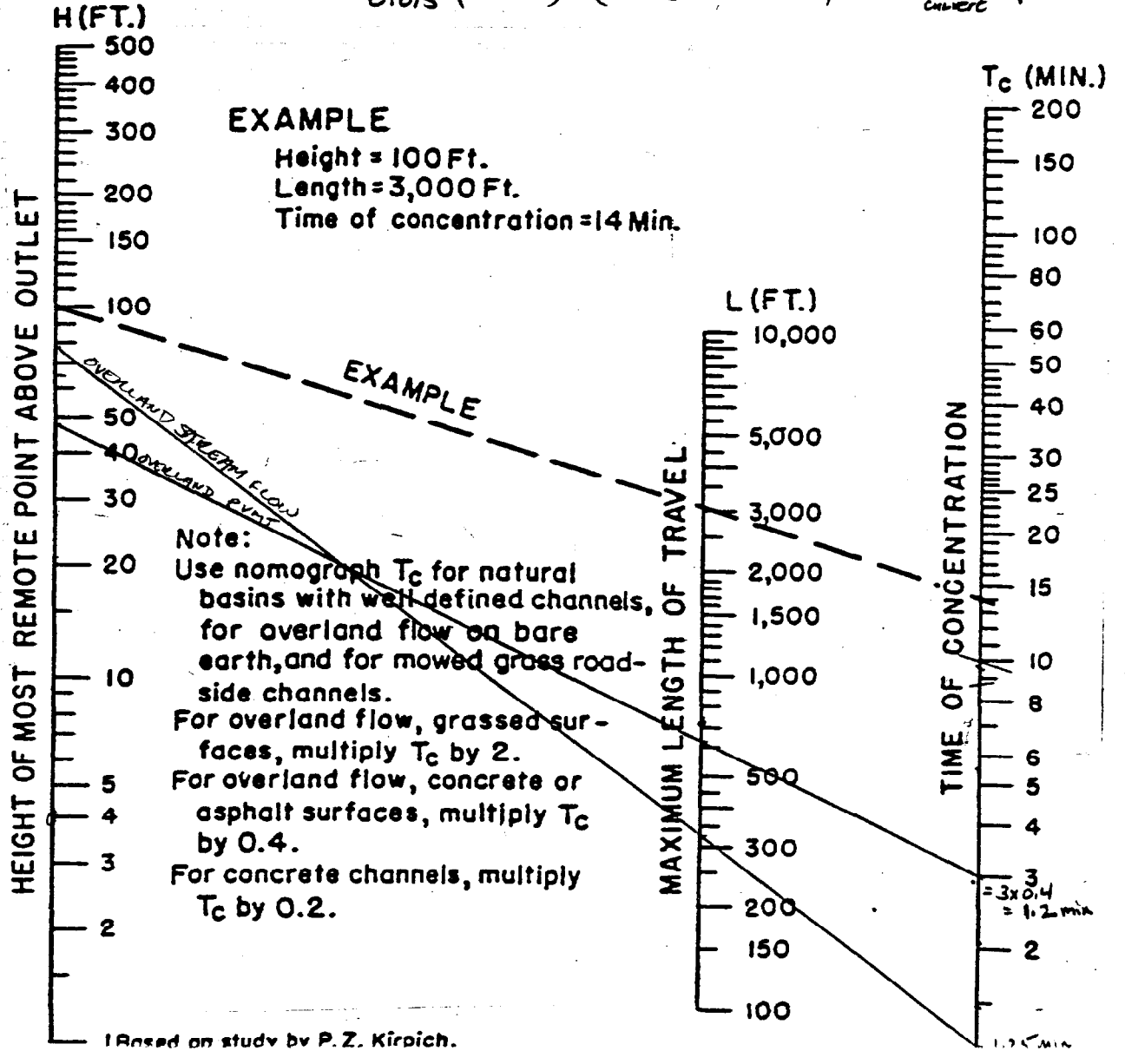
$T_{c_{pipe}} = 38.93\,s = 0.65\,min.$

Culvert flow: assume culvert is a circular 30" RCP

$S_c = \frac{4'}{25'} = 0.16$ $R_H = \frac{D}{4} = 0.625$ $n = 0.015$

$V = \frac{1.49}{0.015}(0.625)^{2/3}(0.16)^{1/2} = 29.05\,fps \Rightarrow T_{c_{culvert}} = \frac{L}{V} = 0.86\,s = 0.1\,min$

OVERLAND FLOW:

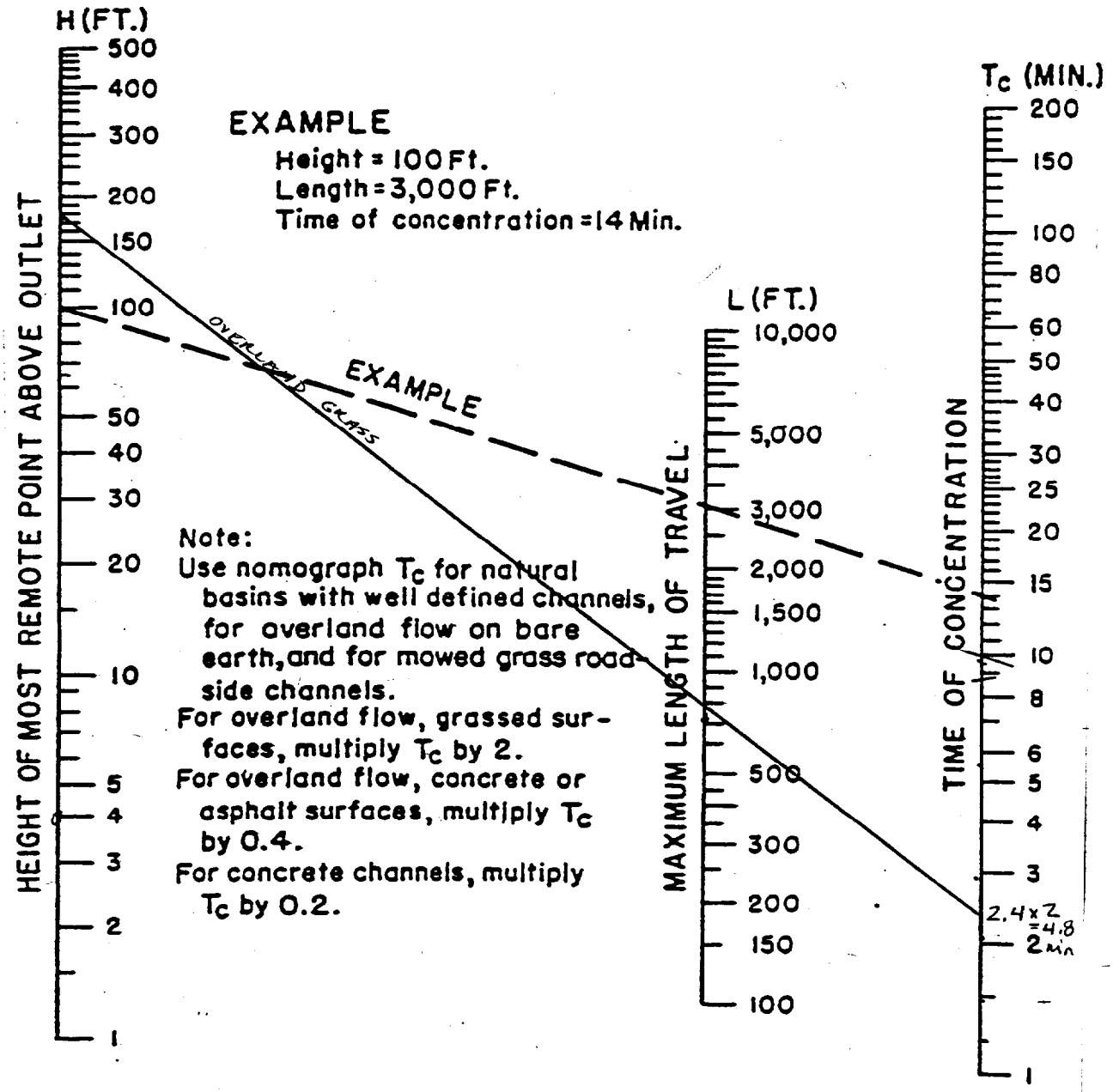


1 Based on study by P. Z. Kirpich.

T_c: scenario #1

T_{c_1} = 0.65 min + 0.01 min + 1.25 min + 1.2 min
= 3.11 min.

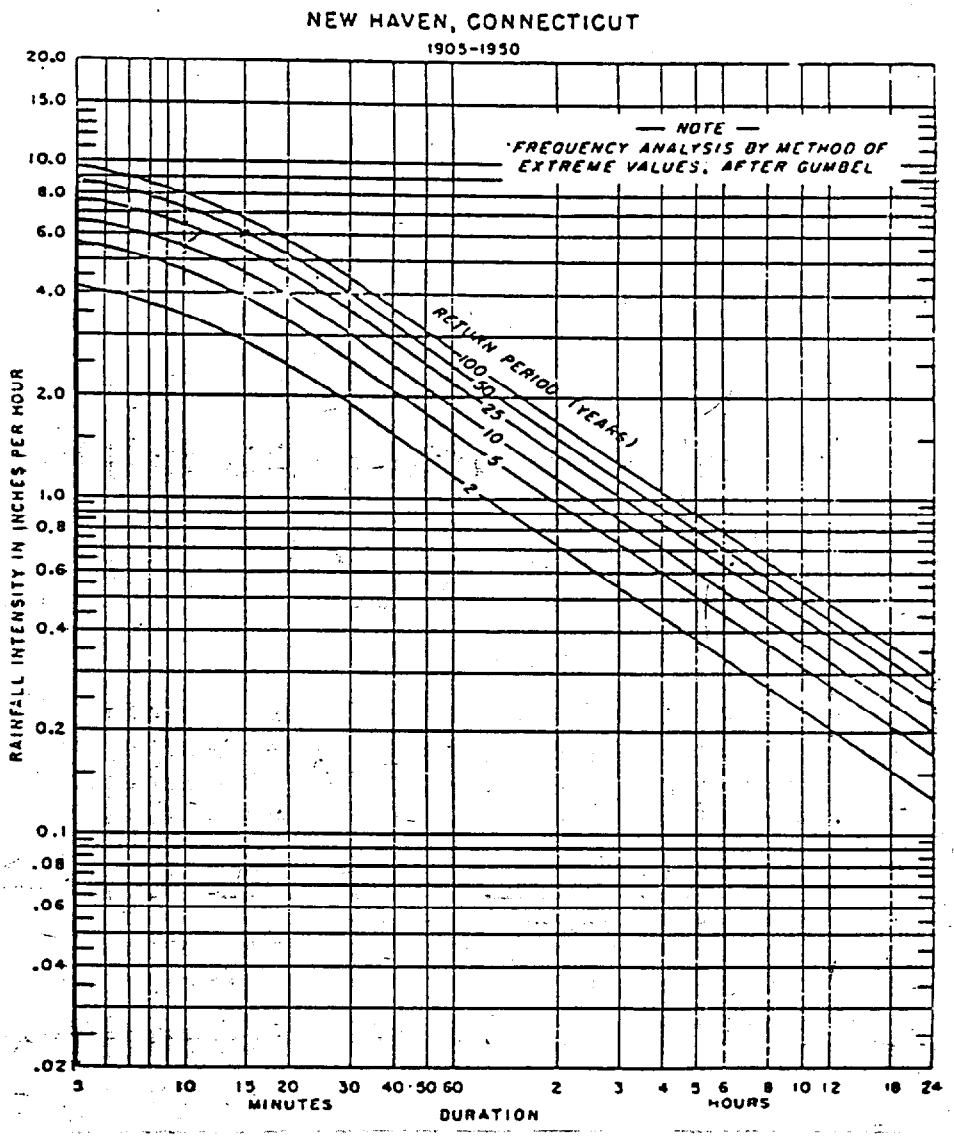
T_c: scenario #2 ; T_{c_2}



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

T_{c_2} = 4.8 min.

	scenario #1	scenario #2
T_c	3.11 min	4.8 min, USE 5 MINUTES (MINIMUM)



$i = 7.6$ in/hr

Peak rate of runoff (Q):

$$Q = CiA = (0.40)(7.6 \tfrac{in}{hr})(7.3\ Ac)$$

$$\boxed{Q = 22.19\ cfs}$$

AREA "A" LANDFILL - WATERSHED 'C'

DRAINAGE PATHS C1 → C6 & C7 → C6

RATIONAL METHOD TO BE USED

$Q = CiA$

Q = FLOW (CFS)
C = RUNOFF COEFFICIENT (DIMENSIONLESS)
i = RAINFALL INTENSITY (IN/HR)
A = AREA (ACRES)

DETERMINE C:

TOTAL AREA OF WATERSHED 'C'. USING PLANIMETER (3 PASSES) = 61.92 IN² (100²) = 619,200 FT² ✓
= 14.21 AC ✓

AREA OF BUILDINGS (3 PASSES W/ PLANIMETER) = 1.26 IN² = 12,600 SF = 0.29 AC ✓

AREA OF PAVEMENT (3 PASSES EACH W/ PLANIMETER) = 5.59 IN² = 55,900 SF = 1.28 AC ✓

REMAINING AREA WOODS = 14.21 − 0.29 − 1.28 = 12.64 AC ✓

DESCRIPTION	AREA	% OF TOTAL	C VALUE	WEIGHTED C
BUILDINGS	0.29	2 ✓	0.95	0.019 ✓
PAVEMENT	1.28	9 ✓	0.90	0.081 ✓
WOODED	12.64	89 ✓	0.30	0.267 ✓

$C_{WEIGHTED}$ = 0.37 ✓

DETERMINE i:

CALCULATE Tc:

SEGMENT	DESCRIPTION	(L) LENGTH	H1 - H2	ΔH	SLOPE	Tc* (MIN)
C1-C2	OVERLAND	240	226 - 197	29 ✓	0.1208 ✓	2.60 ✓
C2-C3	SWALE	155	197 - 178	19 ✓	0.1226 ✓	0.28
C3-C4	RIP RAP SWALE	135	178 - 154	24 ✓	0.1778 ✓	0.21
C4-C5	NATURAL SWALE	300	154 - 98	56	0.1867	1.3
C5-C6	" "	350	98 - 86	12	0.0343	2.7
						7.09
C7-C8	PAVED PARKING	295	231 - 228.5	2.5 ✓	0.0085 ✓	1.52 ✓
C8-C9	18" RCP	75	225 - 214.75	10.25 ✓	0.1367 ✓	0.065 ✓
C9-C10	18" RCP	155	2082 - 204.45	3.75 ✓	0.0242 ✓	0.32 ✓
C10-C2	NATURAL SWALE	260	204.45 - 197	7.45 ✓	0.0287 ✓	0.54 ✓
C2-C6	(FROM ABOVE) →					4.49 ✓
						6.94

* Tc → SEE SHEETS 13 OF 30 THROUGH 17 OF 30..

Tc(MAX) = 7.09

FROM NEW HAVEN RAINFALL DATA w/ 25 YR STORM & 7.09 MIN. DURATION*
* SEE SHEET 12 OF 20..

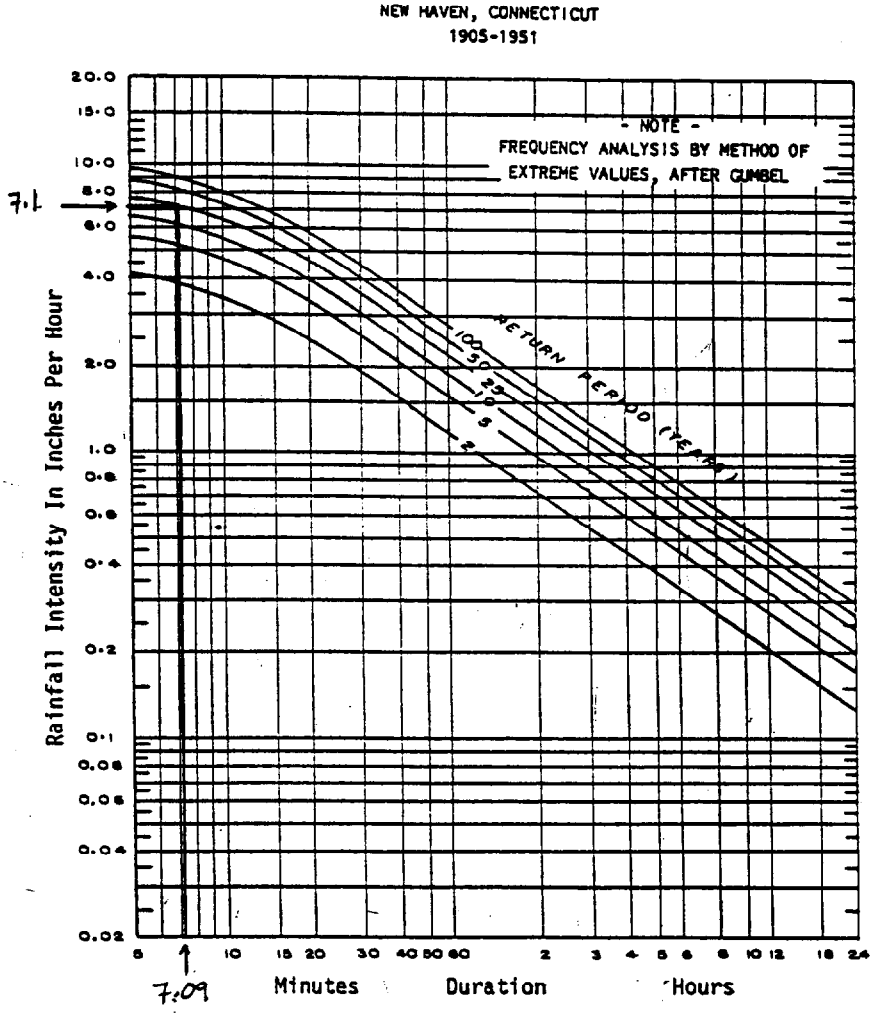
INTENSITY (i) = 7.1 IN/HR ✓

CALCULATE FLOW FROM WATERSHED C

Q = CiA
C = 0.37 ✓
i = 7.1 IN/HR ✓
A = 14.21 AC ✓

Q = 0.37 (7.1 IN/HR) (14.21 AC) = 37.33 CFS ✓

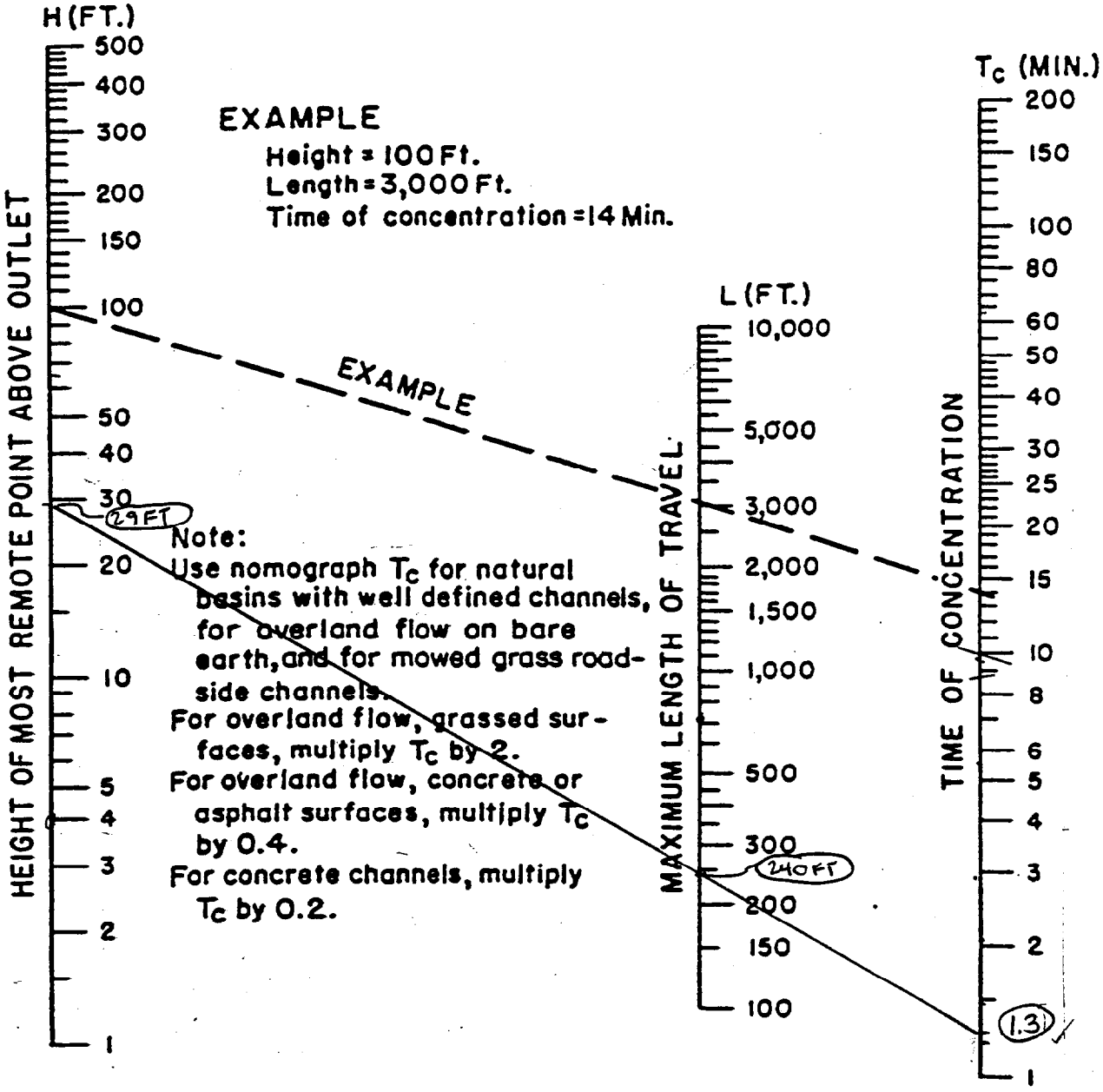
Figure 9-4 - Rainfall Frequency-Intensity-Duration Chart



Source: Connecticut Department of Transportation, Wethersfield, Connecticut.

C1-C2

OVERLAND FLOW (GRASS), 240' LENGTH, ΔH = 29 FT, SLOPE = 0.1208



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

Figure P075. Time of Concentration of Small Drainage Basins

Source: Ref. 81

C1-C2 (CON'D)

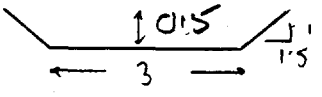
FROM KIRPICH CHART $T_c = 1.3$

HOWEVER, FOR OVERLAND FLOW, GRASSED SURFACES, MULTIPLY BY 2 ∴

$T_{c_{C1-C2}} = 1.3 * 2 = 2.6$ MINUTES

C2-C3 LENGTH (L) = 155', SLOPE (S) = 0.1226

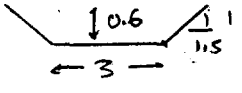
GRASSED SWALE
ASSUME DIMENSIONS



WETTED PERIMETER FULL $= 3 + 2(0.5)\sqrt{1.5^2+1} = 4.8$ FT ✓
AREA $= (3+((1.5)(0.5)))0.5 = 1.875$ SF ✓
$R_H = A/P = 1.875/4.8 = 0.39$ (HYDRAULIC RADIUS)
SLOPE (S) = 0.1226 ; n = 0.030 ✓
USE MANNING: $V = 1.49 R_H^{2/3} S^{1/2}/n$ ✓
$V = 1.49 (0.39)^{2/3} (0.1226)^{1/2}/0.03 = 9.28$ FPS ✓
TIME = L/V = 155/9.28 = 16.7 SEC = 0.28 MIN. ✓
CHECK Q: Q = AV = 1.875 * 9.28 = 17.4 CFS → O.K.

C3-C4 LENGTH (L) = 135 FT, SLOPE (S) = 0.1778.

RIP RAP SWALE
ASSUME DIMENSIONS



WET. PERIM = $3 + 2(0.6)\sqrt{1.5^2+1} = 5.16$ FT
AREA = $(3+(1.5*0.6))0.6 = 2.34$ SF
$R_H = A/P = 2.34$ SF / 5.16 FT = 0.45 FT
SLOPE (S) = 0.1778
n = 0.035 (RIP RAP CHANNELS)
USE MANNING: $V = 1.49 R_H^{2/3} S^{1/2}/n$
$V = 1.49 (0.45)^{2/3} (0.1778)^{1/2}/0.035 = 10.54$ FPS
TIME = L/V = 135/10.54 = 12.8 SEC = 0.21 MIN
CHECK Q: Q = AV = 2.34 SF * 10.54 FPS = 25 CFS → O.K.

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY [illegible] DATE 2/16/94 SUBJECT AREA A LANDFILL SHEET NO. 15 OF 30

CHKD. BY [illegible] DATE 4-18-94 JOB NO. 1256-31-05-04

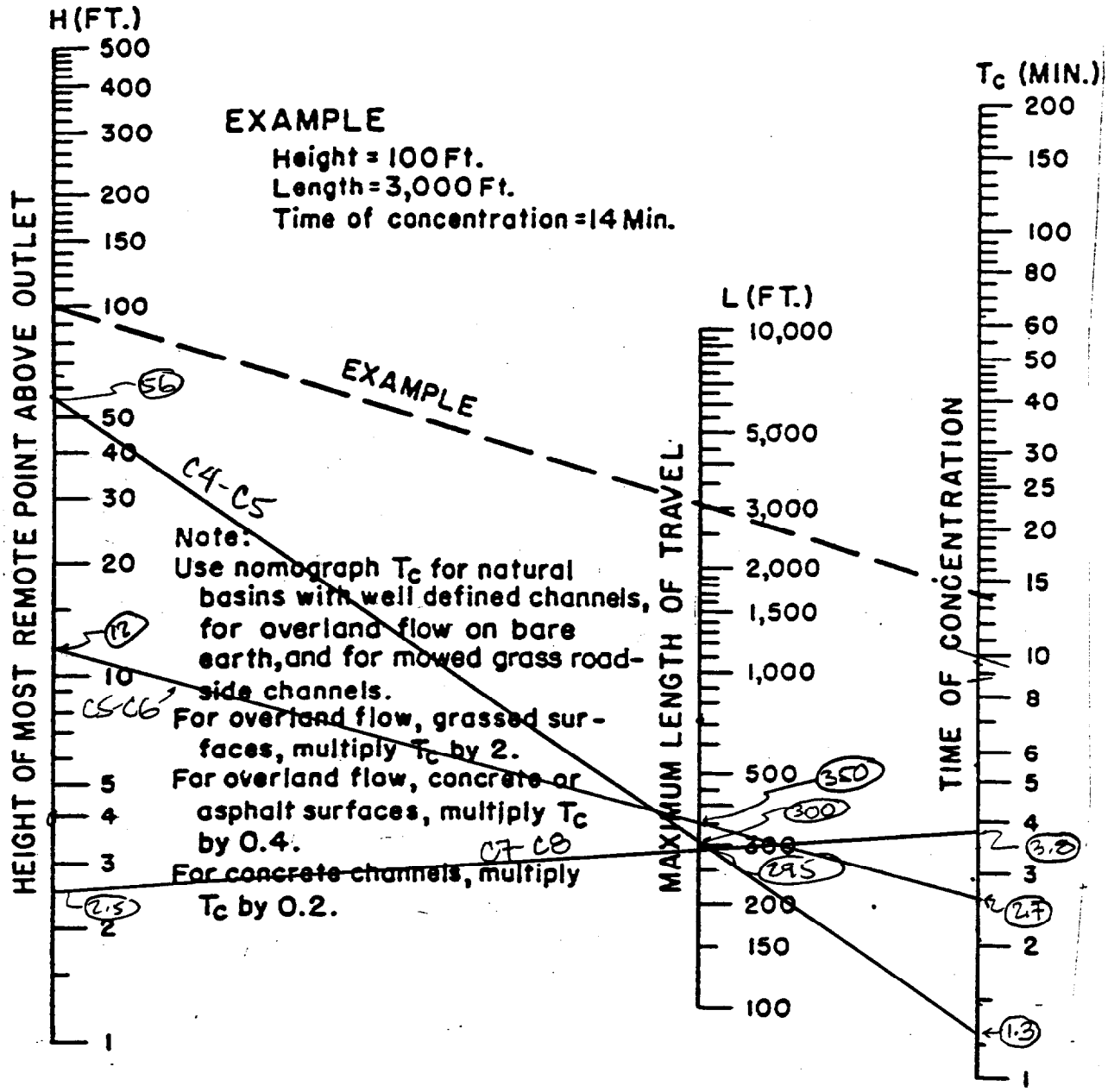
C4-C5 LENGTH (L) = 300 FT, SLOPE (S) = 0.1867, ΔH = 56 FT
FROM KIRPICH CHART, Tc = 1.3 MIN. (SEE SHEET 14 OF 20).

C5-C6 LENGTH (L) = 350 FT, SLOPE (S) = 0.0343, ΔH = 12 FT
FROM KIRPICH CHART, Tc = 2.7 MIN (SEE SHEET 14 OF 20).

C7-C8 LENGTH (L) = 295 FT, SLOPE (S) = 0.0085, ΔH = 2.5 FT
PAVED PARKING - FROM KIRPICH CHART (SEE SHEET 14 OF 20).
Tc = 3.8 MIN. HOWEVER, MULTIPLY BY 0.4 FOR OVERLAND
FLOW ON ASPHALT SURFACE ∴

Tc = 3.8 × 0.4 = 1.52 MIN.

C7-C8 (CON'D)



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

Figure P075. Time of Concentration of Small Drainage Basins

Source: Ref. 81

C8-C9 LENGTH (L) = 75 FT, SLOPE (S) = 0.1367

18" RCP

USE MANNING'S EQ : $V = 1.49 R_H^{2/3} S^{1/2}/n$

R_H (18" PIPE) = 0.375 FT

SLOPE (S) = 0.1367

n = 0.015

$V = 1.49 (0.375)^{2/3} (0.1367)^{1/2} / 0.015 = 19.10$ FPS ✓

TIME = L/V = 75 / 19.10 = 39 SEC = 0.065 MIN ✓

C9-C10 LENGTH (L) = 155 FT, SLOPE (S) = 0.0242

18" RCP

USE MANNINGS EQ: $V = 1.49 R_H^{2/3} S^{1/2}/n$

R_H (18" PIPE) = 0.375 FT

SLOPE (S) = 0.0242

n = 0.015

$V = 1.49 (0.375)^{2/3} (0.0242)^{1/2} / 0.015 = 8.04$ FPS ✓

TIME = L/V = 155 / 8.04 = 19.29 SEC = 0.32 MIN ✓

C10-C2 LENGTH (L) = 260 FT, SLOPE (S) = 0.0287

NATURAL SWALE

USE DIMENSIONS OF C2-C3

R_H = 0.94 FT

SLOPE (S) = 0.0287

n = 0.030

USE MANNINGS EQ: $V = 1.49 R_H^{2/3} S^{1/2}/n$

$V = 1.49 (0.94)^{2/3} (0.0287)^{1/2} / 0.030 = 8.07$ FPS

TIME = L/V = 260 / 8.07 = 32 SEC = 0.54 MIN ✓

AREA "A" LANDFILL - WATERSHED AREA D

DRAINAGE PATH D1 → D6.

RATIONAL METHOD TO BE USED

$Q = CiA$

Q = FLOW (CFS)

C = COEFFICIENT OF PERMEABILITY

i = RAINFALL INTENSITY

A = AREA

DETERMINE C:

TOTAL AREA OF WATERSHED D USING PLANIMETER (3 PASSES) = $16.09 \text{ in}^2 * 100^2 = 160{,}900$ SF = 3.69 Ac

AREA OF BLDGS (3 PASSES w/ PLANIMETER)

$2.07 + 0.72 = 2.79 \text{ in}^2 = 27{,}900$ SF = 0.64 Ac

AREA OF PVMT (3 PASSES w/ PLANIM.)

$2.16 + 1.9 + 1.04 = 5.10 \text{ in}^2 = 51{,}000$ SF = 1.17 Ac

REMAINING AREA (FOREST) = $8.2 \text{ in}^2 = 82{,}000$ SF = 1.88 Ac

---

DESCRIPTION	AREA	% OF TOTAL	C VALUE	WEIGHTED C
BUILDINGS	0.64 AC	17	0.95	0.162
PAVEMENT	1.17 AC	32	0.90	0.288
FOREST	1.88 AC	51	0.30	0.153
			C WEIGHTED =	0.603

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY MG DATE 2-16-94 SUBJECT Area A Landfill SHEET NO. 19 OF 30
CHKD. BY G DATE 4-18-94 JOB NO. 1256-31-05-04

DETERMINE i:

CALCULATE Tc:

SEGMENT	DESCRIPTION	LENGTH (L)	H1 - H2	ΔH	SLOPE	Tc* (MIN)
D1-D2	OVERLAND	215	190 - 156.5	33.5	0.1558	2.0
D2-D3	GRASS SWALE	245	156.5 - 120	36.5	0.1490	0.42
D3-D4	"	290	120 - 86	34	0.1172	0.56
D4-D5	18" RCP	10'	82 - 81.75	0.25	0.0250	0.02
D5-D6	24" RCP	40'	80.5 - 80	0.50	0.0125	0.09
						3.09 MIN

* SEE SHEETS 20 OF 30 THROUGH 22 OF 30

Tc = 3.09 MIN ∴ USE 5 MINUTES

FROM NEW HAVEN RAINFALL DATA w/ 25 yr STORM & 5 MIN. DURATION*

INTENSITY i = 7.6 IN/HR

* SEE ATTACHED SHEET 22 OF 30.

CALCULATE FLOW FROM WATERSHED E

Q = CiA

C = 0.60

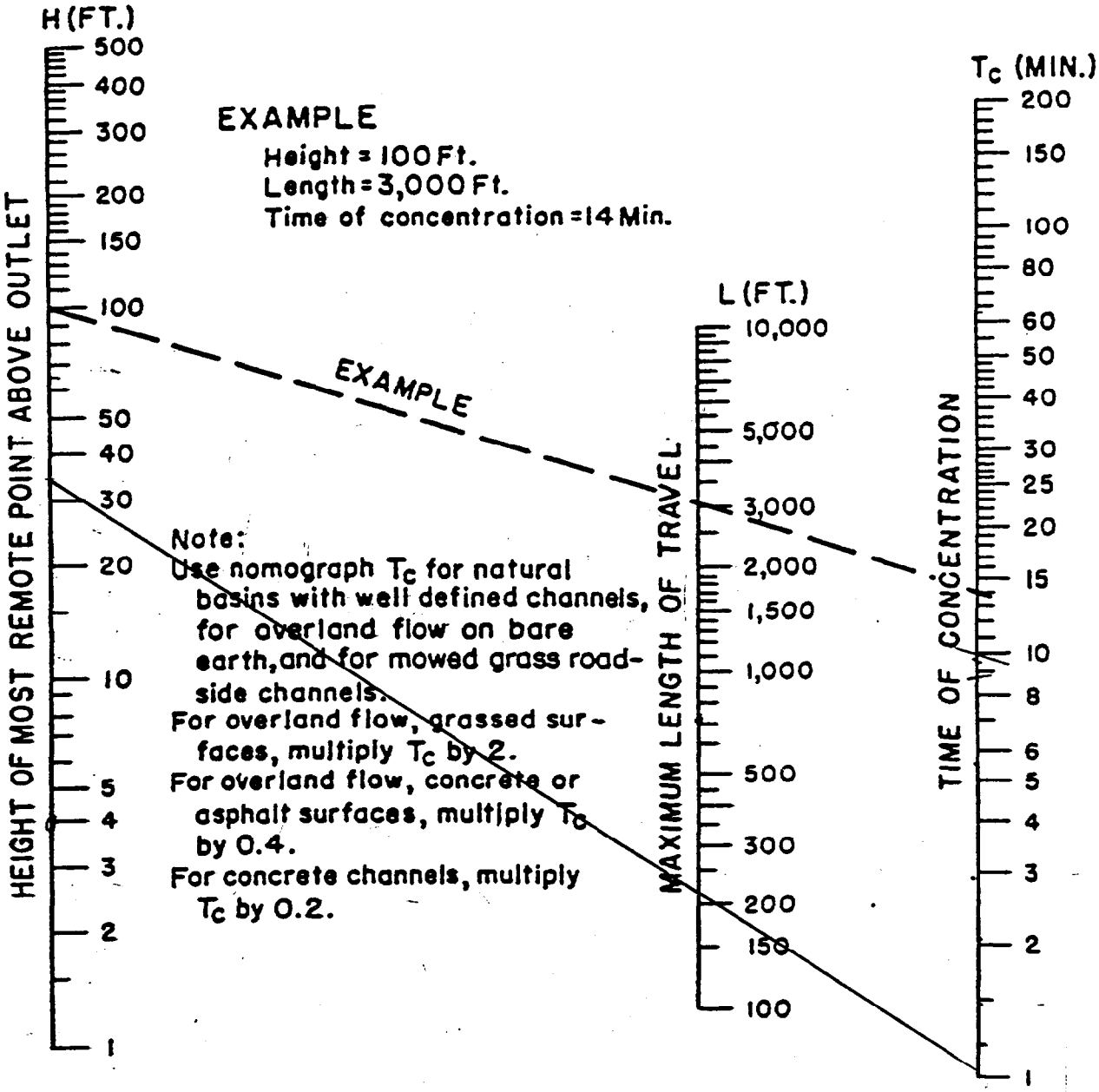
i = 7.6

A = 3.69

Q = (0.60)(7.6)(3.69) = 16.8 CFS

D1-D2

OVERLAND FLOW (GRASS) LENGTH (L) = 215 FT, ΔH = 33.5, SLOPE (S) = 0.1558



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

Figure P075. Time of Concentration of Small Drainage Basins

Source: Ref. 81

21-D2 (CON'D)

FROM KIRPICH CHART

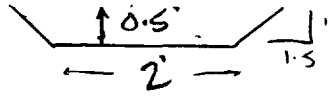
T_c = 1 MIN . MULTIPLY × 2 FOR OVERLAND, GRASSED SURFACES

∴ $T_{c\,E1-E2}$ = 2 MIN ✓

D2-D3

GRASS SWALE

ASSUME DIMENSIONS



LENGTH (L) = 245 FT, SLOPE (S) = 0.1490

WETTED PERIMETER FULL = $2 + 2(0.5)\sqrt{1.5^2+1}$ = 3.8 FT. ✓

AREA = $(2+(1.5*0.5))0.5$ = 1.38 FT² ✓

R_H = A/P = 1.38/3.8 = 0.36 FT ✓ (HYDRAULIC RADIUS)

SLOPE (S) = 0.1490 ✓

n = 0.03 (MANNINGS COEFFICIENT) ✓

$V = 1.49(0.36)^{2/3}(0.1490)^{1/2}/0.03$ = 9.7 FPS (MANNINGS EQ)

TIME = L/V = 245/9.7 = 25.25 SEC = 0.42 MIN ✓

CHECK FLOW: Q = AV = 1.38 * 9.7 = 13.4 CFS → O.K.

D3-D4 LENGTH (L) = 290 FT, SLOPE (S) = 0.1172

ASSUME SAME DIMENSIONS AS D2-D3

R_H = 0.36 ✓

S = 0.1172 ✓

n = 0.03 ✓

$V = 1.49(0.36)^{2/3}(0.1172)^{1/2}/0.03$ = 8.6 FPS ✓ (MANNINGS EQ)

TIME = L/V = 290/8.6 = 33.75 SEC = 0.56 MIN ✓

CHECK FLOW: Q = AV = 1.38 * 8.6 = 11.87 CFS → O.K.

D4-D5 LENGTH (L) = 10 FT, SLOPE (S) = 0.0250, 18" RCP.

USE MANNINGS EQ.: $V = 1.49 R_H^{2/3} S^{1/2}/n$

R_H (18" RCP) = 0.375 ✓

S = 0.0250 ✓

n = 0.015 (RCP) ✓

$V = 1.49(0.375)^{2/3}(0.0250)^{1/2}/0.015$ = 8.16 FPS ✓

TIME = L/V = 10/8.16 = 1.22 SEC = 0.02 MIN ✓

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY Mer DATE 2-16-94 SUBJECT Inner A Landfill SHEET NO. 22 OF 30

CHKD. BY G DATE 4-18-94 JOB NO. 1756-31-05-04

D5-D6 24" RCP, LENGTH (L) = 40 FT, SLOPE (S) = 0.0125

USE MANNINGS EQ : $V = 1.49 R_H^{2/3} S^{1/2} / n$

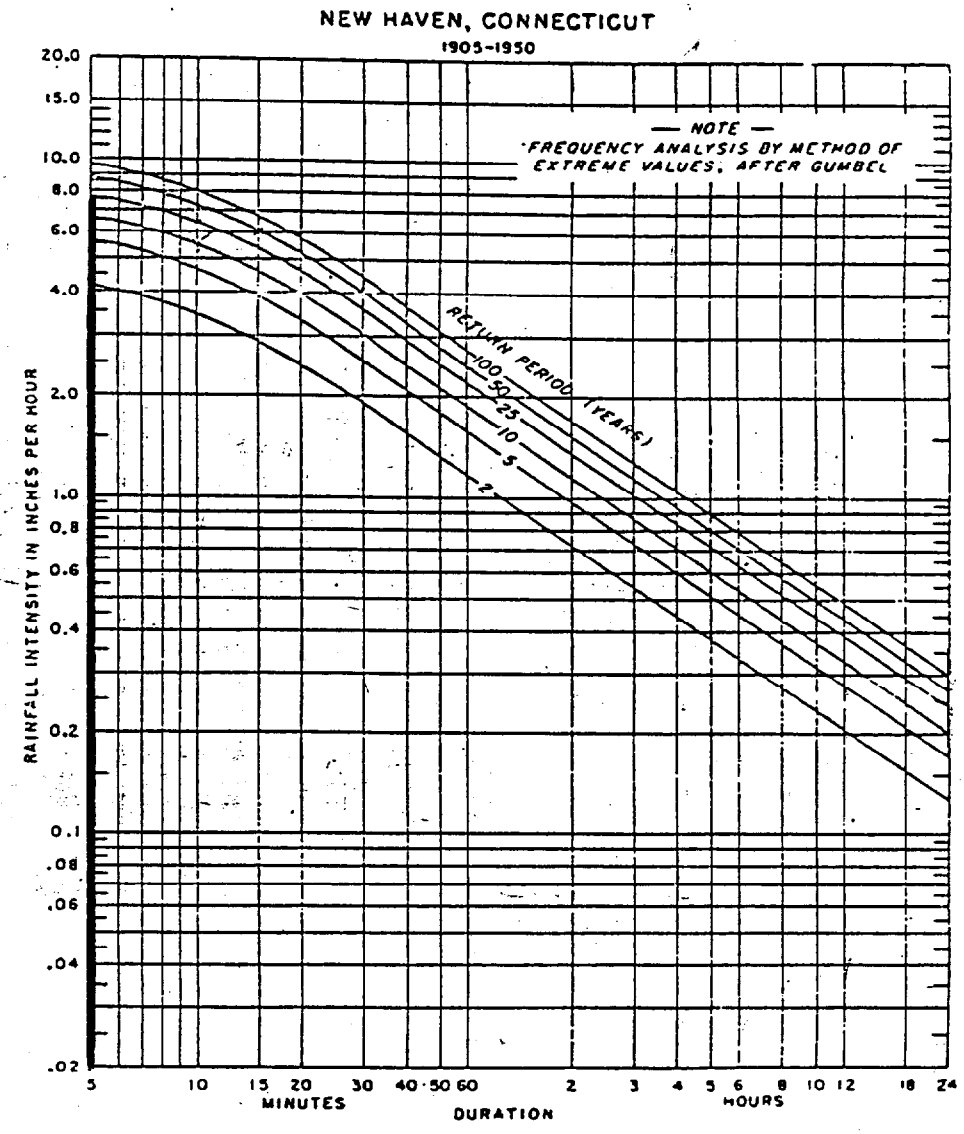
R_H (24" RCP) = 0.50 ✓

S = 0.0125 ✓

n = 0.015 (RCP) ✓

$V = 1.49 (0.5)^{2/3} (0.0125)^{1/2} / 0.015 = 7.00$ FPS ✓

TIME = L/V = 40 / 7.00 = 5.7 SEC = 0.09 MIN ✓



SUMMARY OF PIPED SYSTEM

WATERSHED	C	T_c (MIN)	i (IN/HR)	A (AC)	RATIONAL Q (CFS)
A	0.53	8.4	6.8	4.9	17.7
B	0.46	5.0	7.6	7.3	22.2
C	0.37	7.1	7.1	14.2	37.3
	0.41[1]			26.4	77.18

SUMMARY OF TOTAL SYSTEM

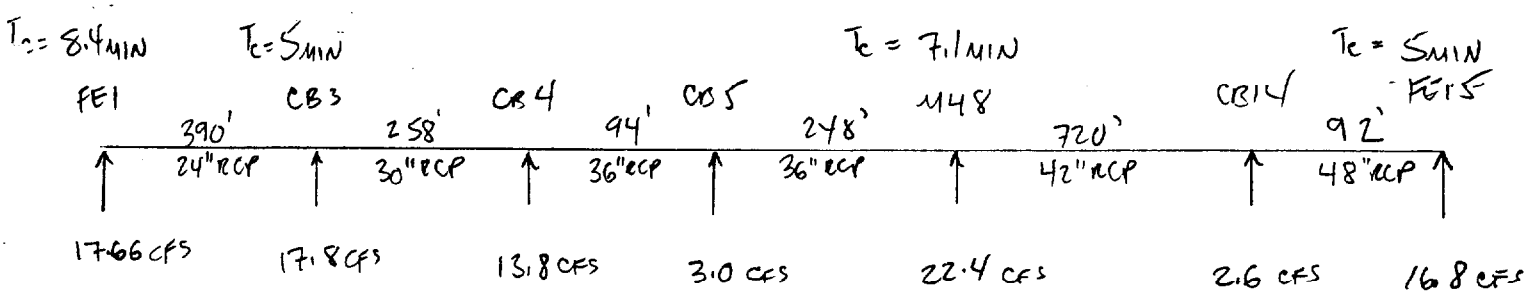
WATERSHED	C	T_c (MIN)	i (IN/HR)	A (AC)	RATIONAL Q (CFS)
A	0.53	8.4	6.8	4.9	17.7
B	0.46	5.0	7.6	7.3	22.2
C	0.37	7.1	7.1	14.2	37.3
D	0.60	5.0	7.6	3.7	16.8
	0.43[1]	—	—	30.1[2]	94.0[3]

1. WEIGHTED AVERAGE (BASED ON AREA / TOTAL AREA)
2. TOTAL
3. TOTAL FLOW REACHING SOUTHERN EDGE OF LANDFILL

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY [illegible] DATE 4-12-94 SUBJECT AREA 'A' LANDFILL DRAINAGE CALCULATIONS FINAL ANALYSIS SHEET NO. 24 OF 30

CHKD. BY G DATE 4-18-94 JOB NO.



Tc (FE1 - CB3)

$Q = AV = 17.66 = (24''\phi)(v)$ $v = 17.66/3.14 = 5.62$ FT/SEC

TIME = L/V = 390 FT / 5.62 FT/SEC = 69 SEC = 1.16 MIN

Tc (CB3 - CB4)

$Q = AV = 17.66 + 17.8 = (30''\phi)(v)$ $V = 35.46 / 4.91 = 7.22$ FPS

TIME = L/V = 258 FT / 7.22 = 35 SEC = 0.60 MIN

Tc (CB4 - CB5)

$Q = AV = 17.66 + 17.8 + 13.8 = (36''\phi)(v)$ $v = 49.26/7.07 = 6.97$

TIME = L/V = 94'/6.97 = 13.5 SEC = 0.22 MIN

Tc (CB5 - M48)

$Q = AV = 17.66 + 17.8 + 13.8 + 3 = (36''\phi)(v)$ $V = 52.3 / 7.01 = 7.4$ FPS

TIME = L/V = 248 FT / 7.4 = 33.5 SEC = 0.56 MIN

Tc (M48 - CB14)

$Q = AV = 52.3 + 22.4 = (42''\phi)(v)$ $V = 74.7 / 9.62 = 7.76$

TIME = L/V = 720 FT / 7.76 = 92.7 SEC = 1.55 MIN

Tc (CB14 - FE15)

$Q = AV = 74.7 + 2.6 = (48''\phi)(v)$ $V = 77.3 / 12.6 = 6.15$ FPS

TIME = L/V = 92' / 6.15 = 15 SEC = 0.25 MIN.

TIME FE1 - FE15 = 4.34 MIN

MAXIMUM TIME = 8.4 + 4.34 = 12.74 MIN → FROM RAINFALL CHART

WITH 12.74 MIN & 25 YR STORM ⇒ I = 6.0 IN/HR.

TOTAL FLOW OF WATERSHED REACHING STORM PIPE SYSTEM

$Q = CiA$

$C = 0.41$ (WEIGHTED AVERAGE)

$i = 6.0$ IN/HR (PREV. CALC)

$A = 26.41$ AC (TOTAL AREA OF WATERSHEDS A, B & C).

$Q = (0.41)(6.0)(26.41) = 64.97$ CFS

COMPARE WITH DESIGN FLOWS FOR SYSTEM

TOTAL DESIGN FLOWS = 77.18 (SUM OF FLOWS ENTERING STORM PIPE SYSTEM FROM WATERSHEDS A, B & C)

FACTOR OF SAFETY = DESIGN FLOW / ACTUAL FLOW

= 77.18 / 64.97 = 1.19

CONCLUSION:

FINAL ANALYSIS HAS SHOWN AN ADDITIONAL FACTOR OF SAFETY OF 19%. THIS MAKES FOR A MORE CONSERVATIVE DESIGN WHICH WILL PROVIDE FOR FUTURE IMPROVEMENTS WITHIN THE DRAINAGE AREAS UPSLOPE OF THE AREA "A" LANDFILL.

Figure 9-2 - Values of Runoff Coefficient (C) for Rational Formula

Land use	C
Business:	
Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi units, detached	0.40-0.60
Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.35
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
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

Land use	C
Lawns:	
Sandy soil, flat, 2%	0.05-0.10
Sandy soil, average, 2-7%	0.10-0.15
Sandy soil, steep, 7%	0.15-0.20
Heavy soil, flat, 2%	0.13-0.17
Heavy soil, average, 2-7%	0.18-0.22
Heavy soil, steep, 7 %	0.25-0.35
Agricultural land:	
Bare packed soil	
Smooth	0.30-0.60
Rough	0.20-0.50
Cultivated rows	
Heavy soil no crop	0.30-0.60
Heavy soil with crop	0.20-0.50
Sandy soil no crop	0.20-0.40
Sandy soil with crop	0.10-0.25
Pasture	
Heavy soil	0.15-0.45
Sandy soil	0.05-0.25
Woodlands	0.05-0.25

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: Virginia Erosion and Sediment Control Handbook, 1980. Virginia Soil and Water Conservation Commission.

From: CGSESC

SHEET 27 OF 30

TABLE XIV.—Values of Effective Absolute Roughness and Friction Formula Coefficients

Conduit Material	Effective Absolute Roughness (Darcy-Weisbach) k (ft)	Manning n (ft½)	Hazen-Williams C*
Closed conduits			
Asbestos-cement pipe	0.001–0.01	0.011–0.015	100–140
Brick	0.005–0.02	0.013–0.017	—
Cast iron pipe			
Uncoated (new)	0.00085	—	—
Asphalt dipped (new)	0.0004	—	—
Cement-lined & seal coated	0.001–0.01	0.011–0.015	100–140
Concrete (monolithic)			
Smooth forms	0.001–0.005	0.012–0.014	—
Rough forms	0.005–0.02	0.015–0.017	—
Concrete pipe	0.001–0.01	0.011–0.015	100–140
Corrugated-metal pipe (½-in. × 2⅔-in. corrugations)			
Plain	0.1 –0.2	0.022–0.026	—
Paved invert	0.03 –0.1	0.018–0.022	—
Spun asphalt lined	0.001–0.01	0.011–0.015	100–140
Plastic pipe (smooth)	0.01	0.011–0.015	100–140
Vitrified clay			
Pipes	0.001–0.01	0.011–0.015	100–140
Liner plates	0.005–0.01	0.013–0.017	—
Open channels			
Lined channels			
a. Asphalt	—	0.013–0.017	—
b. Brick	—	0.012–0.018	—
c. Concrete	0.001–0.03	0.011–0.020	—
d. Rubble or riprap	0.02	0.020–0.035	—
e. Vegetal	—	0.030–0.40 *	—
Excavated or dredged			
Earth, straight and uniform	0.01	0.020–0.030	—
Earth, winding, fairly uniform	—	0.025–0.040	—
Rock	—	0.030–0.045	—
Unmaintained	—	0.050–0.14	—
Natural channels (minor streams, top width at flood stage < 100 ft)	0.1 –3.0	—	—
Fairly regular section	—	0.03 –0.07	—
Irregular section with pools	—	0.04 –0.10	—

* Assume dimensional units contained in 1.32 term in formula. See References (2) (19) (20). (Varies with depth and velocity.)

the values obtained in laboratory tests with clear water and clean conduits.

The range in coefficients for a given pipe material is explained partially by the disturbing influences mentioned previously in the general discussion of coefficients. A coefficient which will yield higher friction losses should be selected for sewers with high disturbing influences.

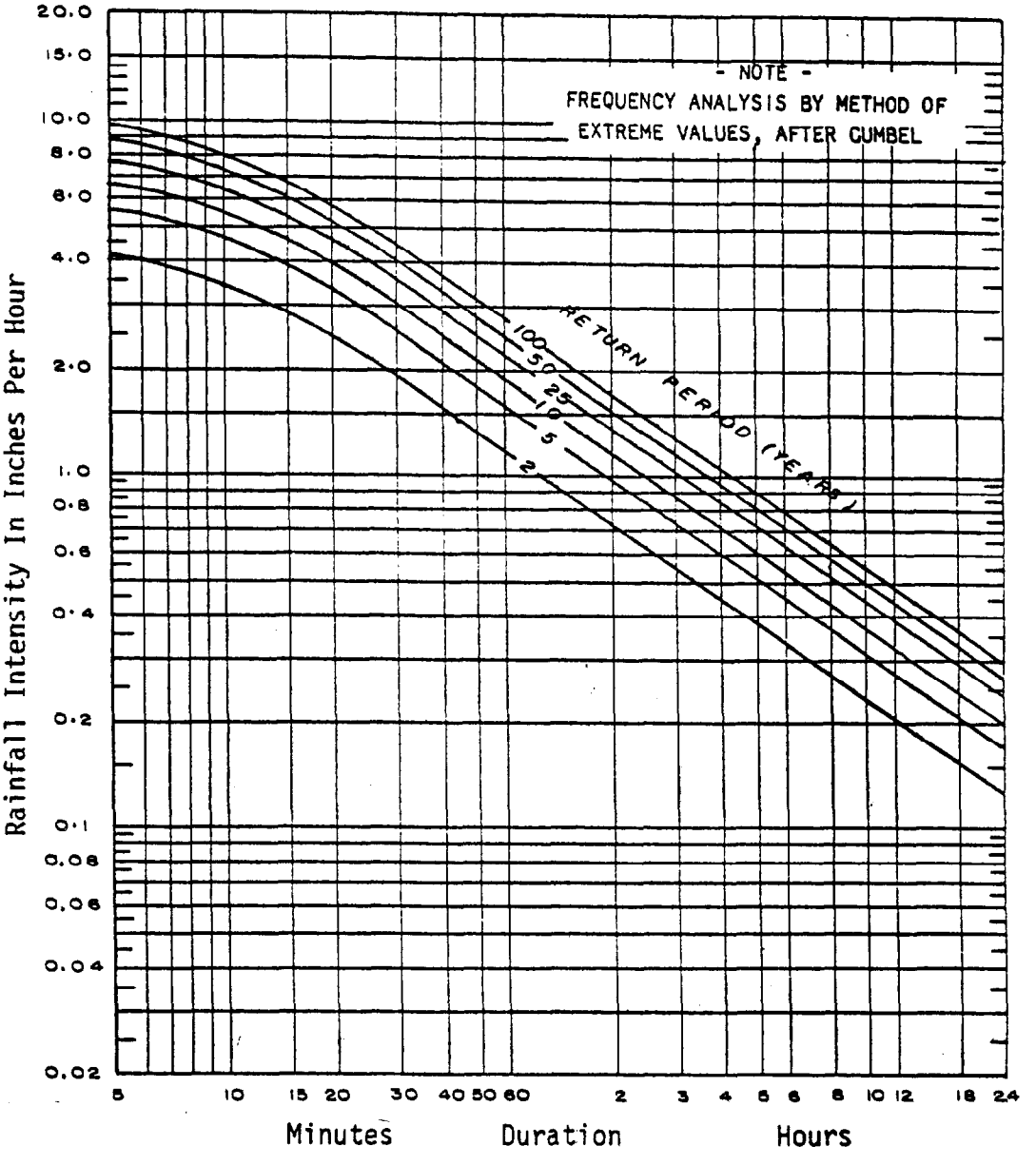
Because of the physical and hydraulic conditions which may influence a friction formula coefficient, the values given in Table XIV for one fric-

FROM. DESIGN & CONSTRUCTION OF SANITARY AND STORM SEWERS

SHEET 28 OF 30

Figure 9-4 - Rainfall Frequency-Intensity-Duration Chart

NEW HAVEN, CONNECTICUT
1905-1951



Source: Connecticut Department of Transportation, Wethersfield, Connecticut.

FROM: CGSESC

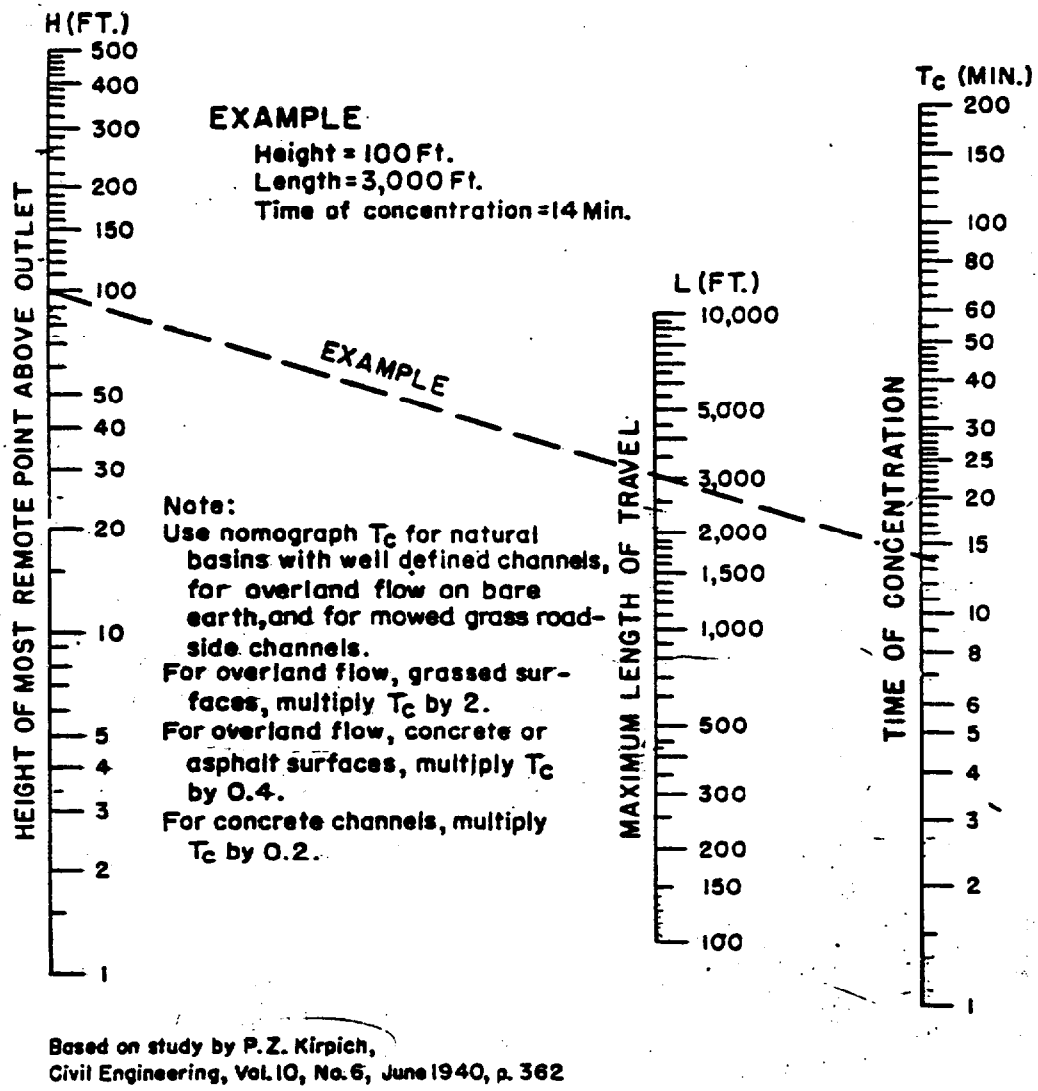


Figure P075. Time of Concentration of Small Drainage Basins

Source: Ref. 81

FROM: DRAINAGE MANUAL - CONNECTICUT DEPARTMENT OF TRANSPORTATION

— KIRPICH CHART —

STORM PIPE DESIGN

STMPIPE.WK3 Print Date: 13-Apr-94

G4-B-94

RAINFALL FLOWS

Watershed "A"	17.7 cfs
Watershed "B"	22.2 cfs
Watershed "C"	37.3 cfs
Watershed "D"	16.8 cfs

SEE SHEET 23 OF 30.

TOTAL 94.0 cfs

Q(in) @ FE1 =	17.7 cfs	100% of Watershed A
Q(in) @ CB3 =	17.8 cfs	80% of Watershed B
Q(in) @ CB4 =	4.4 cfs	20% of Watershed B
	9.3 cfs	25% of Watershed C
Q(in) @ CB5 =	3.0 cfs	8% of Watershed C
Q(in) @ MH8 =	22.4 cfs	60% of Watershed C
Q(in) @ CB14 =	2.6 cfs	7% of Watershed C
Q(in) @ Swale =	16.8 cfs	100% of Watershed D

TOTAL 94.0 cfs

TOTAL FLOWS

Q(in) @ FE1 =	17.7 cfs
Q(in) @ CB3 =	17.8 cfs
Q(in) @ CB4 =	13.8 cfs
Q(in) @ CB5 =	3.0 cfs
Q(in) @ MH8 =	22.4 cfs
Q(in) @ CB14 =	2.6 cfs
Q(in) @ Swale =	16.8 cfs
TOTAL	**94.0 cfs**

Trapezoidal Swale Dimensions

Depth	Width	Side Slope
3.5	4	2
Area	**Wet Perim**	**Hyd. Radius**
38.50	19.65	1.96

NOTE: Groundwater flow volumes not significant to design of storm pipe.

STORM SEWER SYSTEM

Segment	ID Structure inv out	ID Structure inv in	ID Structure Length (ft)	Slope	Pipe Size (in)	Pipe Type	Mannings n	Pipe Area (sf)	Hydraulic Radius (ft)	Mannings V(full) (fps)	Q(full) (A*V) (cfs)	Q(in)	Accum Q(in)	Q(full) minus Accum Q(in)
FE1	92.62	92.56	6	1.00%	24	RCP	0.013	3.14	0.50	7.20	22.62	17.70	17.70	4.9
FE1-MH2	92.56	91.04	152	1.00%	24	RCP	0.013	3.14	0.50	7.20	22.62		17.70	4.9
MH2-CB3	90.94	88.62	232	1.00%	24	RCP	0.013	3.14	0.50	7.20	22.62		17.70	4.9
CB3-CB4	88.12	84.89	258	1.25%	30	RCP	0.013	4.91	0.63	9.34	45.86	17.76	35.46	10.4
CB4-CB5	84.39	83.45	94	1.00%	36	RCP	0.013	7.07	0.75	9.44	66.70	13.77	49.23	17.5
CB5-MH6	83.35	81.67	168	1.00%	36	RCP	0.013	7.07	0.75	9.44	66.70	2.98	52.21	14.5
MH6-MH7	81.57	80.89	68	1.00%	36	RCP	0.013	7.07	0.75	9.44	66.70		52.21	14.5
MH7-MH8	80.79	80.64	12	1.25%	36	RCP	0.013	7.07	0.75	10.55	74.57		52.21	22.4
MH8-MH9	80.14	78.46	168	1.00%	42	RCP	0.013	9.62	0.88	10.46	100.61	22.38	74.59	26.0
MH9-MH10	78.36	78.20	16	1.00%	42	RCP	0.013	9.62	0.88	10.46	100.61		74.59	26.0
MH10-MH11	78.10	76.81	172	0.75%	42	RCP	0.013	9.62	0.88	9.06	87.13		74.59	12.5
MH11-MH12	76.71	75.66	140	0.75%	42	RCP	0.013	9.62	0.88	9.06	87.13		74.59	12.5
MH12-MH13	75.56	74.28	128	1.00%	42	RCP	0.013	9.62	0.88	10.46	100.61		74.59	26.0
MH13-CB14	74.18	73.22	96	1.00%	42	RCP	0.013	9.62	0.88	10.46	100.61		74.59	26.0
CB14-FE15	72.72	72.30	84	0.50%	48	RCP	0.013	12.57	1.00	8.08	101.57	2.61	77.20	24.4
FE15	72.30	72.26	8	0.50%	48	RCP	0.013	12.57	1.00	8.08	101.57		77.20	24.4
FE15-END	72.26	72.00	105	0.25%	See Swale	Riprap Swale	0.03	38.50	1.96	3.88	149.29	16.83	94.03	55.3

APPENDIX G

SETTLEMENT ANALYSIS AND BEARING CAPACITY ANALYSIS

I SETTLEMENT ANALYSIS - AREA 'A' LANDFILL

A. OBJECTIVE

- THE PROPOSED CAP for the AREA A landfill will RESULT IN APPROX. 18" of EARTH PLACED OVER EXISTING GRADES. PREVIOUS BORINGS CONDUCTED AT THE SITE indicate the PRESENCE OF A compressible STRATUM OF MARINE SILTS AND CLAYS AT depths APPROX. 18' below the ground surface (BLOW COUNTS = WOR → 1 /FT). BECAUSE the proposed landfill CAP will RESULT IN A SURCHARGE; SURFACE SETTLEMENT RESULTING FROM COMPRESSION OF THE SILT / CLAY layer is POSSIBLE. As the CAP INCORPORATES AN IMPERMEABLE geomembrane liner, EVALUATION OF THE IMPACT OF ESTIMATED SETTLEMENT ON the long-term serviceability OF the CAP SYSTEM IS NECESSARY. DUE TO the light SURCHARGE loads expected, AND the DEPTH OF THE compressible layer, TOTAL SETTLEMENT IS EXPECTED TO BE MINIMAL.

- TO EVALUATE POTENTIAL SETTLEMENT, UNDISTURBED SAMPLES OF THE SILTS AND CLAYS will BE OBTAINED FOR LABORATORY ANALYSIS. LAB ANALYSIS will CONSIST OF A CONSOLIDATION TEST IN ACCORDANCE WITH ASTM D2435-65T OR LATEST UPDATED VERSION.

- The loading sequence for the consolidation test is based on the following conditions:

BORING/WELL	GRD EL.	d to stratum (ft)	d: MID-PT (ft)	γ_{tot} (1)	GW-EL	d_1: GW (ft)	σ (2)	u (3)	$\bar{\sigma}$ (4)
ZL MW9S	85.3	12-28	20	115	76.0	10.7	2300	668	1632
ZL TB1	84.0	12-20	16	115	76.0	8	1840	500	1340
ZL MW7S	82.8	16-28	15	115	75.0	7.2	1725	450	1275
ZL MW17S	87.48	12-18	20	115	76.0	8.5	2300	531	1769
ZL MW18S	87.94	18-26	22	115	72.0	6.06	2530	378	2152
ZL TB6	88.4	13-17	15	115	78.0	4.6	1725	287	1438

(1) ASSUMED total unit weight of overburden soil = 115 #/ft³

(2) TOTAL STRESS @ DEPTH d = $(\gamma_{tot} \times d)$ psf

(3) PORE WATER PRESSURE @ DEPTH d = $(62.4\#/ft^3 \times d_1)$ psf

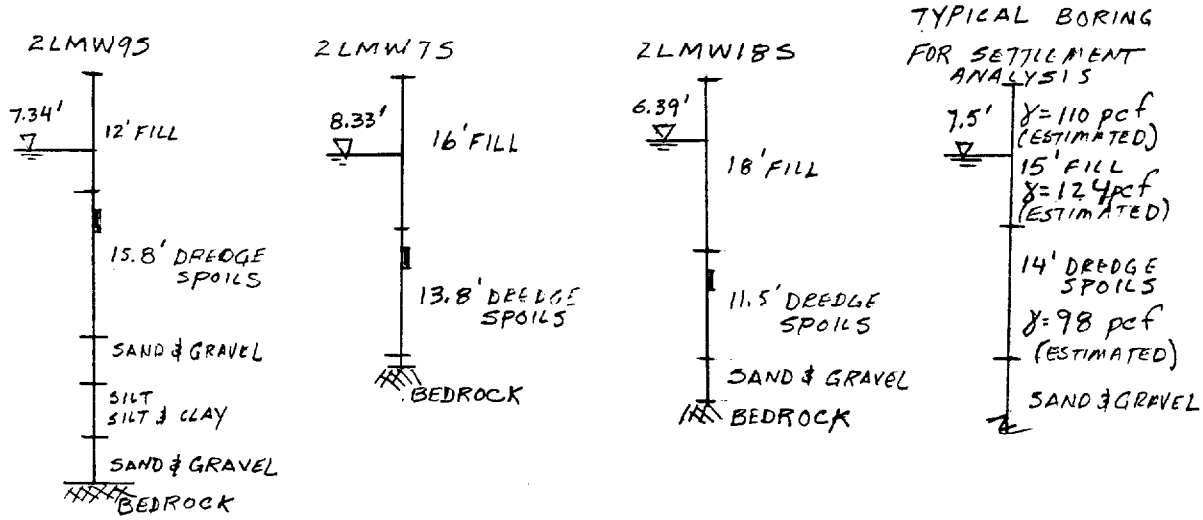
(4) INTERGRANULAR STRESS @ DEPTH d = $(\bar{\sigma} = \sigma - u)$ psf

INTERGRANULAR STRESS @ DEPTH d:

d	$\bar{\sigma}$ (psf)	$\bar{\sigma}$ (TSF)
20	1632	.816
16	1340	.67
15	1275	.638
20	1769	.885
22	2152	1.076
15	1438	.719

* FOR TEST USE load RANGE OF 1/4 TSF → 8 TSF

SOIL BORINGS



CONSOLIDATION TEST RESULTS SEE ATTACHED RESULTS 4-8-94 DR. CLARENCE WELTI, P.E., P.L.

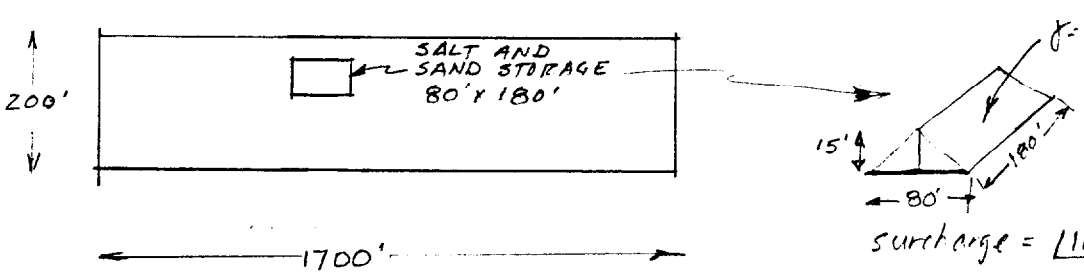
SAMPLE 2LGB1A (15-17') CONSOLIDATION = $0.112 \log \frac{P_F}{P_I}$ P_F = FINAL PRESSURE

$e_0 = 1.827$ (INITIAL VOID RATIO) P_I = INITIAL PRESSURE

SAMPLE 2LGB1C (22-24') CONSOLIDATION = $0.085 \log \frac{P_F}{P_I}$

$e_0 = 1.621$

DESIGN LOADS AT MID-DEPTH OF DREDGE SPOILS : D = 22'



VOL. = ½ (80)(15)(180) = 108,000 ft³

surcharge = $\frac{110 \text{ pcf} \; 108{,}000 \text{ cu ft}}{80' \times 180'}$ = 825 psf

SIMPLIFIED LANDFILL CONFIGURATION

DEAD LOAD DUE TO COVER (18" THICK COVER, γ = 150 pcf)

$\frac{18 \text{ inches}}{12 \text{ inches/foot}} \times 150 \frac{\text{lbs}}{\text{cu ft}} = 225 \frac{\text{lbs}}{\text{sq ft}}$

DEAD LOAD DUE TO SAND/SALT STORAGE $= q \frac{LB}{(L+Z)(B+Z)}$

q = SURCHARGE
L = LENGTH
B = WIDTH
Z = DEPTH

$\frac{825 \text{ psf} \; 80' \times 180'}{(80' + 22')(180' + 22')} = 576 \text{ lb/ft}^2$

@ center of dredge spoils layer

SETTLEMENT ANALYSIS

EXISTING σ @ 22'

$$\sigma = (7.5' \times 110\ pcf) + (7.5' \times 124\ pcf) + (7' \times 98\ pcf) = 2441\ psf$$

$$\mu = 14.5' \times 62.4\ pcf = 905\ psf$$

$$\sigma_0 = \sigma - \mu = 2441 - 905 = \underline{1536\ psf}$$

SETTLEMENT (FORMULAS FROM SPANGLER AND HANDY, SOIL ENGINEERING, 3rd EDITION)

$$e_F = e_0 - C_c \log \frac{P_F}{P_0}$$

e_f = VOID RATIO @ P_F
e_0 = VOID RATIO @ P_0
C_c = COMPRESSION INDEX

$$s = d \left[\frac{e_0 - e_f}{1 + e_0} \right]$$

s = SETTLEMENT
d = THICKNESS OF COMPRESSIBLE LAYER

DUE TO COVER

$$e_F = 1.827 - 0.112 \log \frac{1761}{1536} = 1.820$$

$$s = 14' \left[\frac{1.827 - 1.820}{2.827} \right] = 0.03' = 0.41''$$

DUE TO SAND/SALT STORAGE AND COVER

$$e_f = 1.827 - 0.112 \log \frac{2337}{1536} = 1.806$$

$$s = 14' \left[\frac{1.827 - 1.806}{2.827} \right] = 0.104' = 1.25''$$

DIFFERENTIAL SETTLEMENT = 1.25" - 0.41" = 0.84"

CONCLUSION: This amount of settlement should have minimal impact to the proposed cap. For the proposed design SETTLEMENT IS NOT A CONCERN.

DR. CLARENCE WELTI, P.E., P.C.

GEOTECHNICAL ENGINEERING

227 Williams Street • P.O. Box 397
Glastonbury, CT 06033

(203) 633-4623 / FAX (203) 657-2514

RECEIVED
APR 11 1994
ATLANTIC ENVIRONMENTAL SERVICES, INC.

April 8, 1994

Atlantic Environmental Services Inc.
188 Norwich Avenue; P.O. Box 297
Colchester, CT 06415

Re: Consolidation Testing at U.S. Submarine Base Groton, Connecticut (Project No. 1256-31-03)

Herewith are results of two consolidation tests on samples 2LGB-1C at 22.0'-24.0' and 2LG-B1A at 15.0'-17.0'. The third specimen did not have sufficient cohesive material to perform a consolidation test. There was wood and large size aggregate mixed with the organic material on both ends of the specimen.

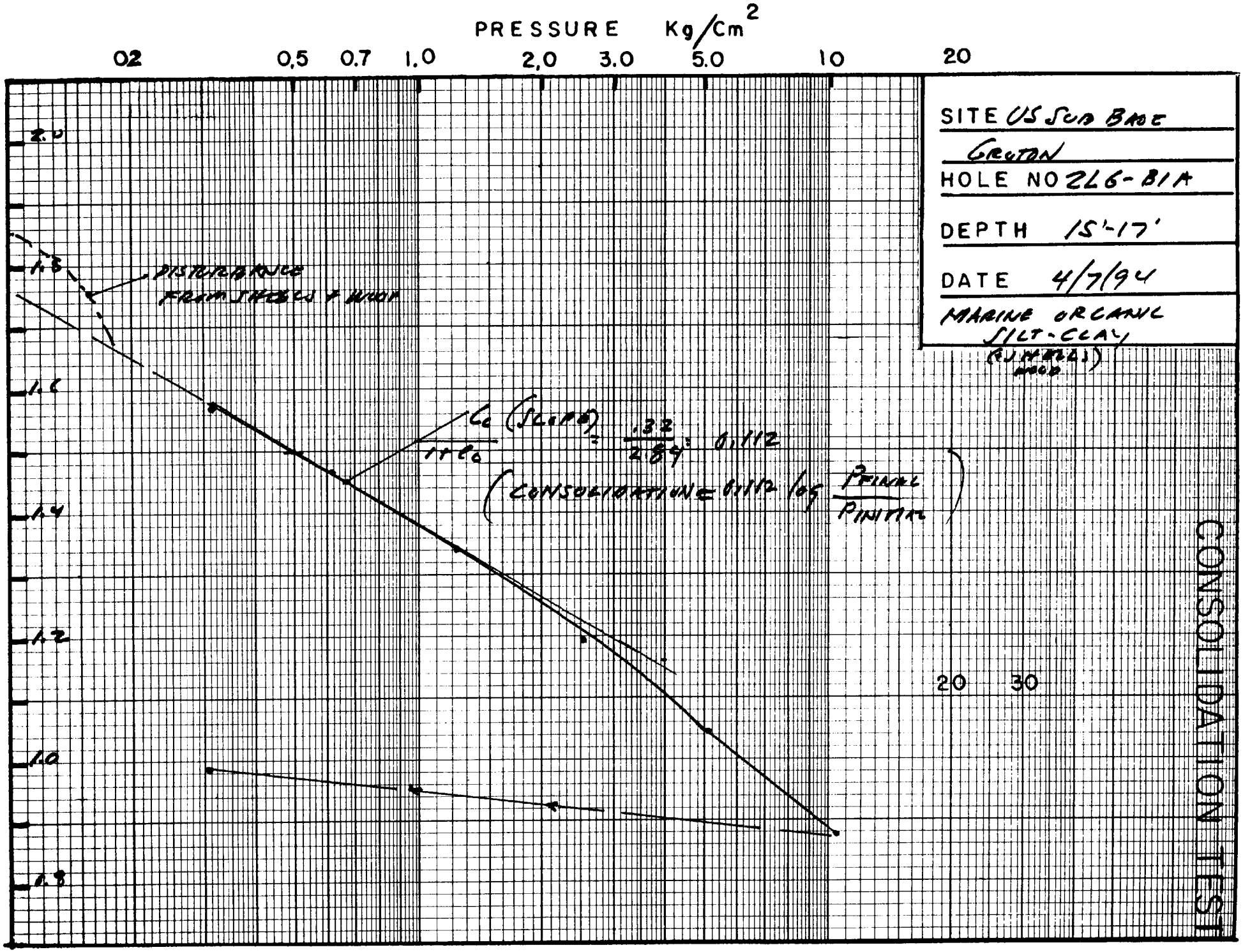
The samples indicate Cc/1+e value of 0.085 and 0.112, respectively. The Cv value appears to be about 0.14 square feet/day.

Please call me if you have any questions.

Very truly yours,

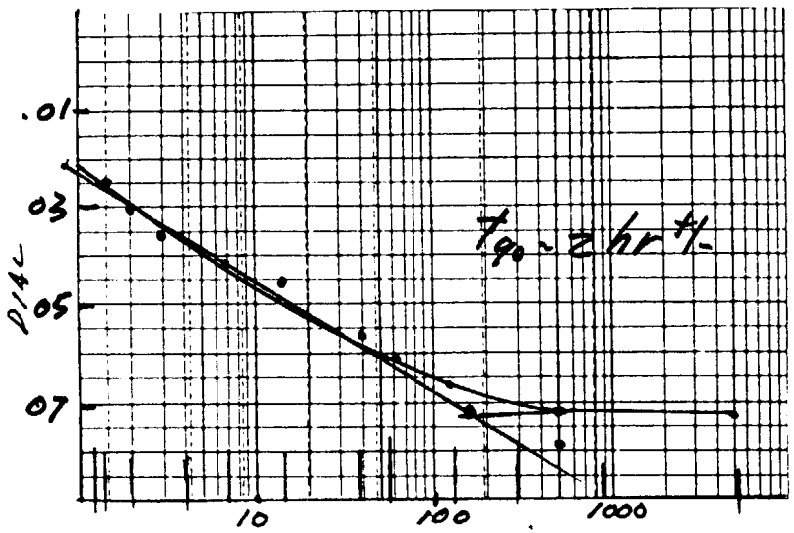
Clarence Welti, PhD, P.E.
President, Dr. Clarence Welti, P.E.,P.C.

CW:nl
Encs.


CONSOLIDATION TEST
PRESSURE Kg/Cm²
0.2
0.5
0.7
1.0
2.0
3.0
5.0
10
20
30
2.0
1.8
1.6
1.4
1.2
1.0
0.8
SITE US SUB BASE GROTON
HOLE NO 2L6-B1A
DEPTH 15'-17'
DATE 4/7/94
MARINE ORGANIC SILT-CLAY (SUMMERS) MOD
DISTURBANCE FROM STRESS + WATER
Cc (SLOPE) = .32 / 2.84 = 0.112
(CONSOLIDATION = 0.112 log Pfinal / Pinitial)

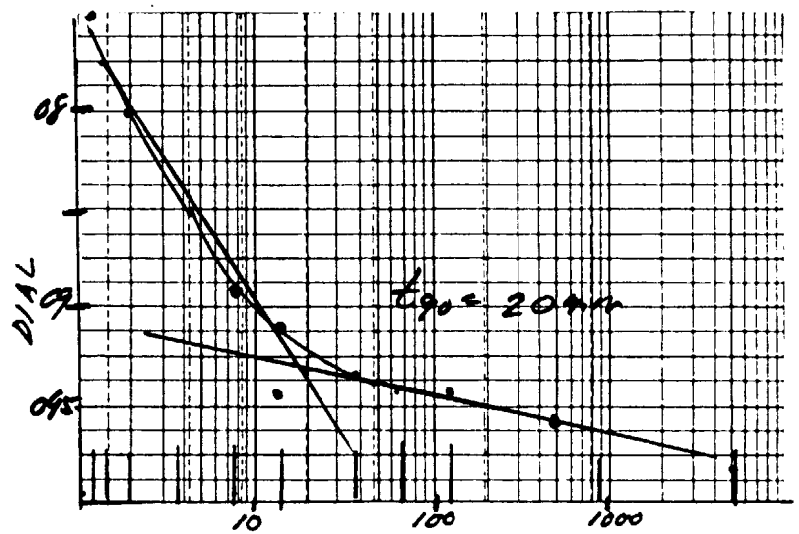
2L6-B1A

0.315 TSF



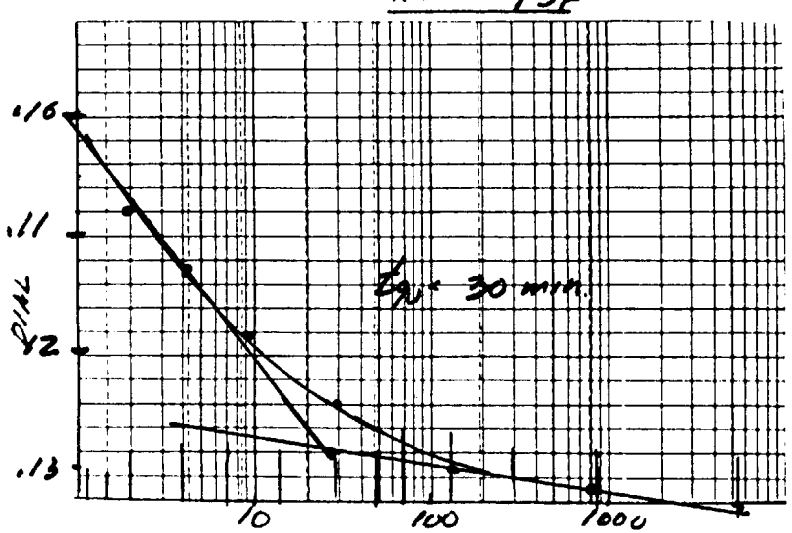
TIME IN MINUTES

0.63 TSF



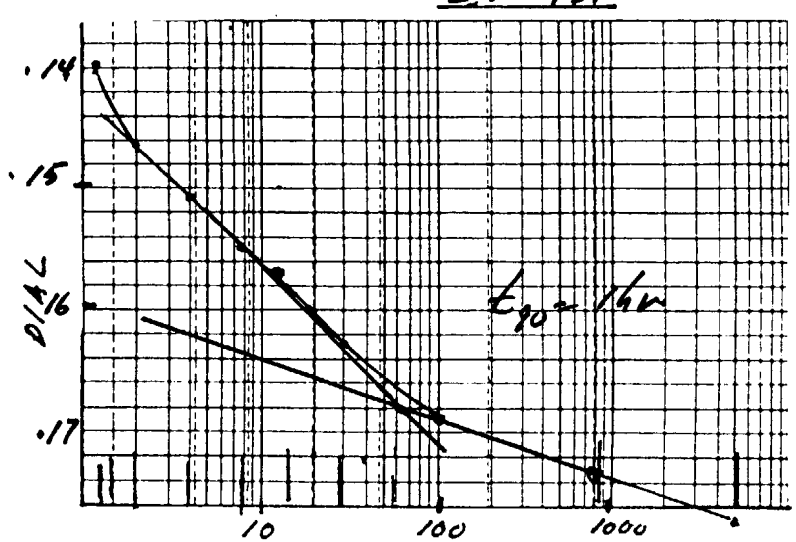
TIME IN MINUTES

1.25 TSF



TIME IN MINUTES

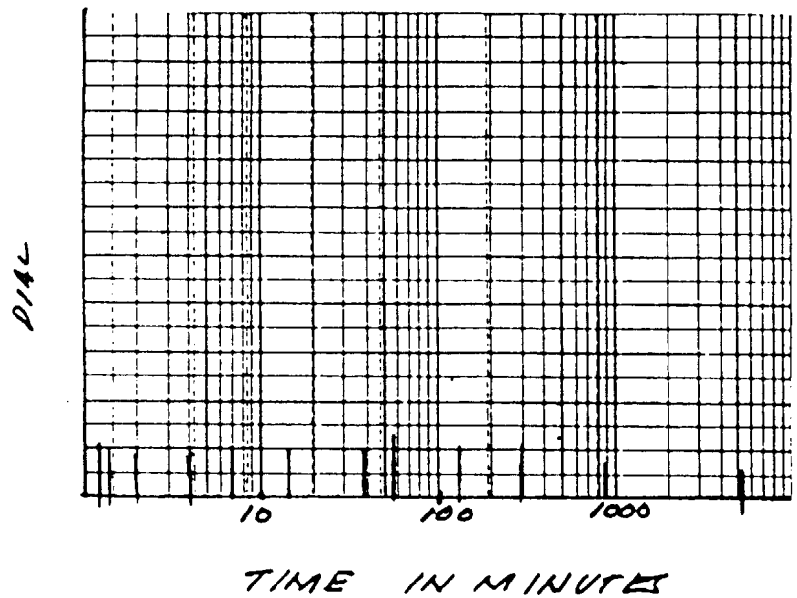
2.52 TSF



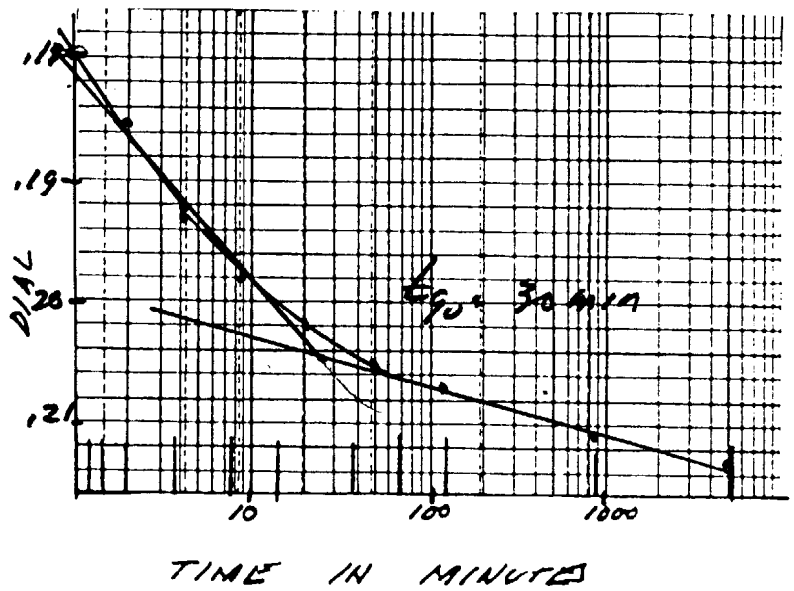
TIME IN MINUTES

2L6-151A

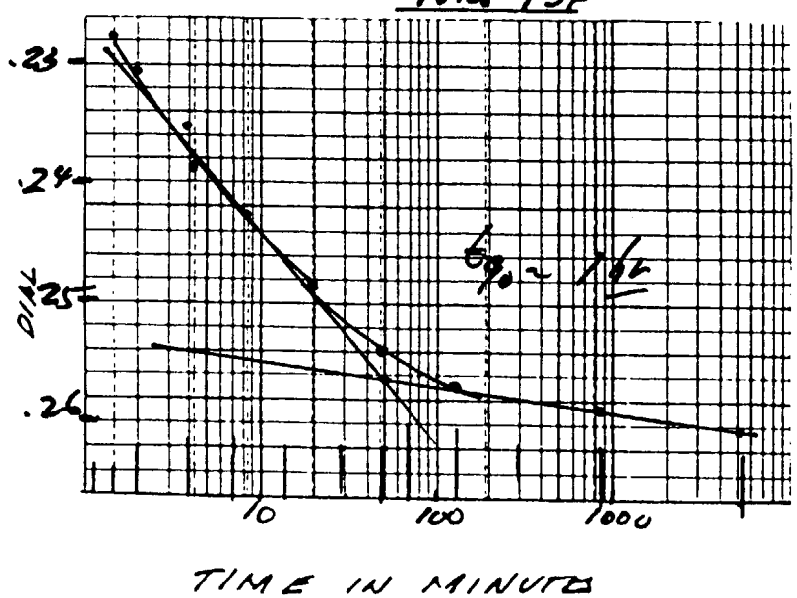
[redacted] TSF



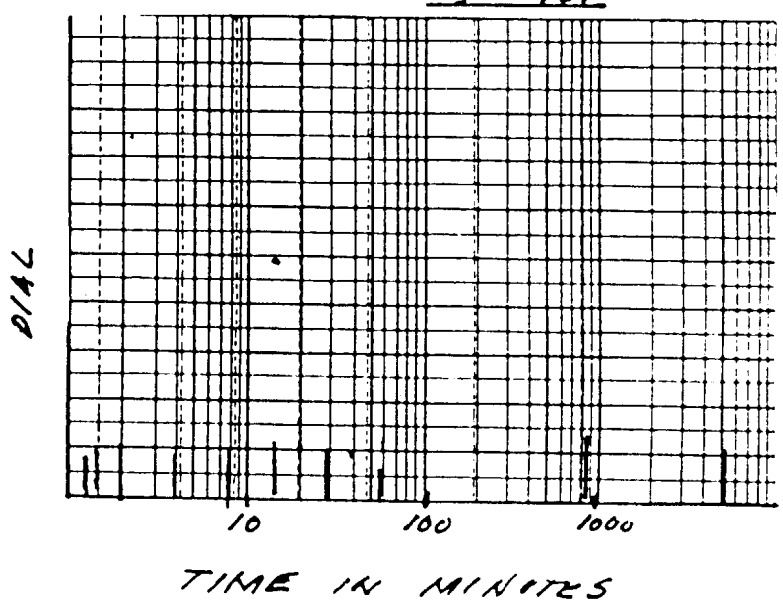
5.04 TSF



10.08 TSF



TSC



DR. CLARENCE WELTI, PE, PC
GLASTONBURY, CONNECTICUT 06033 • (203) 633-4623

CLIENT: Atlantic Environmental
PROJECT: US Sub Base
SUBJECT: Consolidation Test
BY: CW DATE: 4/7/95 SHEET NO. 1

ZL6-B1A @ 15'-17' - MARINE ORGANIC SILT CLAY

RING SIZE = 2.5" x 2cm H (=0.79")

VOL RING = 66.31 cc

SPG = 2.60 +/-

WT OF SOIL DRY = 60.3 grams

VOL OF SOIL = 60.3 ÷ 2.6 = 23.19 cc

VOL OF WATER = 66.31 - 23.19 = 43.12 cc = 43.12 grams

W/C START = 43.12 ÷ 60.3 = 71.5% +/-

$H_0 = \frac{23.19}{66.31} \times 0.79 = .276"$

$e_0 = \frac{\text{VOL VOIDS}}{\text{VOL SOLIDS}} = \frac{43.12}{23.19} = 1.827$

Ln	PRESSURE	R	R/H₀ = ΔE	e (e₀ = 1.827)
1 kg	0.315 TSF	.0725"	.263	1.564
2	0.630 TSF	.0995"	.360	1.467
4	1.260 "	.1347	.488	1.339
8	2.52 "	.1747	.632	1.195
16	5.04 "	.2148	.778	1.049
32	10.08 "	.2610	.945	.881
1 kg	0.315	.2820	.841	.986

TIME FACTOR (90%) = 0.9 +/-

$T = \frac{C_v t}{H^2}$ H = 0.79" = .

$0.90 = \frac{C_v \times 40}{(.79/12)^2} = \frac{40\,C_v}{.0043}$

t AVERAGE 90% = 40 min

$C_v = \frac{.0043 \times 0.90}{40} = .0000975$ ft²/min

= .14 ft²/day

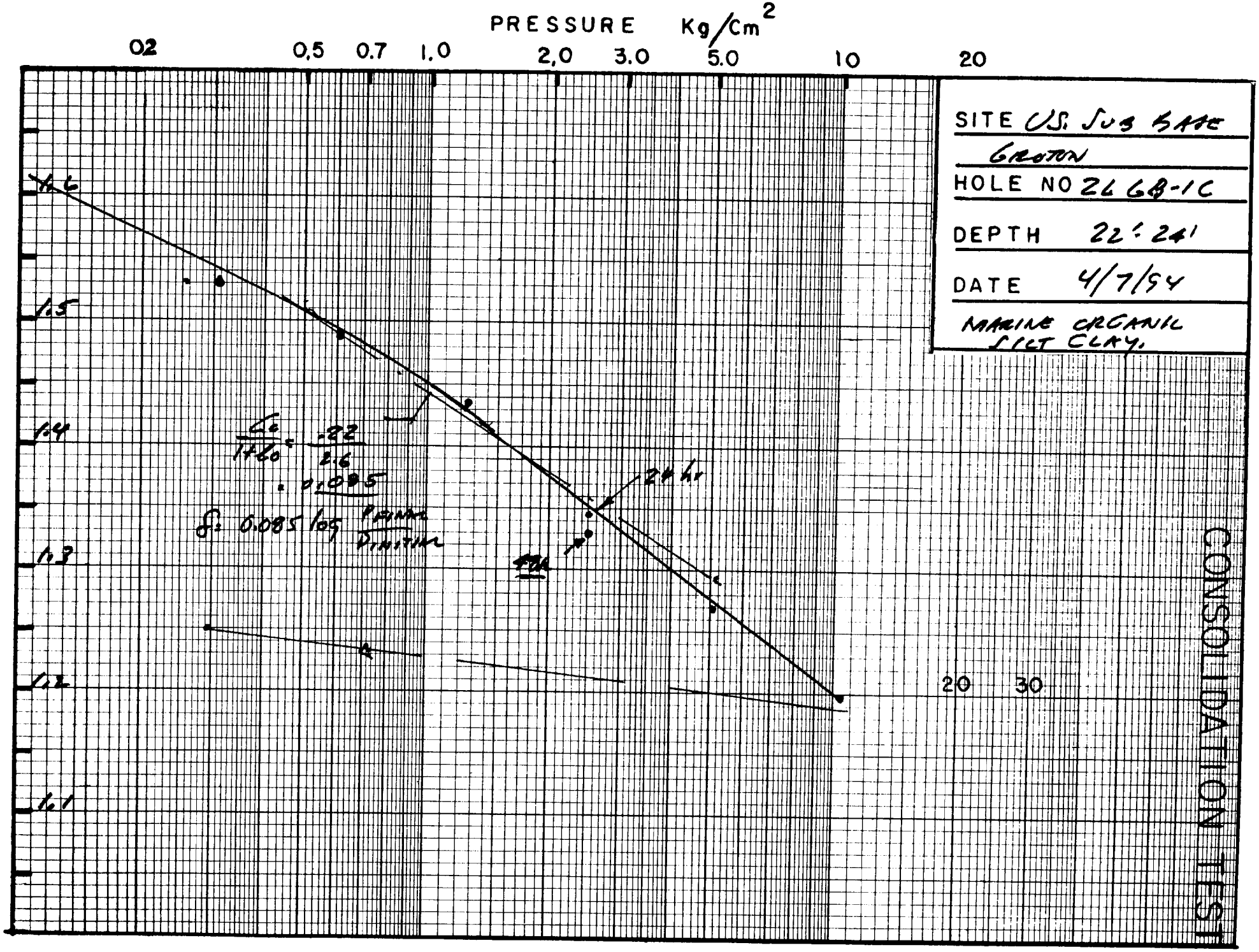
DR. CLARENCE WELTI, PE, PC
GLASTONBURY, CONNECTICUT 06033 • (203) 633-4623

CLIENT Atlantic Environmental
PROJECT U.S. Sub Base
SUBJECT Consolidation Test.
BY CW DATE 4/2/95 SHEET NO. 2

FOR 30' Kick layer @ 90%

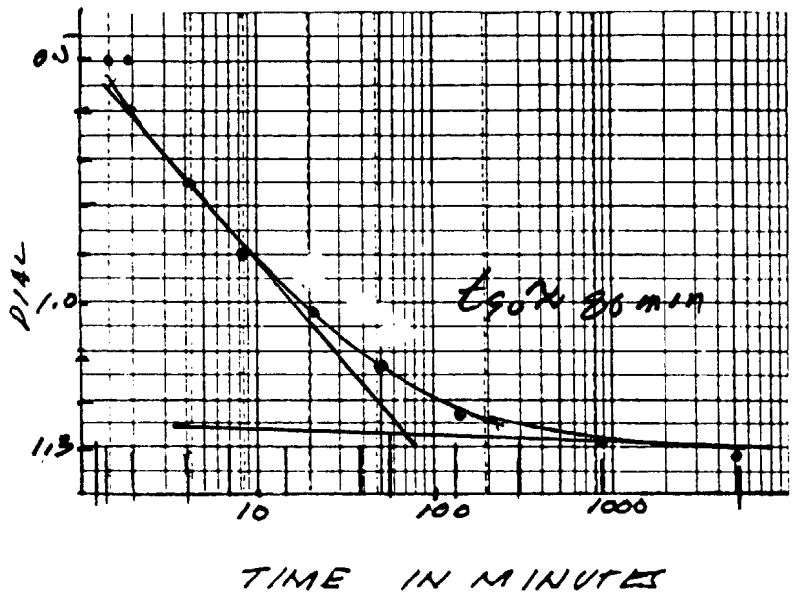
$$t_{90} = \frac{H^2 \times T}{C_v} = \frac{30^2 \times 0.9}{.14} = 5785 \text{ DAYS}$$

$$t_{50} = \frac{30^2 \times 0.2}{.14} = 1285 \text{ DAYS}$$

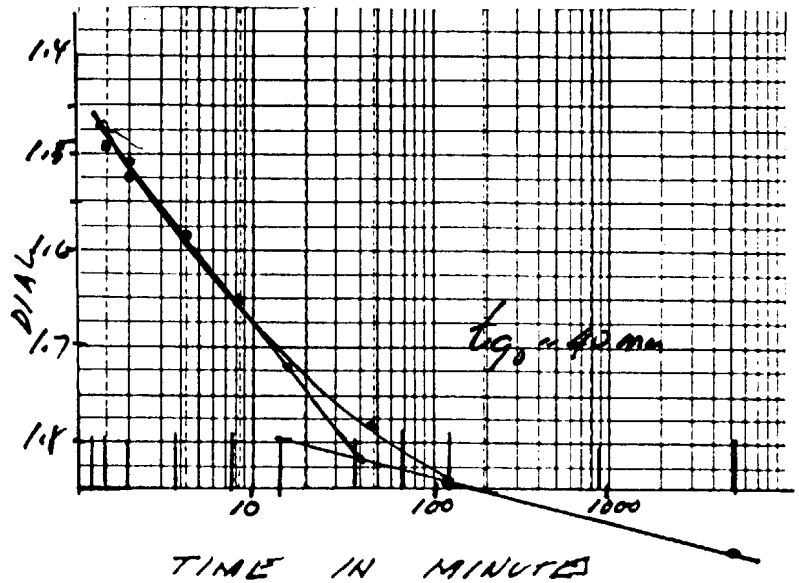

PRESSURE Kg/Cm²
0.2
0.5
0.7
1.0
2.0
3.0
5.0
10
20
1.6
1.5
1.4
1.3
1.2
1.1
20
30
24 hr
SITE U.S. SUB BASE GROTON
HOLE NO 26 GB-1C
DEPTH 22'-24'
DATE 4/7/54
MARINE ORGANIC SILT CLAY.
CONSOLIDATION TEST

226B-1C

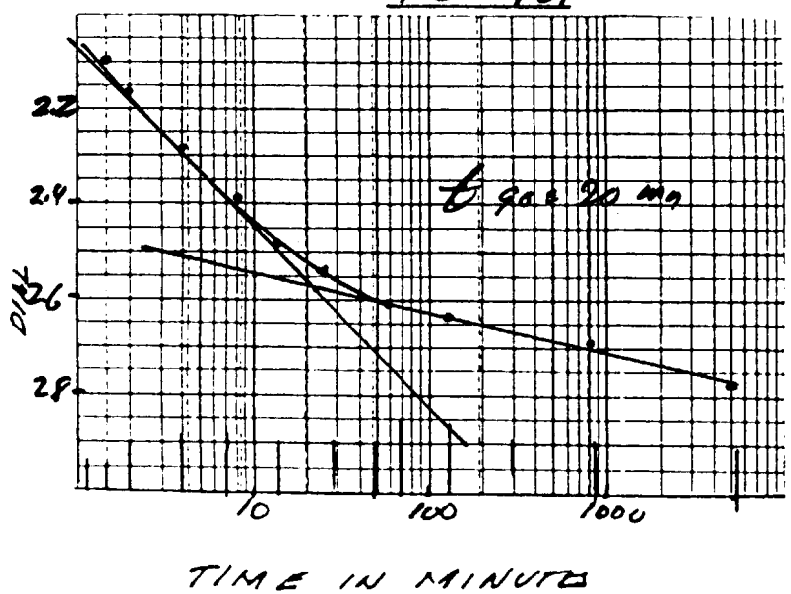
0.315 TSF



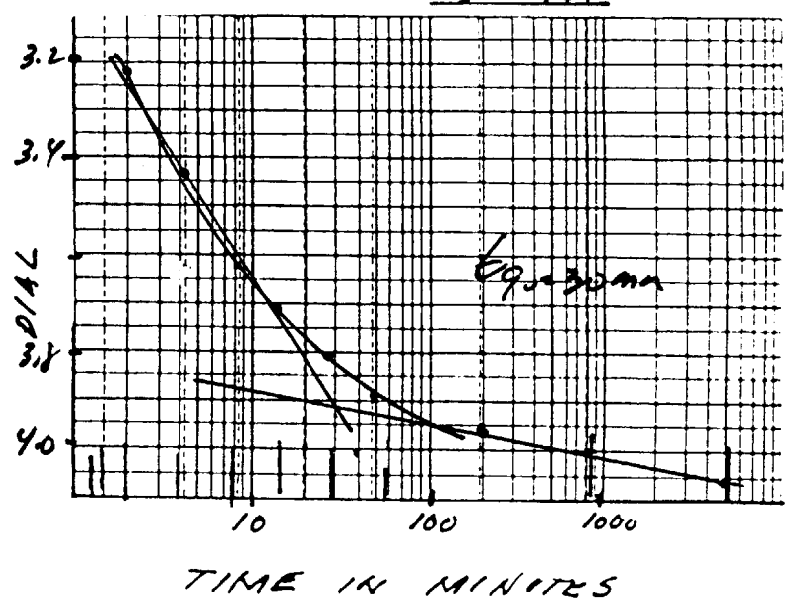
0.63 TSF



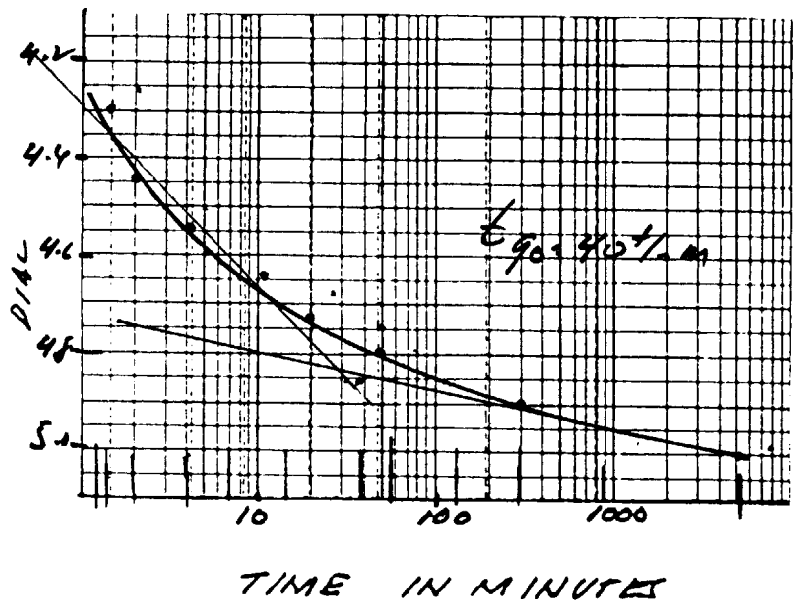
1.26 TSF



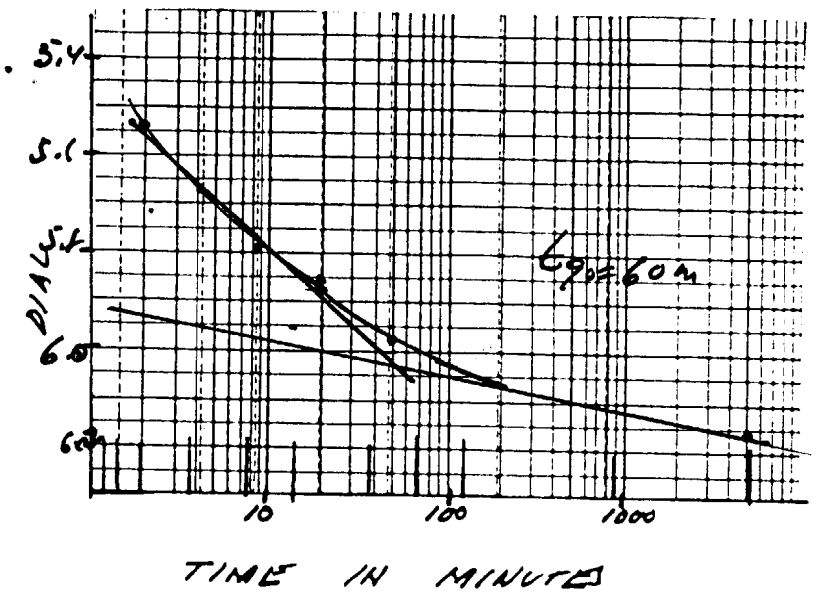
2.52 TSF



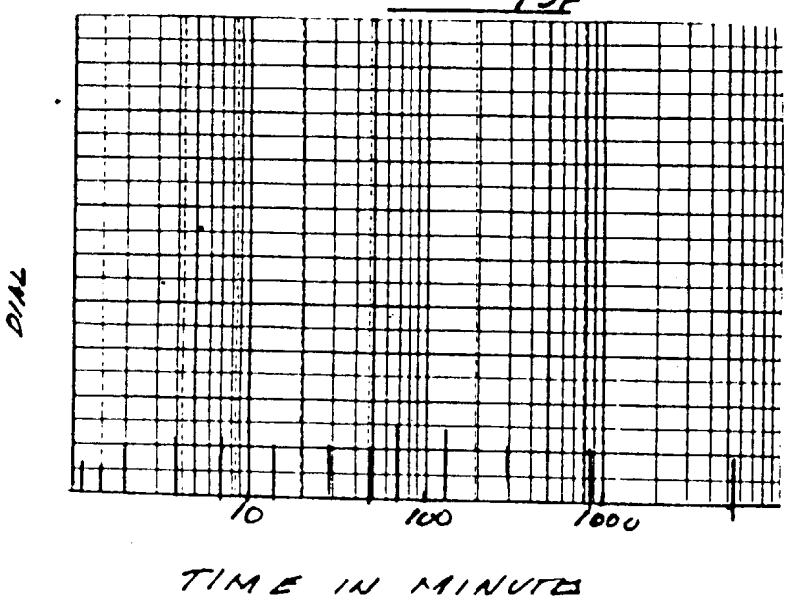
5.04 TSF



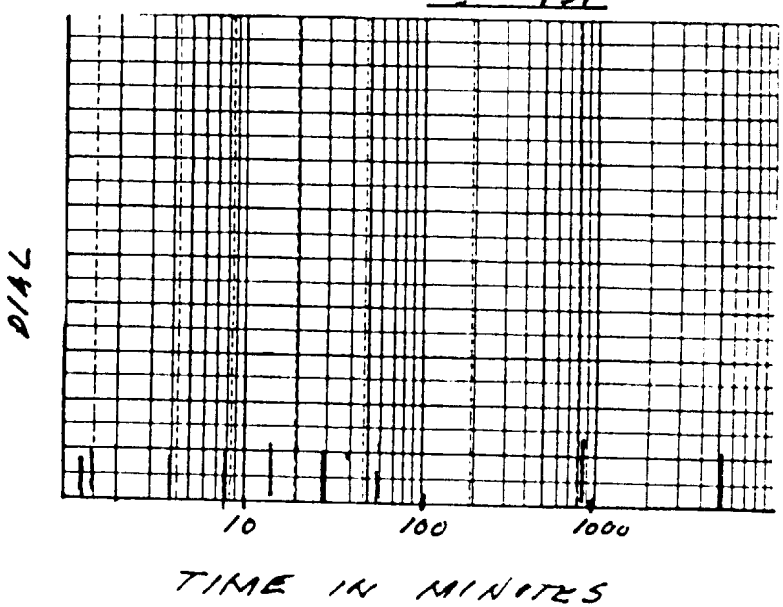
10.08 TSF



TSF



TSC



DR. CLARENCE WELTI, PE, PC
GLASTONBURY, CONNECTICUT 06033 • (203) 633-4623

CLIENT Atlantic Environmental
PROJECT U.S. Sebago.
SUBJECT Consolidation Test
BY CW DATE 4/7/94 SHEET NO. 1

2LGB-1C -22ft-24'

D = 2.5" H = 2 cm

VOL. RING ~ 66.31

SPG = 2.60 ±

WT OF SOIL DRY - 65.8 gram

VOL SOIL = 25.30 cc

VOL VOIDS = 41.00 cc

$h_s = \frac{25.30}{66.31} \times 2 = 1.456$ cm 14.56 mm

$e_0 = \frac{V_v}{V_s} = \frac{41.00}{25.30} = 1.621$

W/C SMPT = 62.3 %

LO	PRESSURE	R	R/h₀ = ε	e	e₀ = 1.621
1 kg	~~0.316 TSF~~	1.314 mm	.090	1.53	
2	.63 TSF	1.950	.133	1.488	
4	1.26	2.738	.188	1.433	
8	2.52	4.264	.293	1.328	
16	5.04	5.054	.349	1.272	
32	10.08	6.180	.423	1.198	
1	.315	5.422	.372	1.249	

Remark t_{90}: average 40 min

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY BCG DATE 4-14-94 SUBJECT AREA A LANDFILL SHEET NO. 1 OF 2
CHKD. BY CB DATE 5-1-94 DREDGE SPOIL BEARING CAPACITY JOB NO. 1256-31-05-04

BEARING STRENGTH: LOCAL SHEAR AT SILT/CLAY STRATUM DUE TO CRANE LOADING;
WORST CASE CRANE LOAD AT DREDGE SPOIL INTERFACE = 3.8 psi
(SEE PAGES 4-6 OF AREA A LANDFILL SURFICIAL CAP DESIGN)
(LOADING CONDITIONS FOR DERIVATION OF WORST CASE LOAD)
STRESS DUE TO SOIL OVERBURDEN = (7.5'x110pcf) + (7.5'x 125pcf) − (7.5'x62.4pcf) = 1294 pcf = 8.99 psi
DEAD LOAD DUE TO COVER = 225 pcf = 1.56 psi
TOTAL STRESS = 1.56 psi + 3.8 psi + 8.99 psi = 14.35 psi
CONVERTING TO TSF: $14.35 \frac{lbs.}{in^2} \frac{ton}{2000 lbs.} \frac{144 in^2}{ft^2} = 1.03$ TSF

725.2.3. TABLE 7-1. Allowable Bearing Pressures of Foundation Materials

Class of Material	*Allowable Bearing Pressure in Tons per Square Foot**
1. Massive igneous rocks and conglomerate, all in sound condition (sound condition allows minor cracks)	100
2. Slate in sound condition (minor cracks allowed)	50
3. Shale in sound condition (minor cracks allowed)	10 ‡
4. Residual deposits of shattered or broken bedrock of any kind except shale	10
5. Glacial Till	10
6. Gravel, well-graded sand and gravel	5
7. Coarse sand	3
8. Medium sand	2
9. Fine sand	1 to 2 ‡
10. Hard clay	5
11. Medium clay	2 †
12. Soft clay	1 †
13. Inorganic silt, shattered shale, or any natural deposit of unusual character not provided for herein	‡
14. Compacted granular fill	2 to 5 ‡
15. Preloaded materials	‡

*The allowable bearing pressure given in this section, or when determined in accordance with the provisions of section 727 will assure that the soils will be stressed within limits that lie safely below their strength. However, such allowable bearing pressure for Classes 9 to 12, inclusive, do not assure that the settlements will be within the tolerable limits for a given structure.

†Alternatively, the allowable bearing pressure shall be computed from the unconfined compressive strength of undisturbed samples, and shall be taken as 1.50 times that strength for round and square footings, and 1.25 times that strength for footings with length width ratios of greater than four (4); for intermediate ratios interpolation may be used.

‡Value to be fixed by the building official in accordance with sections 726.0. and 727.0.

SOIL ENGINEERING, 3RD EDITION, SPANGLER AND HARDY

ATLANTIC ENVIRONMENTAL SERVICES, INC. COLCHESTER, CT

BY BLG DATE 4-14-94 SUBJECT AREA A LANDFILL SHEET NO. 2 OF 2

CHKD. BY G DATE 5-1-94 DREDGE SPOIL BEARING CAPACITY JOB NO. 1256-31-05-04

BEARING CAPACITY FOR SOFT CLAY IS 1 TSF WHICH IS ≅ 1.03 TSF ∴ THE DREDGE SPOILS HAVE SUFFICIENT BEARING CAPACITY TO SUPPORT WORST CASE CRANE LOADS. THIS IS A CONSERVATIVE ESTIMATE AS THE ALLOWABLE PRESSURES IN TABLE 7-1 DO NOT INCLUDE BEARING CAPACITY DERIVED FROM CONFINING STRESS.

APPENDIX H

METHANE GAS SURVEY RESULTS

Methane Survey Results

Area A Landfill

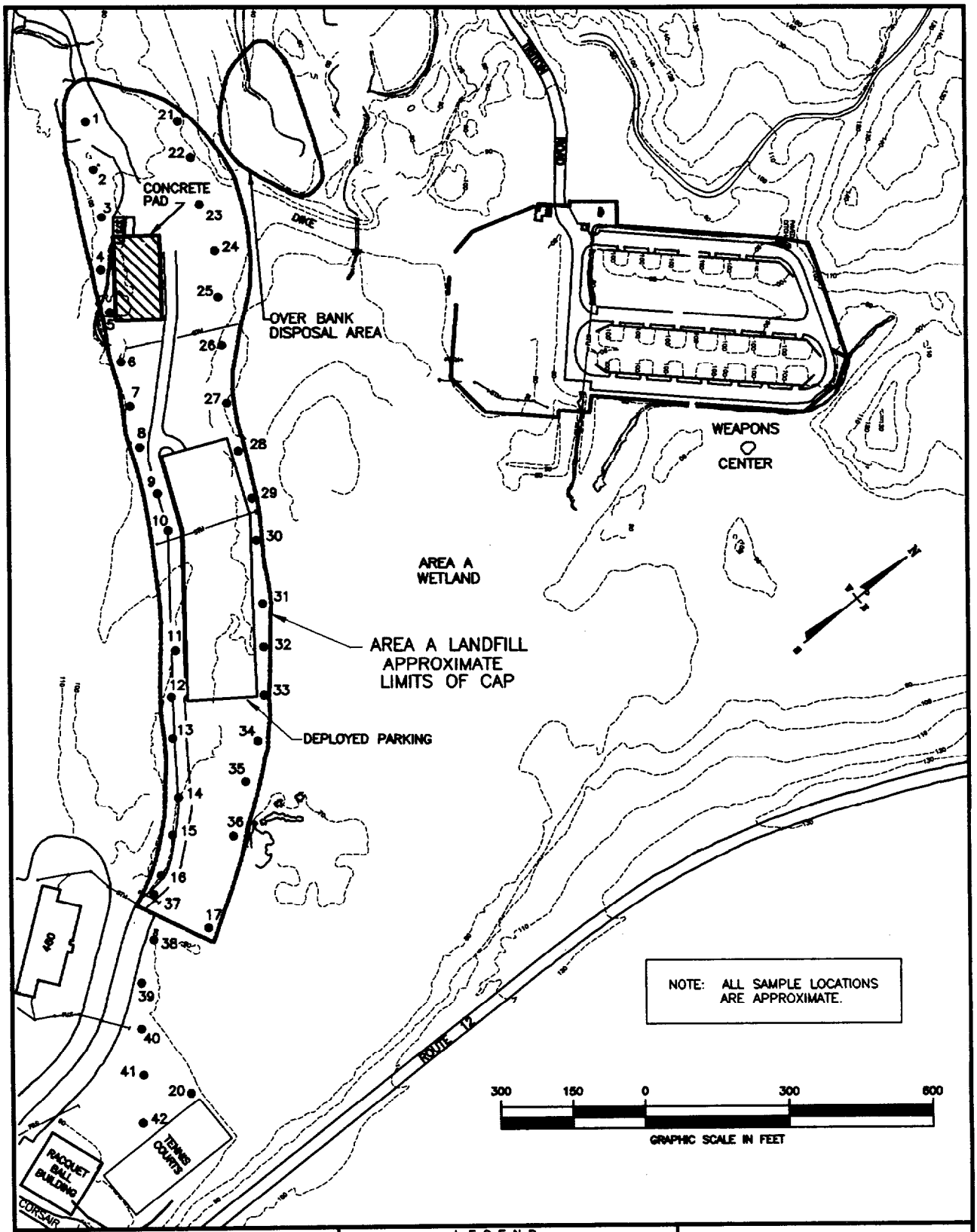
Sample Id.	Result (ppm)	Sample Id.	Result (ppm)	Sample Id.	Result (ppm)	Sample Id.	Result (ppm)
1	ND	14	5,600	27	ND	41	ND
2	ND	15	100	28	ND	42	ND
3	ND	16**	not sampled	29	ND		
4	ND	17	37	30	ND		
5	ND	18	not sampled	31	ND		
6	ND	19**	not sampled	32	1,300		
7	ND	20	ND	33	1,000		
8	ND	21	ND	34	30,000		
9	ND	22	ND	35	ND		
10*	not sampled	23	ND	36*	not sampled		
11	ND	24	ND	37	ND		
12	ND	25	ND	39	ND		
13	90,000	26	ND	40	ND		

* not sampled due to refusal.

** not sampled due to groundwater at or near surface.

NOTES:

1) LEL (Lower Explosive Limit) for methane 5,000 ppm; 25 % of LEL = 1,250 ppm
2) Methane measurements made with an organic vapor analyzer with a flame ionization detector. Soil gas samples collected in accordance with Atlantic SOP 1053.



INSTALLATION RESTORATION STUDY
NAVAL SUBMARINE BASE–NEW LONDON
GROTON, CONN.

LEGEND

1 ● SOIL GAS SURVEY SAMPLE LOCATION

----10---- EXIST CONTOUR
123 BUILDING No.
WATERCOURSE
—STM— STORM SEWER
CATCH BASIN

AREA A LANDFILL
SOIL GAS SURVEY
SAMPLE LOCATIONS

ATLANTIC ENVIRONMENTAL SERVICES, INC.

APPENDIX I

ENVIRONMENTAL PERMIT CHECKLIST

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
HAZARDOUS WASTE			
1. Does the project involve a hazardous waste transfer or storage facility?	X		Limited excavated of soils to be disposed of at a RCRA permitted landfill
2. If yes, will the waste be stored longer than 90 days?		X	
3. Is the required containment provided for spills?	X		
4. Are incompatibles stored separately ?	X		No anticipated incompatible waste streams
5. Is the floor sloped to allow spill collection or, alternatively, are containers elevated to prevent contact with spills ?	X		Soils will be stored in containers or stockpiled on an an impervious liner and covered.
6. Is the truck loading apron bermed to collect spills?	X		
7. Is the facility at least 50 feet from the property line?	X		Separation distance not required as waste are not ignitable or reactive
8. Is a construction permit required ?		X	CERCLA exemption
9. Is an operating permit required ?		X	CERCLA exemption
UNDERGROUND STORAGE TANKS			
10. Does the project involve underground storage of regulated substances ?		X	
11. Will there be closure or removal of an UST ?		X	
12. Will there be installation or modification of a UST?		X	
13. Is a construction permit required ?		X	CERCLA exemption
14. Is notificaiton or registration required ?		N/A	
15. Is leak detection provided ?		N/A	
16. Is the UST used to store heating oil only for consumptive use on the premises ?		N/A	
17. Are regulatory design criteria met ?		N/A	
HAZARDOUS WASTE TANKS			
18. Does the project involve either under or aboveground storage of hazardous waste in tanks ?		X	
19. Will the hazardous waste be stored longer than 90 days?		X	
20. Is a construction permit required ?		X	CERCLA exemption
21. Is an operating permit required ?		X	CERCLA exemption
22. Is notification required ?		X	
23. Is leak detection provided ?		N/A	
24. Is the tank double walled ?		N/A	
25. Is the tank compatible with what will be stored?		N/A	
26. Is a RCRA permit required ?		N/A	
AIR PERMITS			
27. Does the project involve an air pollution source?		X	
28. Compare air source emission with state allowable emissions standards and determine if registration with the state is required.		N/A	
29. Will boilers be installed or modified ?		X	
30. Will a painting/blasting facility be installed or modified?		X	

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
AIR PERMITS (continued)			
31. Are regulated operations or sources such as boilers, incinerators, petroleum storage tanks, fire-fighting training, munition disposal by burning, plating, sand-blasting, rocket and jet engine testing, asbestos application by spraying, fuel transfer, or painting be involved ?		X	
32. Are there other potential air sources ?	X		Potential odors, fugitive dust from excavation activity
33. Is a construction permit required ?		X	CERCLA exemption
34. Are souces rated at over 100 million BTU per hour?		N/A	
35. Are other permits required ?		X	
36. Are emission controls provided (Particulate, SOX, VOX, etc.) ?		N/A	
37. Will there be an air emission source from an Installation Restoration Program (IRP) removal or remedial project ?		X	
38. If yes, will a CERCLA permit exemption apply? (Remedial action conducted entirely on-site)		N/A	
VAPOR RECOVERY			
39. Does the project involve a gasoline filling station?		X	
40. Is a stage I and/or stage II vapor recovery required?		N/A	
41. Are permits required ?		X	
ACQUISITION OF LAND/BUILDINGS			
42. Does the project involve land or building acquisition?		X	
43. Has an environmental site survey been completed?	X		
44. Is the site known to have been used to store, handle, or dispose of hazardous material wastes?	X		Section of landfill was used for the aboveground storage of industrial waste.
45. Is the site, or has it been, occupied by bulk storage tanks?	X		Excavated underground tanks are stored on site.
46. Is asbestos present or likely to be present ?		X	
47. Are PCB transformers present ?		X	Formerly stored at this site.
48. Will necessary permits require environmental testing/cleanup ?		X	CERCLA exemption
49. Will public hearings be required ?		X	
DEMOLITION			
50. Does the project involve demolition ?		X	
51. Is asbestos present or likely to be present ?		X	
52. Will asbestos removal notification be required?		X	
53. Is lead paint present ?		X	
54. Are PCB transformers present ?		X	Formerly stored on concrete pad in southwest portion of landfill.
55. Are any permits required, including concurrence from State Historic Preservation Office on historic/cultural resources ?		X	

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
WITHDRAWAL FROM AQUIFERS			
56. Are undergound storage tanks present ?		X	
57. Does the project involve water withdrawal from an aquifer ?		X	
58. If yes, is the aquifer sole-source ?		N/A	
59. Is notification required ?		X	
60. Are any permits required ?		X	CERCLA exemption
61. Is water withdrawal a result of an Installation Restoration removal or remedial action project?		N/A	
62. If yes, will a CERCLA permit exemption apply?		N/A	
WATER PERMITS			
WATER WITHDRAWALS			
SURFACE WATER WITHDRAWALS			
63. Does the project involve the withdrawal of water from surface water sources for domestic (potable) uses of industrial usage ?		X	
64. Is a water allocation permit required ?		X	
65. Are construction permits required for intake structures?		X	
66. Is notification of regulatory agencies required?		X	
GROUND WATER WITHDRAWALS			
67. Does the project involve the direct withdrawal of groundwater for potable, industrial uses or groundwater clean-up ?		X	
68. Is a water allocation permit required ?		X	
69. Is notification of regulatory agencies required?		X	
70. Are well construction permits required ?		X	
TREATMENT FACILITIES			
71. Does the project include potable water storage (tanks, reservoirs) or treatment (disinfection, pH control, filtering) facilities or expansion of the basewide water distribution systems?		X	
72. If yes, are potable water construction/operating permits needed ?		X	
WASTEWATER DISCHARGES			
DOMESTIC SEWAGE			
73. Will domestic (sanitary) sewage be discharged from the project ?		X	
74. Does the project discharge to a sanitary sewage collection system ?		X	
75. Will new sewer mains be constructed or will the effluent flow increase ?		N/A	
76. Are construction, operating, or sewer extension permits required ?		X	
77. Does the discharge flow to Navy owned STP ?		X	

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
DOMESTIC SEWAGE (continued)			
78. Will the discharge affect the ability of the sewage treatment plant to meet the flow parameters of the NPDES permit ? (If yes, a new permit may be required)		X	
79. Is notification of regulatory agencies required?		X	
80. Does the discharge flow to a publicly owned treatment plant ?		N/A	
81. Is notification required ?		X	
82. Is a connection permit required ?		X	
83. Does the discharge flow to a septic system ?		N/A	
84. Is the septic system new ?		N/A	
85. Is a construction permit required ?		X	CERCLA exemption
86. Is a discharge (to groundwater) permit required?		X	
87. Is the septic system existing ?		N/A	
88. Does it have a permit ?		N/A	
89. Are there flow limitations ?		N/A	
90. Is notification of increased flow required ?		N/A	
91. Does the project involve the construction of a sewage treatment plant ?		X	
92. If, yes, is a NPDES permit required ?		N/A	
INDUSTRIAL DISCHARGES			
93. Is there going to be a discharge of industrial wastewater from the facility ? An industrial discharge can be considered any wastewater generated by any source other than sanitary facilities, such as sinks, urinals water closets, and floor drains. Examples are photographic labs, laundries, plating operations, pesticide-formulation operations, hospitals, explosive manufacturing, numerous organic and inorganic chemical processes, and cooling and blowdown water boilers.	X		Potentially dewatering wastewater
94. Is the discharge going to flow into a sanitary sewage collection system ?	X		
95. If yes, is pretreatment required ?	X		
96. If yes, is a permit required ? (local ordinances may require permits for any industrial connection)	X		
97. Is the discharge going to flow to a storm sewer system, surface water or groundwater ?	X		Dewatering wastewater will be discharged to POTW or surface water body.
98. If yes, is a NPDES permit requied ?		X	
99. Construction permits may be required for outfall structures or wells.		X	
100. Will there be a discharge to the sanitary sewer from an Installation Restoration program removal or remedial action project?	X		
101. If yes, is a permit required?	X		CERCLA exemption if discharge to surface water body.

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
STORM WATER DISCHARGES			
102. Facilities that "discharge storm water associated with industrial activity," includes any site where certain activities are performed. Projects which propose to perform any industrial activity may require (1) modification of an existing NPDES storm water permit or, (2) submission of an application for a new permit. NPDES permits will also be needed if a facility, currently without a permit, constructs an industrial facility.		X	
103. Does the project involve construction activities that disturb more than 5 acres?	X		
104. If yes, is a NPDES permit required?		X	CERCLA exemption
105. Will there be discharge to the storm sewer from an Installation Restoration Program removal or remedial action project?	X		Dewatering wastewater
106. If yes, will a NPDES permit be required?		X	CERCLA exemption
CORPS OF ENGINEERS PERMITS			
107. Does the project describe work in or adjacent to the coastal zone or aquatic sites such as, but not limited to, rivers, streams, lakes, creeks, ponds, estuaries, etc.?		X	Does not require Federal Section 404 Permit
108. Does the project describe work in or adjacent to wetlands?	X		
109. Is the project adjacent to or within a wetland or aquatic environment or will have an impact upon a wetland or aquatic environment, has a wetland delineation been completed? (by Northdiv Code 20)	X		Minor filling at edge of wetland to allow cap installation.
110. If the project will have an impact upon wetlands, or an aquatic environment, has a site approval been issued? (by Northdiv Code 20)		X	
111. Has the wetland delineation been confirmed by the U.S. Army Corps of Engineers or state regulatory agency?		X	Wetlands delineation of the site has been completed, but not confirmed by the COE or regulatory agencies.
112. If a coastal zone consistency determination is required has it been completed? (by Northdiv Code 20)		X	
113. Does the project require utility runs that might cross wetlands or navigable waters? (these may be included in other projects)		X	
114. Does the project include or require access roads that cross wetlands or navigable waters?		X	
115. Does the project include construction of intake/discharge structures or headwalls within a wetland or waterway?	X		Energy dispersion in wetlands for diversion trench

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
STATE WETLANDS PERMITS			
116. Does the state in which the project is sited have wetland and/or dredging regulations which may apply to the project?	X		
117. Does the project describe work within 100' of wetlands?	X		
STATE WATER QUALITY CERTIFICATION			
118. Does the project require state review and approval under the provisions of Section 401, of the Clean Water Act? (Water Quality Certification)		X	CERCLA exemption
APPLIED BIOLOGY PROJECTS			
119. Does the project include installation or maintenance of wood piles, poles, or ties?		X	
120. Is the project a waterfront structure, pier wharf or bulkhead?		X	
121. Does the project include wood structural components?		X	
122. Does the project include landscaping with plants or maintenance of turf, shrubs or trees?		X	
123. Does the project include the application of pesticides other than for the prevention of termites?		X	
124. Has preconstruction treatment to prevent termites (NFGS02284) been omitted from the specification?	X		
125. Is preconstruction treatment to prevent termites other than as specified in NFGS02285?		X	
ASBESTOS REMOVAL			
126. Does the project include the construction, repair or rehabilitation of food service or food storage facilities?		X	
127. Does the project involve potential distrubance of asbestos ?		X	
128. Has an asbestos survey been performed ?		X	
129. Does the project involve renovation, demolition or repair work	X		
130. Is federal, state or local notification required ?		X	
131. Are any state or local permits required ?		X	
132. Is third party monitoring required or recommended ?		X	
133. Is the NAVFAC spec section 02080 included and edited correctly ?		X	
LEAD PAINT REMOVAL			
134. Does the project involve potential disturbance of lead paint ?		X	
135. Has a lead paint survey been performed ?		X	
136. Does the project involve renovation, demolition or repair work ?		X	
137. Is federal, state or local notification required ?		X	
138. Are any state or local permits required ?		X	
139. Is the project at an activity where radon readings of greater than 4 picoCuries/liter have been found in existing buildings ?		X	

ENVIRONMENTAL PERMIT CHECKLIST
INTERIM REMEDIAL ACTION - AREA A CAP
(NORTHNAVFACENGCOMINST 5090.5A)

	YES	NO	COMMENTS
LEAD PAINT REMOVAL (continued)			
140. Does the project involve construction of a new building ?		X	Salt storage shed to be rebuilt
141. Is slab on grade construction involved ?		X	
142. Is the vapor barrier thickness 6 mil or greater ?		N/A	
143. Are all penetrations through the vapor barrier sealed around openings ?		N/A	